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# Comparison of general industry practices for lattice tower design and detailing

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## EXECUTIVE SUMMARY

### INTRODUCTION

This document has the purpose of collecting and comparing the current practices within the industry for the design and fabrication of lattice steel structures for transmission power lines. It also compares the standards most often utilised by the designers around the world, as well as those practices adopted by the industry for tower detailing not defined by the standards.

Frequently fabrication items are not specified by the standards and are intentionally left open, to give freedom to the designers and manufacturers to account for their local market conditions and/or industry particularities. Moreover, some numerical values defined by standards, especially partial safety material factors, can be amended by National Committees.

The concept of a standard has been understood by this CIGRE Working Group B2.08 “Transmission Line Structures” as the result of “knowledge” plus “consensus” of a group of professionals about a specific issue, for a certain period of time. Within this context, the objective of this work is to promote the knowledge by comparing the existing practices, and highlighting their differences and similarities.

This Technical Brochure does not intend to report the consensus on the practices, on any of the issues researched. Moreover, it does not make any judgment or suggestion by any one of this CIGRE Working Group members, on whether or not a specific data or answer is the most correct or convenient, whatsoever. Therefore, the main purpose of the document is to serve as a manual for consultation, leaving to the sole discretion and responsibility of the designers to make their own choices.

Nevertheless, it is reasonable to believe that the content of this Technical Brochure is very useful to all of those involved with the overhead transmission line supports, considering that, as published by CIGRE in [1], the action of the designers through their knowledge, the design practices they use, the standards and guidelines adopted, altogether can consist on a considerable source of discrepancies even when predicting the strength of a same steel support.

The reduction of these discrepancies, allows to better adjusted and more accurate values of the “strength factors” ($\Phi_R$) for the supports, relevant to the application of the probabilistic method proposed by IEC 60826 [2], leading to more reliable and economic designs.

**Key-words:** Overhead Lines, Lattice Towers, Standards, Industry Practices.

## PROBABILISTIC DESIGN OF OVERHEAD LINES

Since the 90's, and especially after the publication of the IEC 60826 [2] International Standard in 2003, more and more structural designers around the world have shifted to the probabilistic method as a basic philosophy for the design of overhead line supports, both for the new lines and sometimes for the uprating and/or upgrading studies of existing ones. Concurrently, it is desirable to have an appropriate framework to assess the probability of failure and by means of this, to evaluate line availability within the context of aiming for more reliable and economic designs.

IEC 60826 has established such framework. By knowing the effect of a climatic load (wind, ice) associated to a return period, it is possible to evaluate the probability of failure by comparing it to the exclusion limit (e =10%) of the component strength. Therefore, on this basis, IEC 60826 sets procedures to properly design to a target reliability level as well as to a desired strength coordination.

**Figure 1 – Different 500 kV Lattice Tower Types**

## STRENGTH FACTOR FOR SUPPORTS

In the recommended strength coordination, the suspension tower is assumed as being the weakest component of the transmission line system. To have such targets accomplished, it is obviously necessary to obtain accurate estimations of the strength factor ($\Phi_R$), coefficient of the IEC 60826 design equation. For the application of the probabilistic methods, the CIGRE Working Group B2.08 has been devoting significant efforts to properly understand, quantify and propose more accurate values for the strength factor $\Phi_R$ of the supports. For the determination of the factors $\Phi_R$ (adjusted for an exclusion limit of 10% at the distribution curve) it becomes necessary to know the statistical distribution curves for the supports' strength, in order that the means and respective standard deviations (coefficient of variation) can be calculated.

These statistical distribution studies relating to the strength of transmission line supports were first published by Paschen et al in 1988 [3] and Riera et al in 1990 [4]. From the Riera et all studies, a value of $\Phi_R$ = 0.93 was then established, which has been largely adopted in new transmission lines projects thereafter, especially in South America. Improvements on the estimation of the strength factor $\Phi_R$ has been permanently researched by the CIGRE WG B2.08 since the early 90's, which are primarily based upon investigations on the discrepancies of the strength of the supports and their main causes. Various studies [1, 5, 6, 7, 8] have been already concluded and published since then.

**Figure 2 – 500 kV Cross Rope Suspension Tower**

## DISCREPANCIES IN TOWER DESIGN

An important source of discrepancies in predicting the strength of lattice steel towers has been explained in the article published by the Electra Journal nº 138 – Oct.1991 [1], when different tower designers using their knowledge, experience, design tools, Standards and practices predicted the strength of two towers. As reported in that article, there are relevant discrepancies among the predictions of different designers, even when they all are requested to predict it for exactly the same structure. Such discrepancies can be partly explained by the different professional practices, Standards and guidelines they use.

In this regard, as previously announced, the main objective of this Technical Brochure is to identify, collect and compare those professional practices, as well as the standards and guidelines currently utilised around the world for the design of lattice steel supports for transmission lines.

## VARIATION IN CURRENT PRACTICES

In recent papers published by CIGRE, as “Assessment of Existing Overhead Line Supports”[9], and “Life Cycle Assessment (LCA) for Overhead Lines” [10], the galvanized lattice steel structures are named as the predominant type of supports utilised for the suspension of the conductors on the overhead high voltage transmission lines (Figures 1, 2, 3, 4, 5). In a simplified way, without regarding to the electrical phases configuration or the tower top geometry, it can be assumed that the lattice steel towers are generally composed, in weight, by 80 to 85% by angles (replaced in some cases by tubes), about 10% of joint plates, by 5 to 10% bolts and accessories and from 3 to 5% zinc coating (Figures 1.1, 1.2).

Apparently, it should not be expected substantial variation in the manufacturing processes and practices, as the work to be done is essentially simple and repetitive, as cutting angles and plates, punching or drilling holes to connect them, after the galvanizing process, with bolts and nuts to be parts of transmission lines supports.

This Technical Brochure shows, however, that, starting from the design, standards used by the designers, and passing on to the industry practices, there are some differences and variations which may result in significant different final outputs. As mentioned before, there are many important detailing elements which are not specified by the standards to allow sufficient freedom to the designers and manufacturers to account for their local market conditions. In these cases, as shown hereinafter, because they are intentionally left open to the discretion of the designer and manufacturer, the differences widen.

An important contribution of this Technical Brochure is to report on the current practices adopted in different countries, for those detailing elements which are not normally specified by the standards. In this context, it may become an important source of reference and advice for those who usually deal with the design and use of the overhead transmission line supports.

**Figure 3 – 500 kV Tower Under Testing**

## STANDARDS AND PRACTICES

As stated before, this Technical Brochure does not have the intent to be a normative document or even a design guide, with recommended practices to be followed by the industry. Therefore, its main objective is to promote the knowledge by reporting on the existing design practices within the industry and highlighting their differences and similarities.

Particularly, the following documents were analysed and compared within the scope of this Technical Brochure:

- ASCE 10-97 standard [11];
- European CENELEC Standard EN 50341-1 [12] and its National Normative Aspects EN 50341-3 [13];
- ECCS European Convention for Construction Steelwork no. 39 [15];
- Brazilian standard and practices;
- Korean standard and practices;

- Additional recommendations and practices from Finland, Iceland and Italy.

It should be also emphasized that, the additional recommendations and practices may represent just the national member opinion and, in many circumstances not necessarily a consensus about that item on that particular country. In any case, however, that means a professional knowledge and experience and the response can offer a good reference for this exercise of comparisons.

## DETAILING PRACTICES

Currently, for the design, calculation and detailing of an Overhead Line Lattice Support, the professionals must follow clients specifications, as well as, national and/or international relevant standards. In most of the cases, however, the information must be complemented by industry practices or supported by the professionals' knowledge and experience.

**Figure 4 – Galvanized lattice steel members**

In the context of this CIGRE Technical Brochure, various items of parameters and considerations currently necessary to be assumed for the design, calculation and detailing of overhead line supports were analysed and discussed.

In a first Part (Clauses 1 to 7), they comprise discussions on subjects such as:

- Maximum permitted length of members;
- Minimum thickness limit of members;
- Bolts (diameter, material, locking devices, minimum gauge, clearances in holes, thread length, bolt length outside the nut, step bolts, punching, assembly torque);
- Nuts (material, dimensions);
- Plane washers (material, dimensions);
- Profiles (profile types, material, fabrication tolerances);
- Piece marking (minimum mark height, depth of marking).

As an example, the item nr.1 of the Technical Brochure (see page 18) deals with de "Maximum Permitted Length of Members". It is explained why it is important to take care about that issue:

*"The maximum physical length of an individual member is generally controlled by the possible restrictions of manufacture, galvanizing, transport and erection.*

*Particular practical limitations are:*

- *Length of raw material;*
- *The ability to handle and maintain straightness;*
- *The size of the hot dip galvanizing bath;*
- *Transportation limits;*
- *In rare situations the maximum weight and handling restrictions.*

*As these are non-technical limitations often based on economics and local conditions, these limitations are indicated in the Industry Practices and Guidelines but generally not in the normative Standards."*

For this example, although no reference is made about the "maximum permitted length for members" on the relevant standards, the national industry practices give a clear indication on what has been adopted around the world about that.

**DESIGN PRACTICES**

In a second Part (Clauses 8 to 18) the guidelines for the design of lattice steel towers are compared between ASCE 10-97 [11], EN 50341-1 [12] and their National Normative Aspects EN 50341-3 [13], and ECCS no. 39 [15]. The additional recommendations of some other countries are also given.

Distinction has been made between:

- What is typical for all steel structures (Clauses 8 to 10):

    - Design of bolted connections (minimum bolt distances: distance between holes and to the end or the edge of the profile; structural design for shear, tension and bearing);

- o Design of tension members;
- o Design of compression members (allowable compression member; local and torsional buckling);

- What is more typical for lattice steel towers (Clauses 11 to 15):

  - o Design of buckling lengths (effective buckling length taking into account end restraints - number of bolts, discontinuity - and eccentricities; buckling lengths to consider for each bracing type; maximum slenderness ratios);
  - o Design of redundant members;
  - o Design of members only under tension;
  - o Design of double angles (tension or compression);
  - o Design of members withstanding vertical loads of men (horizontal and inclined members);

- What is more particular for lattice steel structures (Clauses 16 to 18):

  - o Design of members subjected to axial force and bending;
  - o Design of guyed lattice steel structures;
  - o Design of stub angles and anchor bolts for embedment in the concrete of foundations.

Table 1 gives an idea of which items are discussed in the Standards. ASCE mostly refers to ASTM when material properties are considered. EN 50341-1 refers to the Eurocodes for design of steel structures that are not typical for overhead lines.

**Table 1 – Standards considering OH Line structures**

| Lattice steel tower | | ASCE 10-97 [9] | ASTM [9] | EN 50341-1 EN 50341-3 [10, 11] | Eurocode EN 1993-1-1 [12] | ECCS [13] |
|---|---|---|---|---|---|---|
| Detailing | General | - | x | - | - | - |
| | Material | x | x | x | x | - |
| Design | Bolts | x | - (1) | x | x | x |
| | All steel structures | x | - | x | x | - |
| | Lattice towers | x | - | x | - | x |
| | Particular design | x | - | x | - | x (2) |
| (1) shear and tension provided<br>(2) combined axial load and bending not provided | | | | | | |

## CONCLUSIONS

As described above, all main specifications and assumptions that must be followed, assumed or taken into account for the design, calculation and detailing of overhead lattice steel supports were analyzed and discussed by the Working Group. As a result, a document containing many items and their sub-items was prepared, comprehending what is specified by the most international adopted standards related to overhead lines support design. In addition to that, the Technical Brochure still shows what has been adopted and/or used by five national industry practices from three different continents: America, Europe and Asia.

The CIGRÉ B2.08 Group believes that, even having no aim to be another standard or guide document, the Brochure can be a very useful reference for those directly involved with the design, calculation and fabrication of lattice overhead transmission line steel supports.

**Figure 5 – 500 kV Transmission lines**

# Part 1 – Detailing Practices

## 1 Maximum Permitted Length of Members

The maximum physical length of an individual member is generally controlled by the restrictions of manufacture, transport and erection.

Particular practical limitations are:

- The length of raw material;
- The ability to handle and maintain straightness;
- The size of the hot dip galvanizing bath;
- Transportation limits;
- In rare situations the maximum weight and handling.

As these are non-technical limitations and often based on economics, these limitations are indicated in the Industry Practices and Guidelines but not in the normative Standards.

**ASCE 10-97 Standard**
Not specified

**EN 50341-1:2001 - European CENELEC Standard**
Not specified

**EN 50341-3 - National Normative Aspects**
Not specified

**ECCS 39 Recommendations**
Not specified

**Brazilian Industry Practice**
9 m

**Italian Industry Practice**
9 m

**Finnish Industry Practice**
12 m (practice)

**Korean Industry Practice**
9 m (for angles)

**Icelandic Industry Practice**
12 m, but members longer than 9-10 are seldom used.

**Figure 1.1 – Galvanized members**

**Figure 1.2 – Galvanized pieces**

# 2 Minimum Thickness Limit of Members

The minimum thickness limit of the member affects (See Annex D):

- The life of the member as it affects the thickness of applied zinc;
- The local buckling of sections;
- The vulnerability to damage from manufacture, transport and maintenance;
- The minimum bearing and pullout capacity in a connection.

## 2.1 Members of Lattice Tower

**ASCE 10-97 Standard**

1/8” (3mm)

**EN 50341-1:2001 - European CENELEC Standard**

Not specified (see EN1993-1-1 below).

**EN 50341-3 - National Normative Aspects (See also Table D)**

AT → EN 50341-3-1 – Clause 7.3.1/AT.1 – 4 mm for open sections; 3.5 mm for hollow sections

DE → EN 50341-3-4 – Clause 7.3.1/DE.1 – The thickness of components shall not be less than 4 mm. In case of hollow sections used for tower members the thickness may be reduced to 3 mm if effective protection against corrosion is ensured.
Minimum member thickness and minimum bolt diameter shall be specified under due consideration of potential corrosion effects, especially if thin-walled cold formed sections are used.

NO → EN 50341-3-16 – Clause 7.3/NO.1 – Minimum thickness of plates shall be given in the Project Specification. Minimum thickness of main members shall be 5 mm and for redundant members 4 mm. Minimum thickness for hollow sections shall be 4 mm. By hollow sections care shall be taken for drainage.

SE → EN 50341-3-18 – Clause 7.2/SE.2 – Minimum thickness of material in steel members shall be 4 mm for open sections and 3 mm for hollow sections. For redundant members these values can be reduced to 3 mm and 2.5 mm respectively. Hollow sections shall be equipped with a drainage system.

SI → EN 50341-3-21 – Clause 7.3.1/SI.1 – Component thickness shall not be less than 4 mm. Where hollow profiles are used the thickness may be reduced to 3 mm while ensuring a good anticorrosion protection.

**EN 1993-1-1 (Clause 1.1.2)**

EN 1993-1-1 gives basic design rules for steel structures with material thicknesses $t \geq 3$ mm. For cold formed thin gauge members and plate thicknesses $t < 3$ mm see EN 1993-1-3.

**ECCS 39 Recommendations**

Not specified

**Brazilian Industry Practice**

1/8” (3mm)

**Korean Industry Practice**

Leg members: 5mm
Other members: 3mm

**Finnish Industry Practice**

4 mm

**Icelandic Industry Practice**

4 mm

**Italian Industry Practice**

4 mm

## 2.2 Gusset Plates

**ASCE 10-97 Standard**

3/16” (5mm)

**EN 50341-1:2001 - European CENELEC Standard**

Not specified

**ECCS 39 Recommendations (5.2.1)**

The thickness of the gusset plate should not be less than the thickness of any one member connected to it.

**Brazilian Industry Practice**

Minimum 1/8” (3 mm)
The thickness of the gusset plates should not be less than the thickness of any one member connected to it.

**Korean Industry Practice**

Minimum 4 mm
The thickness of the gusset plate should not be less than any of the connected members.

**Finnish Industry Practice**

4 mm

**Icelandic Industry Practice**

The thickness of the gusset plates should be at least 1mm thicker than any of the connected members.

**Italian Industry Practice**

The thickness of the gusset plates should be at least 1 mm thicker than any of the connected members.

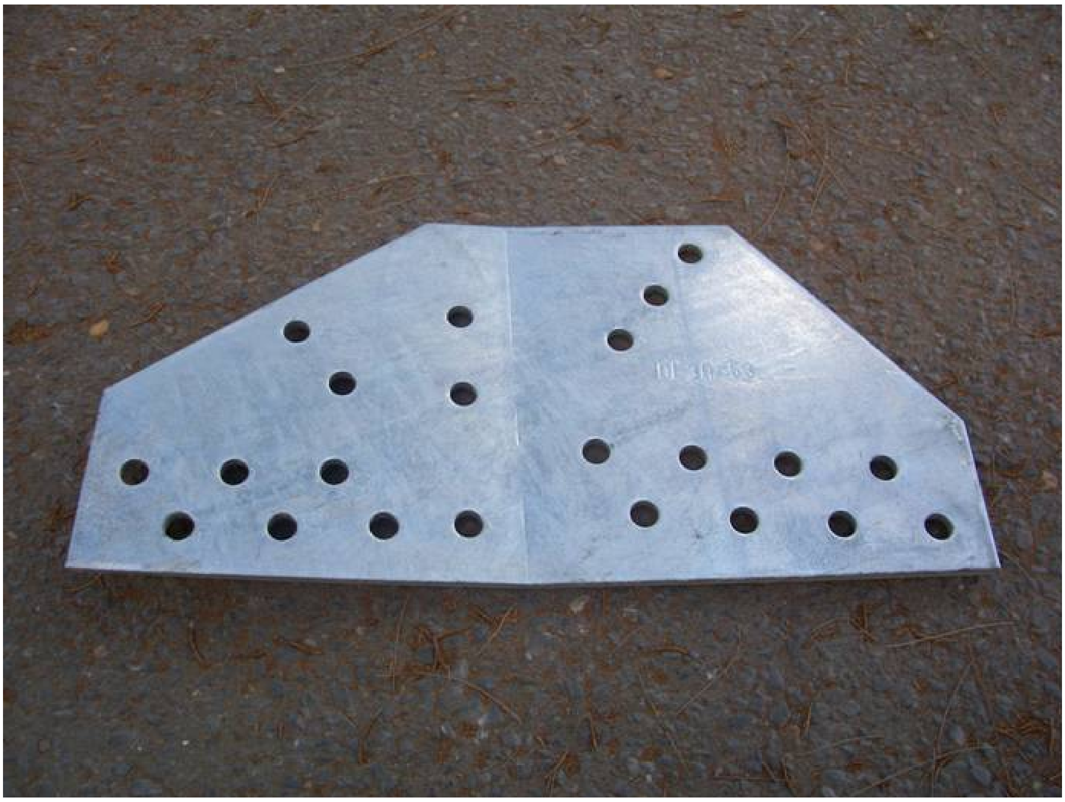


**Figure 2.1 – Typical bent gusset plate**

## 2.3 Foundation Stub Angles

**ASCE 10-97 Standard**

Not specified

**EN 50341-1:2001 - European CENELEC Standard**

Not specified

**ECCS 39 Recommendations**

Not specified

**Brazilian Industry Practice**

The buried stub is to be at least as thick as the leg member above ground.

**Korean Industry Practice**

6 mm.

The buried stub is to be at least as thick as the leg member above ground.

**Finnish Industry Practice**

No recommendation

**Icelandic Industry Practice**

No recommendation

**Italian Industry Practice**

No recommendation

**Figure 2.2 – Stub angle assembly**

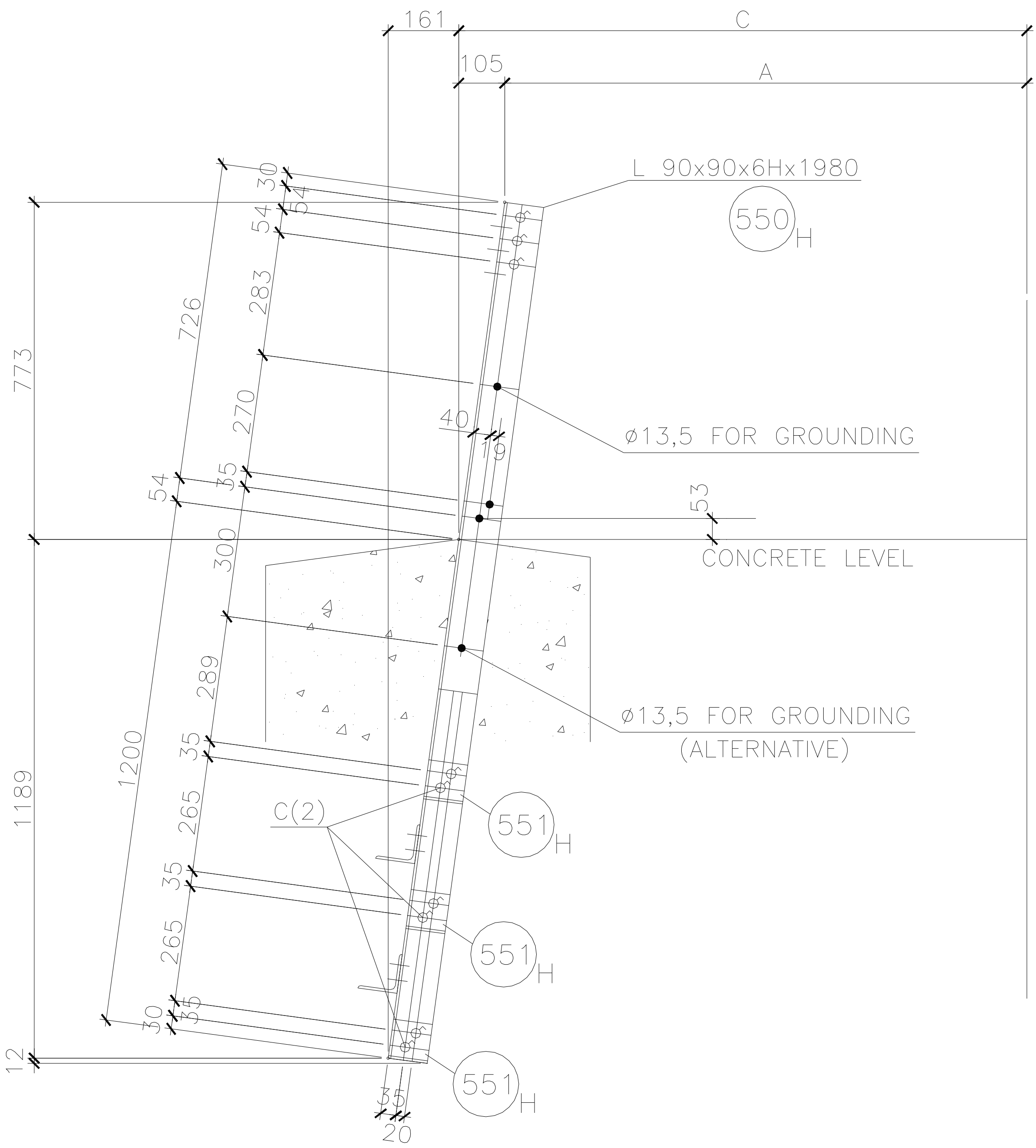



**Figure 2.3 – Typical stub angle detailing**

## 2.4 Buried Foundation Grids

### 2.4.1 Non-corrosive soils

<u>**ASCE 10-97 Standard**</u>
3/16” (5.0 mm)

**EN 50341-1:2001 - European CENELEC Standard**

Not specified

**ECCS 39 Recommendations**

Not specified

**Brazilian Industry Practice**

5 mm (Metric Units), or 3/16” (Imperial Units)

**Figure 2.4 – Steel grillage foundation**

**Korean Industry Practice**

No recommendation

**Finnish Industry Practice**

No recommendation

**Icelandic Industry Practice**

No recommendation

**Italian Industry Practice**

No recommendation

### 2.4.2 Corrosive Soils

**ASCE 10-97 Standard**

3/16” (5mm)

**EN 50341-1:2001 - European CENELEC Standard**

Not specified

**ECCS 39 Recommendations**

Not specified

**Brazilian Industry Practice**

In corrosive soils, investigations shall be carried out to determine the appropriate thickness to be adopted. The minimum thickness is never smaller than 5 mm.

**Korean Industry Practice**

No recommendation

**Finnish Industry Practice**

No recommendation

**Icelandic Industry Practice**

No recommendation

**Italian Industry Practice**

No recommendation

## 2.5 Corrosive Environments

**ASCE 10-97 Standard**

Not specified.

**EN 50341-1:2001 - European CENELEC Standard (7.2.3 & 7.9)**

Except where otherwise specified, when steels are to be galvanized, in order to avoid dull dark grey and excessively thick coating which may result in an increased risk of coating damage, it is recommended that the maximum silicon (Si) and phosphorus (P) contents meet the requirements of EN ISO 1461, sub-clause C.1.4.

Unless otherwise stated in the Project Specification, after completion of all fabrication procedures, all steel material shall be hot-dip galvanized and tested in accordance with EN ISO 1461.

Unless otherwise stated in a Project Specification, when pieces are too large or difficult to galvanize, they shall be protected against corrosion by thermal spraying a zinc coating over the base metal, performed according to EN ISO 14713 and in accordance with EN 22063. Zinc deposit thickness shall be not less than 80 µm. When this system is used, the inside surface of hollow sections shall also be protected.

When a paint coating is to be applied in plant after hot-dip galvanizing of steel structures (Duplex System), this coating shall be done as soon as possible.

**EN 50341-3: National Normative Aspects**

DE → EN 50341-3-4 – Clause 7.9/DE.1 – The selection of the protection system shall be decided upon depending on the requirements which apply for the individual line and shall be specified in a Project Specification.

DK → EN 50341-3-5 – Clause 7.9/DK.1 – Specific requirements for corrosion protection shall be given in the Project Specification.

IS → EN 50341-3-12 – Clause 7.9.2/IS.1 – The corrosion protection is in accordance with the Project Specification.

NO → EN 50341-3-16 – Clause 7.9.2/NO.1 – Galvanizing of steel parts including guys shall be according to the Project Specification.

SE → EN 50341-3-18 – Clause 7.2/SE.3 – If steel material is not rust resistant it shall be protected by galvanizing or painting.

Clause 7.9/SE.1 – Since in general painting is an inferior corrosion protection compared to galvanizing, it is recommended to increase the thickness of the material for painted parts. This is important in marine atmosphere and in corrosive industrial atmosphere, where it must be considered if only painting is sufficient as corrosion protection.

Clause 7.9.2/SE.1 – Zinc coating shall be in accordance with SS-EN ISO 1461. Damages in galvanizing surface can be repaired by spray galvanizing or by painting twice with zinc rich paint, zinc powder 92-95 % of dry weight. The damaged surface shall be carefully cleaned by sand blasting or similar and dried and preheated before treatment.

Clause 7.9.3/SE.1 – The zinc layer of metal spraying shall not be less than the requirements for hot-dip galvanizing.

EE → EN 50341-3-20 – Clause 7.9.3/EE.1 – Zinc deposit thickness by using metal spraying shall be not less than in hot-dip galvanizing.

SI → EN 50341-3-21 – Clause 7.9/SI.1 – Protection system shall be selected following the requirements that belong to the overhead line in question and shall be prescribed in the Project Specification.

PL → EN 50341-3-22 – Clause 7.2.3/PL.1 – Refer to PN-EN ISO 1461:2000.

**ECCS 39 Recommendations**

Not specified

**Brazilian Industry Practice**

Each situation shall be assessed accordingly.

**Korean Industry Practice**

No recommendation

**Finnish Industry Practice**

No additional recommendation

**Icelandic Industry Practice**

No additional recommendation

**Italian Industry Practice**

No additional recommendation

# 3 Bolts

Bolts used in lattice structures have, generally, dimensions and material quality to make the most efficient use of the bolt and the connected material. The use of metric or imperial size bolts is dependent on the country or design and/or manufacture.

## 3.1 Diameter

**ASCE 10-97 Standard and ASCE Manual 52**

1/2", 5/8", 3/4", 7/8", 1" (as in ASTM A394)
M12, M16, M20, M24, M30 (as in ASTM A394M)

**EN 50341-1:2001 - European CENELEC Standard**

Not specified

**EN 50341-3: National Normative Aspects**

DE → EN 50341-3-4 – Clause 7.3.6.2/DE.3 – M12, M16, M20, M24, M27, M30. Diameters of bolts less than 12 mm are not permissible for structurally loaded members. The minimum strength quality for bolts M12 is 5.6. The maximum permissible diameter of threads of mechanically loaded bolts and the diameters of related boreholes are determined by the width of the angle legs and may be taken from Table 3.1.

Clause 7.3.1/DE.1 – If weakened by boreholes, angle sections with a width below 35 mm and flat bars with a width below 30 mm are inadmissible.

**Table 3.1 – Maximum bolt diameters for given widths of angle legs (DE)**

| Dimension of bolt (mm) | M12 | M16 | M20 | M24 | M27 | M30 |
|---|---|---|---|---|---|---|
| Maximum diameter of borehole (mm) | 14 | 18 | 22 | 26 | 29 | 32 |
| Minimum width of angle leg (mm) | 35 | 50 | 60 | 70 | 75 | 80 |

DK → EN 50341-3-5 – Clause 7.9/DK.1 – Specific requirements for corrosion protection shall be given in the Project Specification.

NO → EN 50341-3-16 – Clause 7.9.2/NO.1 – Galvanizing of steel parts including guys shall be according to the Project Specification.

SI → EN 50341-3-21 – Clause 7.3.6.2/SI.3 – M12, M16, M20, M24, M27, M30. Screws with less than 12 mm diameter and with hardness class below 5.6 may not be used for fastening of construction supporting elements. Maximum allowable screw and nut as well as bore diameters depend on the width of the ray of an angular bar and can be obtained from Table 3.1.

**ECCS 39 Recommendations**

Not specified

**Brazilian Industry Practice**

1/2”, 5/8”, 3/4”, 7/8”, 1”

M12, M14, M16, M20, M24

**Korean Industry Practice**

M16, M20, M24

Standard KS B1002 & KS B1012 (KS: Korean Industrial Standards)

**Finnish Industry Practice**

M12, M16, M20, M24, M30 (ISO 262)

**Icelandic Industry Practice**

M12, M14, M16, M20, M24, M27, M30

**Italian Industry Practice**

M12, M16, M20, M24, M27, M30

## 3.2 Material

**ASCE 10-97 Standard**

ASTM A394:

**Type “0”**

Low or medium carbon steel, zinc-coated (hot dip):

$f_u$ = 510.0 N/mm$^2$

$f_v$ = 380.5 N/mm$^2$ (for the thread)

$f_v$ = 316.5 N/mm$^2$ (for the body)

**Type “1”**

Medium carbon steel, with heat treatment, zinc-coated (hot dip):

$f_u$ = 827.5 N/mm$^2$

$f_v$ = 513.0 N/mm$^2$ (thread and body)

**Type “2”**

Low carbon martensilic steel, zinc-coated (hot dip):

$f_u$ = 827.5 N/mm$^2$

$f_v$ = 513.0 N/mm$^2$ (thread and body)

**Type “3”**

Corrosion resistant steel, with heat treatment:

$f_u$ = 827.5 N/mm$^2$

$f_v$ = 513.0 N/mm$^2$ (thread and body) where:

$f_u$ = ultimate tensile strength of bolt
$f_v$ = allowable shear stress

**EN 50341-1:2001 - European CENELEC Standard (7.2.1)**

In accordance with Eurocode 3 EN 1993-1-1 and EN 10025.

**EN 50341-3: National Normative Aspects**

DE → EN 50341-3-4 – Clause 7.2/DE.2 – In general, only the strength qualities 4.6, 5.6, 8.8 and 10.9 according to DIN EN 20898-1 shall be used.

**ECCS 39 Recommendations**

Not specified

**Brazilian Industry Practice**

Use of both ASCE and ISO 898 standards. Use of only one type of standard for project and a maximum number of two different diameters in each structure.

Use of ISO 898 specification, as below (See also Table 3.2):

**ISO 898-1 Class 5.8**

Low or medium carbon steel, zinc-coated:

$f_u$ = 520 N/mm$^2$

$f_v$ = 381 N/mm$^2$ (for the thread)

$f_v$ = 322 N/mm$^2$ (for the body)

**ISO 898-1 Class 8.8**

Medium carbon steel, with heat treatment, zinc-coated:

$f_u$ = 800 N/mm$^2$ (diameter $\leq$ 16mm)

$f_u$ = 830 N/mm$^2$ (diameter > 16mm)

$f_v$ = 496 N/mm$^2$ (thread and body, diameter $\leq$16mm)

$f_v$ = 515 N/mm$^2$ (thread and body, diameter >16mm)

**Korean Industry Practice**

Standard KS B0233 & KS B0234
Zinc-coated (hot dip)

M16 & M20 (Gr. 5.8) $f_u$ = 520 N/mm$^2$, $f_y$= 420 N/mm$^2$ (thread and body)
M24 (Gr. 8.8) $f_u$ = 830 N/mm$^2$, $f_y$= 660 N/mm$^2$ (thread and body)

$f_u$ = ultimate tensile strength of the bolt
$f_y$ = yield strength

**Finnish Industry Practice**

Grade 8.8 (ISO 898-1)

$f_y$ = 640 MPa, $f_u$ = 800 MPa

**Icelandic Industry Practice**

Grade 8.8 with $f_u$ = 800 MPa and $f_y$ = 640 MPa

**Italian Industry Practice**

Grade 6.8
Normal type

$f_u = 600\ N/mm^2$
$f_y = 480\ N/mm^2$

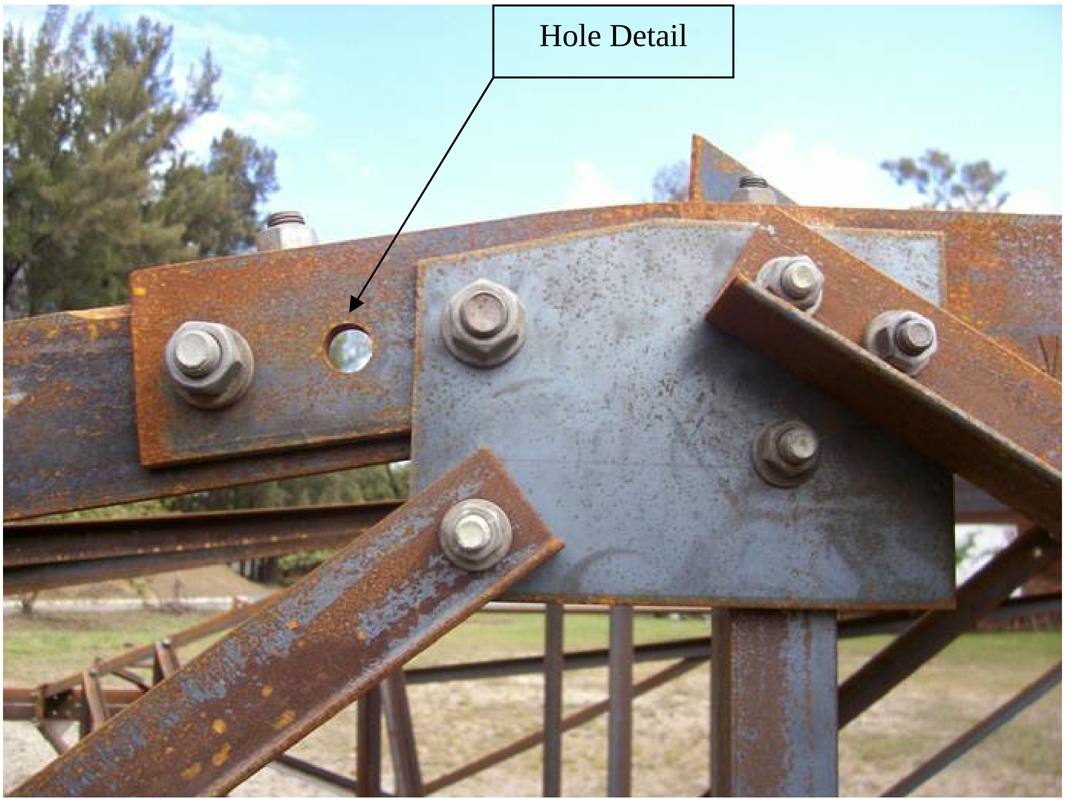


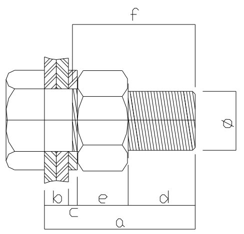

**Figure 3.1 – Typical lattice tower joint**

# Table 3.2 – Length of Metric Bolts - Design Table

## LENGTHS OF ISO 898 BOLTS

### ACCORDING TO BRAZILIAN PRACTICE

| M 12 | | | | M 16 | | | | M 20 | | | | M 24 | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| b | a | c | d | b | a | c | d | b | a | c | d | b | a | c | d |
| 6 | 30(A) | 2x3 | 7 | 6 | 35(B) | 2x3 | 8 | 6 | 35(B) | 3 | 8 | 6 | 40( C) | 3 | 10 |
| 7 | " | 3 | 9 | 7 | " | 2x3 | 7 | 7 | 40( C) | 2x3 | 9 | 7 | " | 3 | 9 |
| 8 | " | 3 | 8 | 8 | " | 3 | 9 | 8 | " | 2x3 | 8 | 8 | 45(D) | 2x3 | 10 |
| 9 | " | 3 | 7 | 9 | " | 3 | 8 | 9 | " | 3 | 10 | 9 | " | 2x3 | 9 |
| 10 | 35(B) | 2x3 | 8 | 10 | " | 3 | 7 | 10 | " | 3 | 9 | 10 | " | 3 | 11 |
| 11 | " | 2x3 | 7 | 11 | 40( C) | 2x3 | 8 | 11 | " | 3 | 8 | 11 | " | 3 | 10 |
| 12 | " | 3 | 9 | 12 | " | 2x3 | 7 | 12 | 45(D) | 2x3 | 9 | 12 | " | 3 | 9 |
| 13 | " | 3 | 8 | 13 | " | 3 | 9 | 13 | " | 2x3 | 8 | 13 | 50(E) | 2x3 | 10 |
| 14 | " | 3 | 7 | 14 | " | 3 | 8 | 14 | " | 3 | 10 | 14 | " | 2x3 | 9 |
| 15 | 40( C) | 2x3 | 8 | 15 | " | 3 | 7 | 15 | " | 3 | 9 | 15 | " | 3 | 11 |
| 16 | " | 2x3 | 7 | 16 | 45(D) | 2x3 | 8 | 16 | " | 3 | 8 | 16 | " | 3 | 10 |
| 17 | " | 3 | 9 | 17 | " | 2x3 | 7 | 17 | 50(E) | 2x3 | 9 | 17 | " | 3 | 9 |
| 18 | " | 3 | 8 | 18 | " | 3 | 9 | 18 | " | 2x3 | 8 | 18 | 55(F) | 2x3 | 10 |
| 19 | " | 3 | 7 | 19 | " | 3 | 8 | 19 | " | 3 | 10 | 19 | " | 2x3 | 9 |
| 20 | 45(D) | 2x3 | 8 | 20 | " | 3 | 7 | 20 | " | 3 | 9 | 20 | " | 3 | 11 |
| 21 | " | 2x3 | 7 | 21 | 50(E) | 2x3 | 8 | 21 | " | 3 | 8 | 21 | " | 3 | 10 |
| 22 | " | 3 | 9 | 22 | " | 2x3 | 7 | 22 | 55(F) | 2x3 | 9 | 22 | " | 3 | 9 |
| 23 | " | 3 | 8 | 23 | " | 3 | 9 | 23 | " | 2x3 | 8 | 23 | 60(G) | 2x3 | 10 |
| 24 | " | 3 | 7 | 24 | " | 3 | 8 | 24 | " | 3 | 10 | 24 | " | 2x3 | 9 |
| 25 | 50(E) | 2x3 | 8 | 25 | " | 3 | 7 | 25 | " | 3 | 9 | 25 | " | 3 | 11 |
| 26 | " | 2x3 | 7 | 26 | 55(F) | 2x3 | 8 | 26 | " | 3 | 8 | 26 | " | 3 | 10 |
| 27 | " | 3 | 9 | 27 | " | 2x3 | 7 | 27 | 60(G) | 2x3 | 9 | 27 | " | 3 | 9 |
| 28 | " | 3 | 8 | 28 | " | 3 | 9 | 28 | " | 2x3 | 8 | 28 | 65(H) | 2x3 | 10 |
| 29 | " | 3 | 7 | 29 | " | 3 | 8 | 29 | " | 3 | 10 | 29 | " | 2x3 | 9 |
| 30 | 55(F) | 2x3 | 8 | 30 | " | 3 | 7 | 30 | " | 3 | 9 | 30 | " | 3 | 11 |
| 31 | " | 2x3 | 7 | 31 | 60(G) | 2x3 | 8 | 31 | " | 3 | 8 | 31 | " | 3 | 10 |
| 32 | " | 3 | 9 | 32 | " | 2x3 | 7 | 32 | 65(H) | 2x3 | 9 | 32 | " | 3 | 9 |
| 33 | " | 3 | 8 | 33 | " | 3 | 9 | 33 | " | 2x3 | 8 | 33 | 70(I) | 2x3 | 10 |
| 34 | " | 3 | 7 | 34 | " | 3 | 8 | 34 | " | 3 | 10 | 34 | " | 2x3 | 9 |
| 35 | 60(G) | 2x3 | 8 | 35 | " | 3 | 7 | 35 | " | 3 | 9 | 35 | " | 3 | 11 |
| 36 | " | 2x3 | 7 | 36 | 65(H) | 2x3 | 8 | 36 | " | 3 | 8 | 36 | " | 3 | 10 |
| 37 | " | 3 | 9 | 37 | " | 2x3 | 7 | 37 | 70(I) | 2x3 | 9 | 37 | " | 3 | 9 |
| 38 | " | 3 | 8 | 38 | " | 3 | 9 | 38 | " | 2x3 | 8 | 38 | 75(J) | 2x3 | 10 |
| 39 | " | 3 | 7 | 39 | " | 3 | 8 | 39 | " | 3 | 10 | 39 | " | 2x3 | 9 |
| 40 | 65(H) | 2x3 | 8 | 40 | " | 3 | 7 | 40 | " | 3 | 9 | 40 | " | 3 | 11 |
| 41 | " | 2x3 | 7 | 41 | 70(I) | 2x3 | 8 | 41 | " | 3 | 8 | 41 | " | 3 | 10 |
| 42 | " | 3 | 9 | 42 | " | 2x3 | 7 | 42 | 75(J) | 2x3 | 9 | 42 | " | 3 | 9 |
| 43 | " | 3 | 8 | 43 | " | 3 | 9 | 43 | " | 2x3 | 8 | 43 | 80(K) | 2x3 | 10 |
| 44 | " | 3 | 7 | 44 | " | 3 | 8 | 44 | " | 3 | 10 | 44 | " | 2x3 | 9 |
| 45 | 70(I) | 2x3 | 8 | 45 | " | 3 | 7 | 45 | " | 3 | 9 | 45 | " | 3 | 11 |
| | | | | | | | | 46 | " | 3 | 8 | 46 | " | 3 | 10 |
| | | | | | | | | 47 | 80(K) | 2x3 | 9 | 47 | " | 3 | 9 |
| | | | | | | | | 48 | " | 2x3 | 8 | 48 | 85(L) | 2x3 | 10 |
| | | | | | | | | 49 | " | 3 | 10 | 49 | " | 2x3 | 9 |
| | | | | | | | | 50 | " | 3 | 9 | 50 | " | 3 | 11 |
| | | | | | | | | 51 | " | 3 | 8 | 51 | " | 3 | 10 |




| Ø | e | f |
|---|---|---|
| M12 | 11 | 22 |
| M16 | 15 | 26 |
| M20 | 18 | 30 |
| M24 | 21 | 34 |

NOTES: Maximum thread length inside the washer 1mm
Minimum clearance to tight 2mm

| | | |
|---|---|---|
| ISO 898/1, Class 5,8<br>ISO 898/2, Class 5<br>DIM.: ISO 4016, ISO 4034 | LENGTH TABLE FOR METRIC BOLTS WITH PLAN WASHERS c=3mm | TABLE |

## 3.3 Locking Devices

Locking methods are used to prevent nuts from loosening tightening due to dynamic or thermal load, and in extreme circumstances tampering by vandals.

- Adequate tightening of the nuts;
- Deformation of the thread;
- Application of thread locking material;
- Welding;
- Spring washer;
- Tamper proof nut;
- Swaged nut.

**ASCE 10-97 Standard**

Not specified

**EN 50341-1:2001 - European CENELEC Standard**

Not specified

**EN 50341-3: National Normative Aspects**

DE → EN 50341-3-4 – Clause 7.3.6.2/DE.1 – Bolted connections shall be secured against loosening. This can be achieved by an adequate tightening of the nuts. Spring washers may be used as an indication for sufficient tightening.

SE → EN 50341-3-18 – Clause 7.3.7/SE.2 – The nut shall be locked by punching or chisel hack on the threads or in another secure way.

EE → EN 50341-3-20 – Clause 7.3.6.2.3/EE.1 – Connections with bolts shall be secured against loosening in service.

SI →EN 50341-3-21 – Clause 7.3.6.2/SI.1 – Screw nuts shall be protected from unscrewing.

**ECCS 39 Recommendations**

Not specified

**Brazilian Industry Practice**

The use of locking device is specified by each utility choosing between palnuts or lock washers.

In the case of palnuts, the material may be in SAE 1010/1020 steel. For lock washers, the material shall be according to SAE-1055/1065 steel.

**Korean Industry Practice**

No recommendation

**Finnish Industry Practice**

Deformation of the thread

**Icelandic Industry Practice**

Spring washer of type DIN 127B. It shall be noted that this spring washer is not accepted for bolts of grade 8.8 according to the DIN standards. The Icelandic practice is however acceptable when using this washer with grade 8.8 bolts.

**Italian Industry Practice**

No recommendation

### 3.4 Minimum Gauge (Structure and Foundation)

The minimum gauge is important from the corrosion point of view to guarantee the minimum distance of the connection even in an aggressive environmental.

**ASCE 10-97 Standard**

Specifies only Standard A394 (minimum gauge 1/2") without any restrictions on its use.

**EN 50341-1:2001 - European CENELEC Standard**

Not specified

**ECCS 39 Recommendations**

Not specified

**Brazilian Industry Practice**

M12 (1/2")

In corrosive environments or soils, appropriate measurements should be taken to obtain the degree of corrosiveness.

**Korean Industry Practice**

M16 (23mm)

**Finnish Industry Practice**

No recommendation

**Icelandic Industry Practice**

No recommendation.

**Italian Industry Practice**

No recommendation

### 3.5 Clearances in holes

Currently (normal OHL supports) the hole diameters are larger than the bolt shank to allow easy assembly. The increase in bolt size must allow for tolerance of fabrication and free zinc remaining in the hole after galvanizing.

**ASCE 10-97 Standard**

1/16" (1.6 mm)

**EN 50341-1:2001 - European CENELEC Standard**

Not specified

**EN 50341-3: National Normative Aspects**

DE → EN 50341-3-4 – Clause 7.3.6.2/DE.1 – 2.0 mm for M12, M16, M20, M24, M27, M30

DK → EN 50341-3-5 – Clause J.11/DE.1 – 1.5 to 2.0 mm for M16 and M20

**ECCS 39 Recommendations**

Not specified

**Brazilian Industry Practice**

1.5 mm for metrical bolts and 1/16” for bolts in inches

**Korean Industry Practice**

1.5 mm for M16, M20
2.0 mm for M24

**Finnish Industry Practice**

1.5 mm when d < 22 mm
2 mm otherwise

**Icelandic Industry Practice**

1.5 mm for M12, M16, M20
2.0 mm for M24, M27, M30

**Italian Industry Practice**

1.5 mm for M12, M16, M20, M24
2.0 mm for M27, M30

**Figure 3.2 – Typical main member's connections**

## 3.6 Thread Length

It is important that there is sufficient thread length to allow installation of the nut and any washers. However if the thread length is too long, load between connected members is applied to a threaded section of a bolt and there will be a reduced capacity compared to the capacity of the unthreaded section. Avoidance of this requires coordination between the total thickness (grip length) of the members, the length of the bolt and the threaded length (See Table 3.2).

**ASCE 10-97 Standard**

Not specified, but quotes standard A394, which also refers to the ANSI standard for bolts:

**Table 3.3 – Thread lengths for bolts (ANSI)**

| ANSI B18.2.1/81 | | ANSI B18.2.3.5M/79 | |
|---|---|---|---|
| d | Length inches (mm) | d | Length (mm) |
| 1/2" | 1"1/4 (31.8) | M12 | 30.0 |
| 5/8" | 1"1/2 (38.1) | M14 | 34.0 |
| 3/4" | 1"3/4 (44.5) | M16 | 38.0 |
| 7/8" | 2" (50.8) | M20 | 46.0 |
| 1" | 2"1/4 (57.2) | M24 | 54.0 |

In some circumstances the above values result in the thread falling within the shear plane and an adjustment in strength is required

**EN 50341-1:2001 - European CENELEC Standard**

Not specified.

**EN 50341-3: National Normative Aspects**

SE → EN 50341-3-18 – Clause 7.3.7/SE.2 – For bolts with shear load the threads shall end outside the connected parts. The outlet of the threads may end up to 5 mm into one of the connected parts with a maximum of one third of the thickness of the connected part.

**ECCS 39 Recommendations**

Not specified

**Brazilian Industry Practice**

Thread lengths shall be specified in such a way that the thread is excluded from the shear plane (See Table 3.2).

**Korean Industry Practice**

| | |
|---|---|
| M16 | 35 mm |
| M20 | 40 mm |
| M24 | 50 mm |

**Finnish Industry Practice**

No recommendation but practice is to use DIN 7990

| | DIN 7990 |
|---|---|
| **d** | **Length (mm)** |
| M12 | 17.75 |
| M16 | 21.0 |
| M20 | 23.5 |
| M24 | 26.0 |
| M30 | 30.5 |

The thread should not penetrate further than half the thickness of the member closest to the nut.

**Icelandic Industry Practice**

Thread length shall be selected in such a way that the thread is excluded from the shear plane.

**Italian Industry Practice**

No recommendation

### 3.7 Bolt length outside the nut or the lock washer (palnut)

The length of thread protruding from the nut is an indication that the correct length of bolt has been selected and installed. A length of threaded bolt protruding from the nut also allows the full function of the nut when the bolt is in tension or shear. Capture of moisture can occur if the end of the bolt is recessed into the nut and facing upwards. Too much thread protruding is an indication that the members may not be fully clamped and the nut is bound on the limit of the threads.

**ASCE 10-97 Standard**

Not specified

**EN 50341-1:2001 - European CENELEC Standard**

Not specified

**ECCS 39 Recommendations**

Not specified

**Brazilian Industry Practice**

3 to 12 mm with a minimum of one thread (p)
p = pitch of thread

**Korean Industry Practice**

Three threads (3 p)

| | |
|---|---|
| M16 | 3 x 2.0 = 6.0 mm |
| M20 | 3 x 2.5 = 7.5 mm |
| M24 | 3 x 3.0 = 9.0 mm |

**Finnish Industry Practice**

No recommendation. In practice two threads (2 p)

**Icelandic Industry Practice**

Minimum two threads (2 p) and preferably three threads (3 p).

**Italian Industry Practice**

No recommendation

### 3.8 Step bolts

#### 3.8.1 General

Usually step bolts are used to climb the towers for maintenance/inspections works.

**ASCE 10-97 Standard**

According to ASTM A394, all dimensions must be settled in agreement with the purchaser.
Material shall be of type “0”, unless the purchaser and the supplier agree upon any other material.

**EN 50341-1:2001 - European CENELEC Standard**

Not specified

**EN 50341-3: National Normative Aspects**

See Clause 15 (AT, BE, DE, NL, NO, SE, CZ, EE, SI).

**ECCS 39 Recommendations**

Not specified

**Brazilian Industry Practice**

Use step bolts with 2 nuts and the following head dimensions:
Diameter: M16 (or 5/8“) ± 0.7 mm
Length: 220 mm ± 2 mm
Head diameter: 35 mm ± 2 mm
Head height: 10 mm ± 1 mm

**Korean Industry Practice**

Use step bolts with 2 nuts

**Finnish Industry Practice**

No recommendation

**Icelandic Industry Practice**

No recommendation

**Italian Industry Practice**

No recommendation

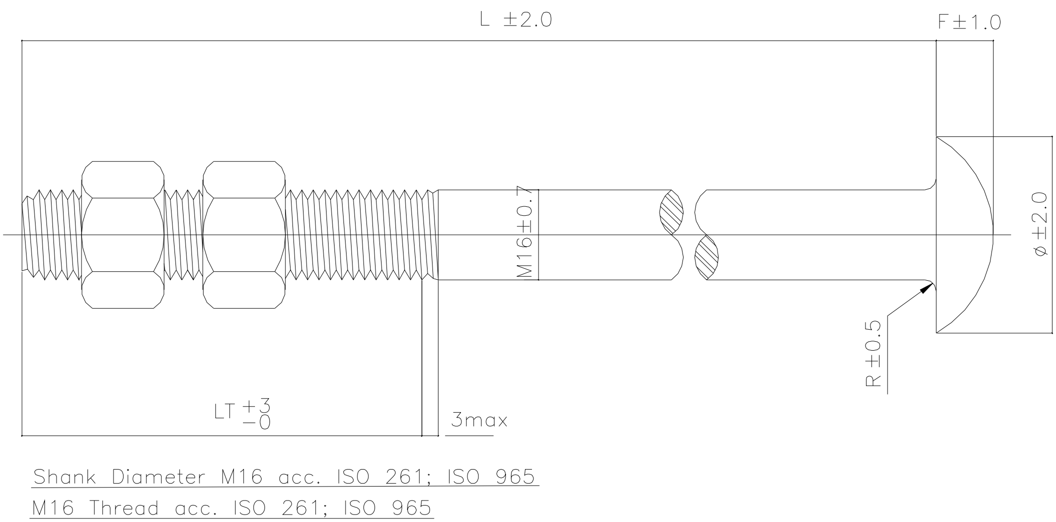



**Figure 3.3 – Typical step bolt detailing**

**Figure 3.4 – Step bolts assembling**

### 3.8.2 Material

**<u>ASCE 10-97 Standard</u>**

ASTM A394 - Type “0” or as agreed between purchaser and supplier

**<u>EN 50341-1:2001 - European CENELEC Standard</u>**

Not specified

**<u>ECCS 39 Recommendations</u>**

Not specified

**<u>Brazilian Industry Practice</u>**

ASTM A394 – type “0” or SAE – 1020 (SAE: Society Automotive Engineering)

**Korean Industry Practice**
The same as item 3.2

**Finnish Industry Practice**
5.8 (ISO 898-1)

**Icelandic Industry Practice**
No recommendation, but usually of grade 8.8

**Italian Industry Practice**
No recommendation

### 3.8.3 Minimum diameter size

**ASCE 10-97 Standard**
In agreement with the purchaser

**EN 50341-1:2001 - European CENELEC Standard**
Not specified

**EN 50341-3: National Normative Aspects**
See Clause 15 (DE: M24; NL: M20; SI: M24)

**ECCS 39 Recommendations**
Not specified

**Brazilian Industry Practice**
M16 (5/8”)

**Korean Industry Practice**
M16

**Finnish Industry Practice**
M16

**Icelandic Industry Practice**
M16

**Italian Industry Practice**
No recommendation

### 3.8.4 Spacing

**Standard ASCE 10-97**
In agreement with the purchaser

**EN 50341-1:2001 Overhead electrical lines exceeding AC 45 kV - CENELEC**

Not specified

**EN 50341-3: National Normative Aspects**

See Clause 15 (DE & SI: 333 mm with max. spacing of 403 mm and max. variation of 100 mm; NL: max. spacing of 300 mm and min. spacing of 250 mm with max. variation of 15 mm; PL: max. spacing of 400 mm).

**ECCS 39 Recommendations**

Not specified

**Brazilian Industry Practice**

Between 350 and 450 mm

**Korean Industry Practice**

Between 350 and 450 mm

**Finnish Industry Practice**

Between 300 and 400 mm

**Icelandic Industry Practice**

Avoid different spacing between bolts.

**Italian Industry Practice**

Between 300 and 400 mm

#### 3.8.5 Serviceable length

**ASCE 10-97 Standard**

In agreement with the purchaser

**EN 50341-1:2001 - European CENELEC Standard**

Not specified

**EN 50341-3: National Normative Aspects**

See Clause 15 (DE, NL, SI: 150 mm)

**ECCS 39 Recommendations**

Not specified

**Brazilian Industry Practice**

140 mm

**Korean Industry Practice**

180 mm for M16, 260 mm for M24

**Finnish Industry Practice**

150 mm

**Icelandic Industry Practice**
150 mm

**Italian Industry Practice**
190 mm

### 3.8.6 Minimum height above ground for step bolts

**ASCE 10-97 Standard**
In agreement with the purchaser

**EN 50341-1:2001 - European CENELEC Standard**
Not specified

**Recommendations For Angles in Lattice Transmission Towers – ECCS 1985**
Not specified

**Brazilian Industry Practice**
Above 3 m

**Korean Industry Practice**
1.5 m

**Finnish Industry Practice**
2 m

**Icelandic Industry Practice**
No recommendation

**Italian Industry Practice**
The first horizontal bracing above the ground

## 3.9 Holes punching

Due considerations shall be made on defining the limit of material thickness between the methods of drilling and punching. The effect of the material ductility has to be duly considered (See also Annex D).

**ASCE 10-97 Standard**
Not specified

**EN 50341-1:2001 - European CENELEC Standard**
Not specified

**EN 50341-3: National Normative Aspects (See also Table D)**

DE → EN 50341-3-4 – Clause 7.3.6.2/DE.2 – Steel according to EN 50341-3-3, Clause 7.2/DE.1 is sufficiently ductile and may be punched. However, this does not apply to holes for bolts in angle sections and plates of more than 12 mm thickness.
Structural members of cross-arms permanently loaded in tension shall not be punched.

NL → EN 50341-3-15 – Clause 11.8/NL.1 – Punching of holes is not allowed for the fittings attachment points at the towers. Edges of holes shall be rounded off. The average impact-test value of three tests on Sharpy-V notch test pieces, shall be at least 27 J at 0 °C.

NO → EN 50341-3-16 – Clause 7.3/NO.1 – Holes for bolts may normally be punched in angles and plates up to 12 mm thickness.

SE → EN 50341-3-18 – Clause 7.3.7/SE.2 – Holes may be punched in steel up to a maximum thickness of 13 mm and in qualities in accordance with SS-EN 10 025+A1 and SS-EN 10 113 or steel with corresponding characteristics, if the thickness is less than the hole diameter. For other steels it shall be checked that punching does not cause cracks or brittleness.
In other cases the hole shall be drilled or alternatively punched with a diameter 3 mm smaller than the nominal diameter and thereafter drilled to the final diameter.

SI → EN 50341-3-21 – Clause 7.3.6.2/SI.2 – Punching may not be used for thicknesses above 12 mm. Construction elements of cross-arms that will be under constant stress in tension may not be punched.

**ECCS 39 Recommendations**

Not specified

**Brazilian Industry Practice**

Holes for bolts are normally punched depending on the angles and plates thicknesses and the steels quality involved.

Currently limits are:

| | |
|---|---|
| A36 steel: | $t \leq d_0$ |
| A572 steel, grade 50: | $t \leq d_0$ - 1/16” |
| A572 steel, grade 60: | $t \leq d_0$ - 1/16” |

where: t - thickness of angles and plates limited to 16 mm
$d_0$ - hole diameter

On the other cases, holes are drilled or, alternatively punched with a smaller diameter and drilled afterwards to the final diameter.

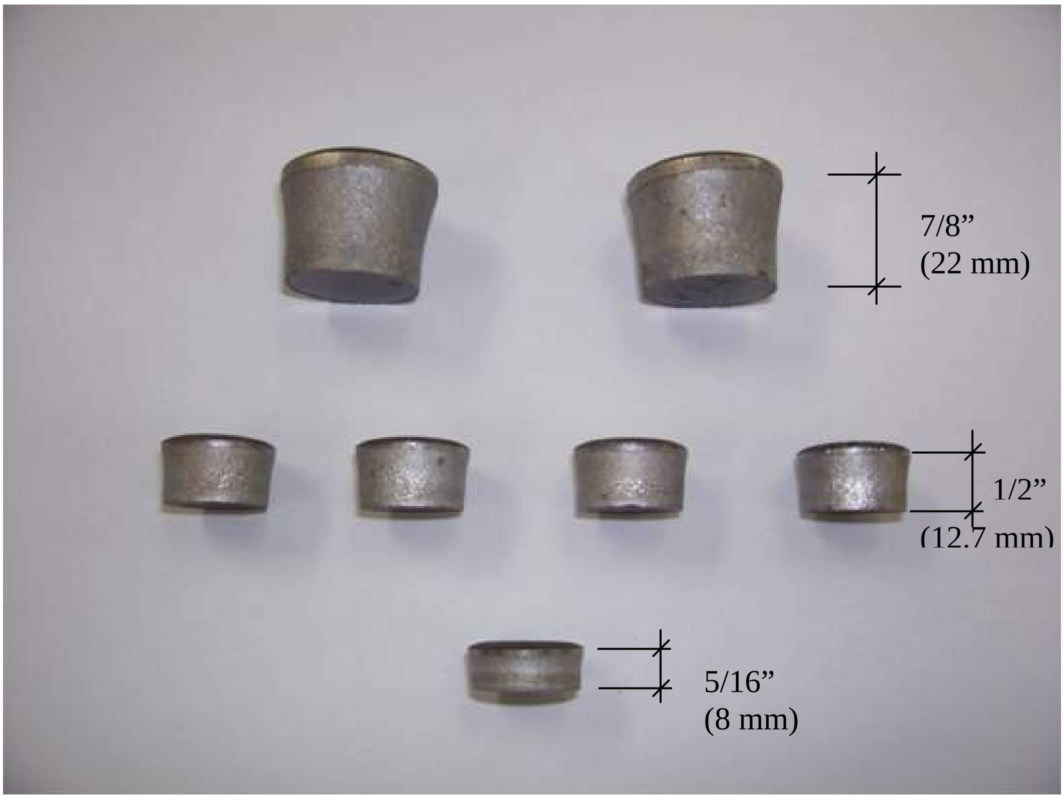



**Figure 3.5 – Punched material of holes**

**Figure 3.6 – Punched holes**

**Korean Industry Practice**

t - maximum thickness in one punching operation = 12 mm.

**Finnish Industry Practice**

S235 steel: $t \leq d_0$ & $t \leq 19$ mm

S355 steel: $t \leq d_0$ & $t \leq 14$ mm

**Icelandic Industry Practice**

Punching of bolt holes is permitted up to 13 mm thickness for S355 if member is not subjected to fatigue and the diameter of the hole does not exceed the thickness of the material.

**Italian Industry Practice**

Fe 360 steel: $t \leq 18$ mm

Fe 510 steel: $t \leq 16$ mm

For higher thickness it is possible to punch only a preliminary hole with a diameter 3mm lower than the final diameter for drilling after.

### 3.10 Assembly Torque (Bolt tightening)

In the majority of the towers, the connections are bearing type and slip to take up any clearances is not critical. Therefore the assembly torque value is important not for the resistance of the connection, but to minimize possible loosening of the nut due to vibrations.

**ASCE 10-97 Standard**

Not specified

**EN 50341-1:2001 - European CENELEC Standard**

Not specified

**ECCS 39 Recommendations**

Not specified

**Brazilian Industry Practice**

The following practical formula may be used to relate the torque with the bolt tension (elastic analysis):

$$T = K\, P\, d$$

where: T - torque

P - tension caused by the torque

d - nominal diameter of the bolt

K - coefficient depending on the thread angle and the friction coefficient of materials (adopted = 0.20)

(Note): *Forces produced by maximum torques may be expressed as a percentage of the ultimate tensile strength of the bolts ($f_u$ ) as follows:*

| | | |
|---|---|---|
| *Type “0”* | *Body : 35% $f_u$* | *thread : 45% $f_u$* |
| *Types “1”, “2”, “3”* | *Body : 21% $f_u$* | *thread : 28% $f_u$* |

Table 3.4 contains the maximum and minimum recommended torques and the associated tension forces produced on the bolts:

**Table 3.4 – Torques and tension forces - Brazilian practices**

| d | TORQUE [N.m] | | Tension Force [kN] | |
|---|---|---|---|---|
| | MIN. | MAX. | MIN. | MÁX. |
| M12 | 30 | 45 | 12.50 | 18.75 |
| M14 | 50 | 75 | 17.86 | 26.79 |
| M16 | 75 | 115 | 23.44 | 35.94 |
| M20 | 130 | 220 | 32.50 | 55.00 |
| M24 | 200 | 390 | 41.67 | 81.25 |
| 1 / 2 " | 35 | 55 | 13.78 | 21.65 |
| 5 / 8 " | 70 | 105 | 22.05 | 33.07 |
| 3 / 4 " | 120 | 190 | 31.50 | 49.87 |
| 7 / 8 " | 180 | 300 | 40.49 | 67.49 |
| 1 " | 250 | 450 | 49.21 | 88.58 |

**Korean Industry Practice**

T = K P d

where: T - torque ( N.m)

P - tension caused by the torque

$P = (0.6 \sim 0.7)\, f_y\, A_s$

d - nominal diameter of the bolt

K - coefficient depending on the thread angle and of the friction coefficient of materials: K = 0.12

K = 0.15 without oil

**Table 3.5 – Tightening torques – Korean practices**

| d | K = 0.12 Torque (Nm) Min. | K = 0.12 Torque (Nm) Max. | K = 0.15 Torque (Nm) Min. | K = 0.15 Torque (Nm) Max. |
|---|---|---|---|---|
| M16 | 70 | 90 | 90 | 110 |
| M20 | 140 | 170 | 180 | 210 |
| M24 | 310 | 370 | 390 | 460 |

* M16 & M20: Gr. 5.8, M24: Gr. 8.8

**Finnish Industry Practice**

Practice for bolts (Grade 8.8) in lattice towers:

**Table 3.6 – Tightening torques – Finnish practices**

| d | Torque T (Nm) | d | Torque T (Nm) | d | Torque T (Nm) |
|---|---|---|---|---|---|
| M12 | $70^{\pm 10}$ | M16 | $170^{\pm 20}$ | ≥ M20 | 300 |

**<u>Icelandic Industry Practice</u>**

Practice for bolts of grade 8.8

**Table 3.7 – Tightening torques – Icelandic practices**

| d | Torque T (Nm) |
|---|---|
| M12 | 75 |
| M16 | 170 |
| M20 | 330 |
| M24 | 560 |

**<u>Italian Industry Practice</u>**

**Table 3.8 – Tightening torques - Italian practices**

| d | Torque T Nm | d | Torque T Nm | d | Torque T Nm |
|---|---|---|---|---|---|
| M12 | 70 | M18 | 200 | M24 | 430 |
| M14 | 100 | M20 | 280 | M27 | 570 |
| M16 | 150 | M22 | 360 | M30 | 700 |

# 4 Nuts

## 4.1 Material

**ASCE 10-97 Standard**

According to ASTM A563

**EN 50341-1:2001 - European CENELEC Standard (7.2.1)**

As per Eurocode 3 EN 1993-1-1 and EN 10025

**ECCS 39 Recommendations**

Not specified

**Brazilian Industry Practice**

According to ASTM A563

| Bolt type | Nut |
|---|---|
| ASTM A394 Type “0" | ASTM  A563 grade A |
| ASTM A394 Type “1” | ASTM A563 grade DH |

According to ISO 898-2

| Bolt type | Nut |
|---|---|
| ISO 898-1 Class 5.8 | ISO 898-2 Class 5 |
| ISO 898-1 Class 8.8 | ISO 898-2 Class 8 |

**Korean Industry Practice**

In accordance with KS B0234, KS B1012 (KS:  Korean Industrial Standards).

**Finnish Industry Practice**

Property class 8 (ISO 898-2)

**Icelandic Industry Practice**

Property class 8 (ISO 898-2)

**Italian Industry Practice**

No recommendation

## 4.2 Dimensions

**ASCE 10-97 Standard**

According to ASTM A563

**EN 50341-1:2001 - European CENELEC Standard (7.2.1)**

As per Eurocode 3 EN 1993-1-1 and EN 10025

**ECCS 39 Recommendations**

Not specified

**Brazilian Industry Practice**

According to ASTM A563

| Bolt type | Nut |
|---|---|
| ASTM A394 Type “0" | ASTM  A563 grade A |
| ASTM A394 Type “1” | ASTM A563 grade DH |

According to ISO 898-2

| Bolt type | Nut |
|---|---|
| ISO 898-1 Class 5.8 | ISO 898-2 Class 5 |
| ISO 898-1 Class 8.8 | ISO 898-2 Class 8 |

**Korean Industry Practice**

In accordance with KS B0234, KS B1012 (KS:  Korean Industrial Standards).

**Finnish Industry Practice**

Property class 8 (ISO 898-2)

**Icelandic Industry Practice**

Property class 8 (ISO 898-2)

**Italian Industry Practice**

No recommendation

**Figure 4.1 – Typical galvanized tower nuts**

# 5 Plane Washers

Washers are currently used in overhead line lattice supports bolted connections, aiming for protection of the galvanized surfaces during the tightening operation and better loads distribution. They should preferably be installed on the side connection where the tightening will occur.

### 5.1 Material

**ASCE 10-97 Standard**

Not specified

**EN 50341-1:2001 - European CENELEC Standard (7.2.1)**

In accordance with Eurocode 3 EN 1993-1-1 and EN 10025 and EN 10113.

**ECCS 39 Recommendations**

Not specified

**Brazilian Industry Practice**

According to ASTM A283 or other carbon steels of low, medium or high strength.

**Korean Industry Practice**

In accordance with KS B1326 (KS: Korean Industrial Standards).

**Finnish Industry Practice**

No additional recommendation (S235JRG2)

**Icelandic Industry Practice**

No additional recommendation

**Italian Industry Practice**

No additional recommendation

### 5.2 Dimensions

**ASCE 10-97 Standard**

Not specified

**EN 50341-1:2001 - European CENELEC Standard**

Not specified

**ECCS 39 Recommendations**

Not specified

**Brazilian Industry Practice**

Washers shall be circular or square according to ANSI B18.22.1 - “Type B-Narrow” for bolts in inches or according to ISO 7091 for metric bolts.

The plane washers thickness may vary from a minimum of 3 mm to a maximum of 6.4 mm, with a dimensional tolerance of ± 0.4 mm. There must be used no more than two different thicknesses of plane washers in each structure.

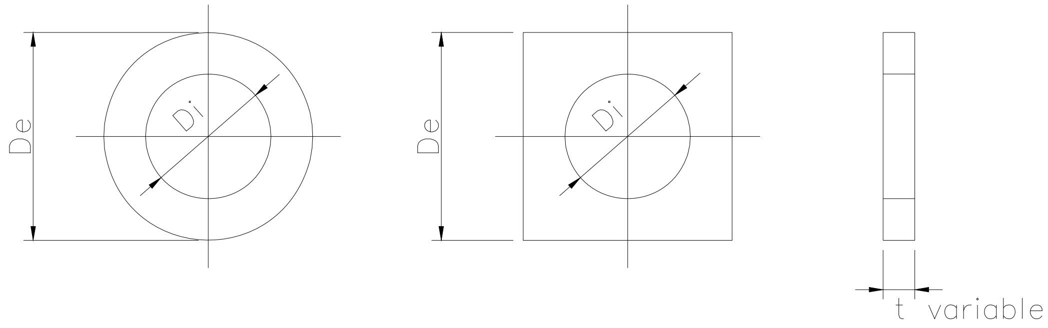



**Figure 5.1 – Types of washers – Brazilian practices**

**Table 5.1 – Washers dimensions and tolerances - Brazilian practices**

| d | $D_e$ | $D_i$ | d | $D_e$ | $D_i$ |
|---|---|---|---|---|---|
| 1/2” | 25.4 ± 0.4 | 14.3 ± 0.4 | M12 | 23.5 ± 0.5 | 13.5 ± 0.4 |
| 5/8” | 32.0 ± 0.5 | 17.5 ± 0.5 | M14 | 27.7 ± 0.5 | 15.5 ± 0.5 |
| 3/4” | 35.2 ± 0.5 | 20.7 ± 0.5 | M16 | 29.5 ± 0.5 | 17.5 ± 0.5 |
| 7/8” | 37.6 ± 0.5 | 23.8 ± 0.5 | M20 | 36.5 ± 0.5 | 22.0 ± 0.5 |
| 1” | 44.7 ± 0.5 | 27.0 ± 0.5 | M24 | 43.5 ± 0.5 | 26.0 ± 0.5 |

**Korean Industry Practice**

In accordance with KS B1326 ( KS : Korean Industrial Standards)

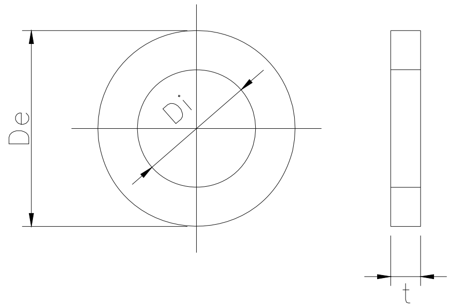



**Figure 5.2 – Washers dimensions – Korean practices**

**Table 5.2 – Washers dimensions tolerances - Korean practices**

| d | $D_e$ | $D_i$ | t |
|---|---|---|---|
| M16 | 30.0 -0.8 | 18.0 +0.7 | 3.2 ±0.4 |
| M20 | 37.0 -1.0 | 22.0 +0.8 | 3.2 ±0.4 |
| M24 | 44.0 -1.0 | 26.0 +0.8 | 4.5 ±0.5 |

**Finnish Industry Practice**

In accordance with DIN 7989, with thickness 6 mm (practice).

**Icelandic Industry Practice**

Two washers are used; one flat washer DIN 1441 and one spring lock washer DIN 127b.

**Italian Industry Practice**

No recommendation

# 6 Profiles

Profiles to be used in the fabrication of overhead lines lattice supports shall be shaped by means of hot-rolling or cold-forming processes.

## 6.1 Type of Profiles for Steel Members

**ASCE 10-97 Standard**

Hot-rolled angles and cold formed profiles

**EN 50341-1:2001 - European CENELEC Standard (7.2.3)**

Hot-rolled and cold formed angles

**ECCS 39 Recommendations (3)**

Rolled angle and cold formed angles

**Brazilian Industry Practice**

Hot-rolled angles and hot-rolled channel sections
In specific cases and upon approval by the purchaser, cold formed profiles may be used.

**Korean Industry Practice**

Hot-rolled angles and cold formed profiles

**Finnish Industry Practice**

Hot-rolled angles and cold formed profiles

**Icelandic Industry Practice**

Hot-rolled angles and cold formed profiles

**Italian Industry Practice**

Hot-rolled and cold formed angles

## 6.2 Material

The steels to be used in the fabrication of overhead lines lattice supports shall be of the low carbon steel types, which present good resistance and ductility to guarantee strength and workability.

**ASCE 10-97 Standard**

- Structural steel: ASTM A36.
- High-strength low-alloy structural steel: ASTM A242.
- Structural steel with 42.000 psi minimum yield point: A529.
- Hot-rolled carbon steel sheet and strip, structural quality: ASTM A570.
- High-strength low-alloy structural Columbium-Vanadium steels of structural quality: A572.
- High-strength low-alloy structural steel with 50,000 psi minimum yield point to 4-in. thick: A588.
- Steel sheet and strip, hot-rolled and cold-rolled, high-strength low-alloy with improved corrosion resistance: ASTM A606.

– Steel sheet and strip, hot-rolled and cold-rolled high-strength low-alloy, Columbium or Vanadium, or both, hot-rolled and cold-rolled: ASTM A607.

**Figure 6.1 – Raw material – Steel angles stock**

**EN 50341-1:2001 - European CENELEC Standard (7.2.1)**

Generally in accordance with Eurocode 3 EN 1993-1-1, EN 10025 and EN 10149-1

**EN 50341-3: National Normative Aspects**

DE → EN 50341-3-4 – Clause 7.2/DE.1 – In general only the structural steel types S235JO and S235J2G3/G4 as well as S355JO and S355J2G3/G4 according to DIN EN 10025 shall be used.

DK → EN 50341-3-5 – Clause 7.2.8/DK.1 – Structural steel of other quality is allowed as specified in the Project Specification.

NL → EN 50341-3-15 – Clause 7.2.3/NL.1 – Si-contents < 0,04% or > 0,12% and ≤ 0,30%.NO → EN 50341-3-16 – Clause 7.2/NO.1 – Materials shall be selected according to the Project Specification.

SE → EN 50341-3-18 – Clause 7.2/SE.1 – The material shall be selected in accordance with the referenced codes in clause EN 50341-3-18 and SS-EN Standards with NAD(S).

EE → EN 50341-3-20 – Clause 7.2.8/EE.1 – Structural steels with other quality may be permitted by the Project Specification.

PL → EN 50341-3-22 – Clause 7.2/PL.1 – Materials used in the fabrication of transmission line supports shall comply with PN-EN 10025-1:2007, PN-EN 10025-2:2007, PN-EN 10025-3:2007:2007 and PN-EN 10020:2003.

**ECCS 39 Recommendations (2)**

These recommendations apply to steel with the following yield stresses $\sigma_r$ for thicknesses t < 30 mm: Fe 360, $\sigma_r$ = 240 N/mm$^2$; Fe 430, $\sigma_r$ = 265 N/mm$^2$; Fe 510, $\sigma_r$ = 360 N/mm$^2$.

**Brazilian Industry Practice**

According to ASCE 10-97 and Brazilian Standards NBR 8850

**Korean Industry Practice**

According to KS D3503 (KS: Korean Industrial Standards):

- Mild : SS 400
- High tensile : SS 540
- Gusset plates : SS 400 & SWS 490

**Finnish Industry Practice**

Hot-rolled angles:

- S235 JRG2 EN 10025
- S355 J0 EN 10025

Cold formed angles:

- EN 10149-1…3

**Icelandic Industry Practice**

S355 J0

**Italian Industry Practice**

No recommendation

## 6.3 Tolerances

**ASCE 10-97 Standard**

ASTM-36 is quoted as a reference for milling tolerances.

No fabrication tolerances are given but the recommendation is to take them into consideration.

Milling Tolerances according to ASTM A6/A6M

**Table 6.1 – ASTM A6M and ASTM A6 Flange tolerances**

| ASTM A6M | | ASTM A6 | |
|---|---|---|---|
| Flange Width (mm) | Tolerance (mm) | Flange Width (Inches) | Tolerance (Inches) |
| b ≤ 50 | +1.0<br>-1.0 | b ≤ 2” | +3/64”<br>-3/64” |
| 50 < b < 75 | +2.0<br>-2.0 | 2” < b < 3” | +1/16”<br>-1/16” |
| 75 ≤ b ≤ 100 | +3.0<br>-2.0 | 3” ≤ b ≤ 4” | +1/8”<br>-3/32” |
| 100 < b ≤ 150 | +3.0<br>-3.0 | 4” < b ≤ 6” | +1/8”<br>-1/8” |
| 150 < b ≤ 200 | +5.0<br>-3.0 | 6” < b ≤ 8” | +3/16”<br>-1/8” |

**EN 50341-1:2001 - European CENELEC Standard**

Not specified (see EN1993-1-1 below).

**EN 1993-1-1 (Clause 3.2.5)**

The dimensional and mass tolerances of rolled steel sections, structural hollow sections and plates should conform with the relevant product standard, ETAG or ETA unless more severe tolerances are specified. For structural analysis and design the nominal values of dimensions should be used.

**ECCS 39 Recommendations**

Not specified

**Brazilian Industry Practice**

Milling tolerances as per ASTM A6/A6M (Table 3.11). Fabrication tolerances according to Figure 6.2 and Table 6.1.

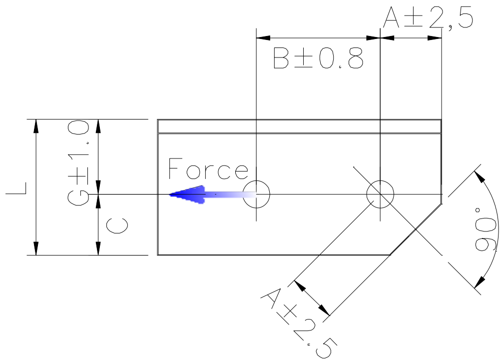



**Figure 6.2 – Tolerances on bolted connections distances**

**Table 6.2 – Tolerances for flange widths b (See L in Figure 6.2)**

| ASTM A6M | | |
|---|---|---|
| ± 1 | to | b ≤ 50 mm |
| ± 2 | to | 50 mm < b < 75 mm |
| +3 or -2 | to | 75 mm ≤ b ≤ 100 mm |
| ± 3 | to | 100 mm < b ≤ 150 mm |
| +5 or -3 | to | b > 150 mm |

| ASTM A6 | | |
|---|---|---|
| ± 1/32 in | to | b ≤ 1 in |
| ± 3/64 in | to | 1 in < b ≤ 2 in |
| ± 1/16 in | to | 2 in < b < 3 in |
| +1/8 in or -3/32 in | to | 3 in ≤ b ≤ 4 in |
| ± 1/8 in | to | 4 in < b ≤ 6 in |
| +3/16 in or -1/8 in | to | b > 6 in |

**Korean Industry Practice**

Fabrication Tolerances:

Distance between holes : ± 1.0 mm

Gauge (G): ± 1.0 mm

**Finnish Industry Practice**

EN 10056-2 (milling tolerances – Table 6.3.):

**Table 6.3 – EN 10056-2 Flange tolerances**

| EN 10056-2 | |
|---|---|
| Flange Width (mm) | Tolerance (mm) |
| b ≤ 50 | ± 1.0 |
| 50 < b ≤ 100 | ± 2.0 |
| 100 < b ≤ 150 | ± 3.0 |
| 150 < b ≤ 200 | ± 4.0 |
| 200 < b | +6.0/-4.0 |

Fabrication tolerances (company practice – Table 6.4):

**Table 6.4 – Distances between bolts**

| Distance between bolt groups (bolts) (mm) | Tolerance (mm) |
|---|---|
| 0≤ d ≤ 2000 | ± 1.0 |
| 2000 < d ≤ 4000 | ± 1.5 |
| 4000 < d | ± 2.0 |

Tolerances within the bolt group ± 1.0 mm for end and edge distances as well as distances between holes.

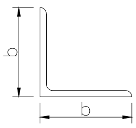
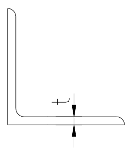
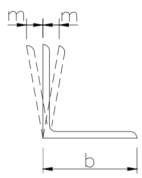
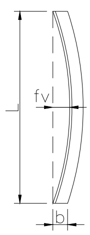
**Icelandic Industry Practice**

No recommendation

**Italian Industry Practice**

No recommendation

## Table 6.5 – Dimensional tolerances according to ASTM-A6 and NBR 6109

<table>
<tr><th colspan="11">DIMENSIONAL TOLERANCES ACCORDING TO ASTM A6 AND NBR 6109</th></tr>
<tr><th rowspan="3">DESIGNATION</th><th rowspan="3">FIGURE</th><th rowspan="3">DIMENSIONS (mm)</th><th colspan="8">TOLERANCES (mm)</th></tr>
<tr><th colspan="2">NBR-6109</th><th colspan="6">ASTM A6</th></tr>
<tr><th>RESTRICT<br>R</th><th>NORMAL<br>N</th><th colspan="3">ANGLE mm</th><th colspan="3">ANGLE pol</th></tr>
<tr><td rowspan="5">LENGTH OF FLANGE (b)</td><td rowspan="5"></td><td>b ≤ 50</td><td>+ 1,0<br>- 1,0</td><td>+ 1,5<br>- 1,5</td><td colspan="3">+ 1,0<br>- 1,0 R</td><td colspan="3">+ 1,2<br>- 1,2</td></tr>
<tr><td>50 < b < 75</td><td>+ 1,5<br>- 1,5</td><td>+ 3,0<br>- 2,5</td><td colspan="3">+ 2,0<br>- 2,0 R</td><td colspan="3">+ 1,6<br>- 1,6</td></tr>
<tr><td>75 ≤ b ≤ 100</td><td>+ 1,5<br>- 1,5</td><td>+ 3,0<br>- 2,5</td><td colspan="3">+ 3,0<br>- 2,0 R</td><td colspan="3">+ 3,2<br>- 2,4</td></tr>
<tr><td>100 < b ≤ 150</td><td>+ 2,0<br>- 2,0</td><td>+ 3,0<br>- 2,5</td><td colspan="3">+ 3,0<br>- 3,0 N</td><td colspan="3">+ 3,2<br>- 3,2</td></tr>
<tr><td>150 < b ≤ 200</td><td>+ 3,0<br>- 3,0</td><td>+ 5,0<br>- 3,0</td><td colspan="3">+ 5,0<br>- 3,0 N</td><td colspan="3">+ 4,8<br>- 3,2</td></tr>
<tr><td rowspan="6">THICKNESS OF FLANGE (t)</td><td rowspan="6"></td><td></td><td colspan="2">ALL THICKNESS</td><td>≤5</td><td>>5 ≤10</td><td>>10</td><td>≤5</td><td>>5 ≤10</td><td>>10</td></tr>
<tr><td>b ≤ 50</td><td colspan="2">± 0,30</td><td>± 0,2</td><td>± 0,2</td><td>± 0,3</td><td>± 0,25</td><td>± 0,25</td><td>± 0,30</td></tr>
<tr><td>50 < b < 75</td><td colspan="2">± 0,40</td><td>± 0,3</td><td>± 0,4</td><td>± 0,4</td><td>± 0,30</td><td>± 0,35</td><td>± 0,35</td></tr>
<tr><td>75 ≤ b ≤ 100</td><td colspan="2">± 0,40</td><td></td><td></td><td></td><td></td><td></td><td></td></tr>
<tr><td>100 < b ≤ 150</td><td colspan="2">± 0,50</td><td></td><td></td><td></td><td></td><td></td><td></td></tr>
<tr><td>150 < b ≤ 200</td><td colspan="2">± 0,60</td><td></td><td></td><td></td><td></td><td></td><td></td></tr>
<tr><td rowspan="2">OUT OF SQUARENESS FLANGE (m)</td><td rowspan="2"></td><td>b ≤ 100</td><td>1,0</td><td rowspan="2">2,5% de b</td><td colspan="3">0,026 mm/mm</td><td colspan="3">3/128 in/in</td></tr>
<tr><td>b > 100</td><td>1,5</td><td colspan="3">2,6 % de b<br>(1,5º)</td><td colspan="3">2,3 % de b<br>(1,5º)</td></tr>
<tr><td rowspan="3">STRAIGHTNESS OF MEMBERS (fv)</td><td rowspan="3"></td><td>b < 75</td><td colspan="2">fv ≤ 0,40% L</td><td colspan="3">fv ≤ 0,40% L</td><td colspan="3">fv ≤ 0,42% L</td></tr>
<tr><td>75 ≤ b ≤ 150</td><td colspan="2">fv ≤ 0,40% L</td><td colspan="3">fv ≤ 0,20% L</td><td colspan="3">fv ≤ 0,21% L</td></tr>
<tr><td>150 < b ≤ 200</td><td colspan="2">fv ≤ 0,25% L</td><td colspan="3">fv ≤ 0,20% L</td><td colspan="3">fv ≤ 0,21% L</td></tr>
<tr><td>OUT OF SQUERENESS END (a)</td><td></td><td>ALL</td><td>≤ 1,5% de b</td><td>≤ 2,5% de b</td><td colspan="3">≤ 2,6% de b<br>ou 1,5º</td><td colspan="3">≤ 2,3% de b<br>ou 1,5º</td></tr>
</table>

**NBR – BRAZILIAN STANDARDS**

# 7 Piece marking

The management of materials and erection require easy identification of the components. This is generally done by applying a mark to the steel by cold punching prior to any surface treatment.

### 7.1 Minimum Mark Height

**ASCE 10-97 Standard**

12.7 mm

**EN 50341-1:2001 - European CENELEC Standard**

Not specified

**ECCS 39 Recommendations**

Not specified

**Brazilian Industry Practice**

13 mm

**Korean Industry Practice**

No recommendation

**Finnish Industry Practice**

10 to15 mm (company specification)

**Icelandic Industry Practice**

12 mm

**Italian Industry Practice**

No recommendation

### 7.2 Depth of Marking

**ASCE 10-97 Standard**

Not specified

**EN 50341-1:2001 - European CENELEC Standard**

Not specified

**ECCS 39 Recommendations**

Not specified

**Brazilian Industry Practice**

Depth of markings shall be around 0.8 mm, in such a way, that they can be clearly readable after the galvanizing process. The pieces can not be damaged in any way by the marking punching procedures.

**Korean Industry Practice**

No recommendation

**Finnish Industry Practice**

Marking has to be readable after hot-dip galvanizing.

**Icelandic Industry Practice**

No recommendation

**Italian Industry Practice**

No recommendation


**Figure 7.1 – Typical pieces marking**

# Part 2 – Design practices

# Part 2.1 – General design practices for steel structures

## 8 Design of bolted connections

### 8.1 Minimum bolt distances

See also Annex D.

#### 8.1.1 General

The minimum distances bolt to bolt and bolt to the edges of the member(s) have an effect on the capacity of the connection and on the ease of assembly.

**ASCE 10-97 Standard**

Distances vary according to the allowable shear and bearing stresses adopted and can be reduced when those stresses are reduced.

Minimum end and edge distances for redundant members may be reduced in accordance to the expressions given under 8.1.3.

**EN 50341-1:2001 - European CENELEC Standard (J.11)**

Distances vary according to the allowable shear and bearing stresses.
Distances are not specified for inclined directions.

**EN 50341-3: National Normative Aspects**

See next clauses

**ECCS 39 Recommendations (10)**

Distances vary according to the allowable shear stresses.

The distances are not specified for inclined directions and are shown the end, centres and edge distances only for profiles.

**Brazilian Industry Practice**

In items 6.3, 8.1.2, 8.1.3 and 8.1.4 minimum distances are given in accordance to the ASCE specifications for members carrying load, considering bolts as per A394 types “0”, “1”, “2” and “3” and bolts type M12, M14, M16, M20, M24, as per ISO 898-classe 5.8 and 8.8.

The maximum allowable bearing stress used is “$f_p = 1.083\ f_u$”, where $f_u$ is the ultimate tensile strength of the bolt or plate material.

**Korean Industry Practice**

No recommendation

**Finnish Industry Practice**

No additional recommendation

**Icelandic Industry Practice**

Distances can vary according to the member loading. In general the bolt distances are categorized into three groups determined by Figure 8.1 and Table 8.1

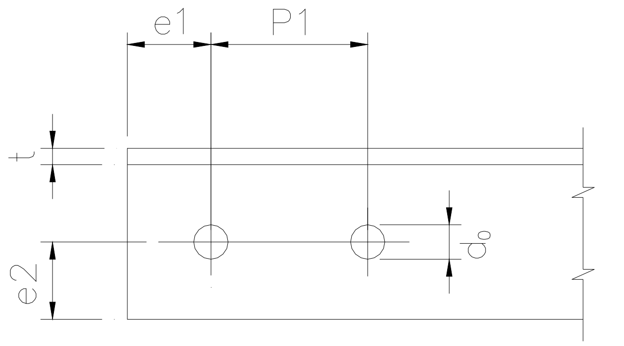



**Figure 8.1 – Minimum Icelandic design bolt distances**

**Table 8.1 – Icelandic design bolt distances**

| Category | $e_1$ | $e_2$ | $P_1$ |
|---|---|---|---|
| Minimum distance | 1,5 $d_0$ | 1,2 $d_0$ | 2,5 $d_0$ |
| General distance | 1,8 $d_0$ | 1,4 $d_0$ | 2,75 $d_0$ |
| Increased distance | 2,0 $d_0$ | 1,5 $d_0$ | 3,0 $d_0$ |

**Italian Industry Practice**

No additional recommendation

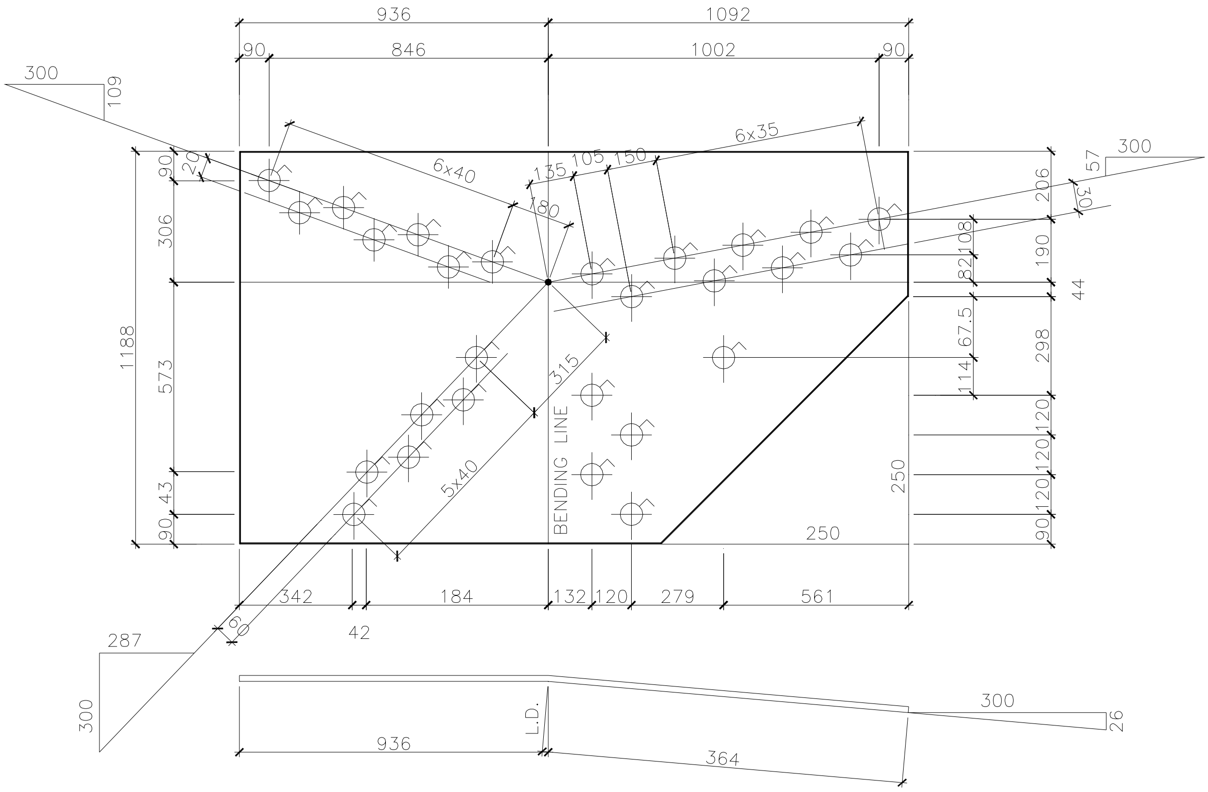



Figure 8.2 – Typical gusset plate detailing

Figure 8.3 – Leg members connections

### 8.1.2 Distance (s) between holes

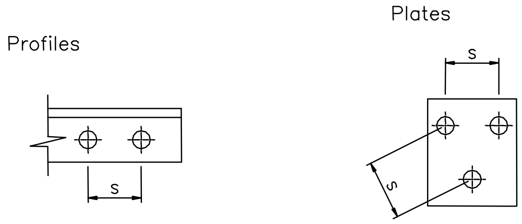



**Figure 8.4 – Distances between holes**

**ASCE 10-97 Standard**

$$s \geq (1.2\ P / f_u\ t) + 0.6\ d$$

s - distance between holes
P - force transmitted by the bolt
$f_u$ - ultimate tensile strength of plate or bolt
t - plate thickness
d - nominal diameter of the bolt

s ≥ nut diameter + 3/8”  (recommendation for assembly )

| Maximum nut diameter (mm) ANSI B18.2.2/81 | ANSI B18.2.4.1M/79 |
|---|---|
| 1/2” - 22.0 | M12 - 20.8 |
| 5/8” - 27.5 | M14 - 24.3 |
| 3/4” - 33.0 | M16 - 27.7 |
| 7/8” - 38.5 | M20 - 34.6 |
| 1” - 44.0 | M24 - 41.6 |

**EN 50341-1:2001 - European CENELEC Standard (J.11.2)**

$$s = [(P\ \gamma_{M2} / 0.96\ f_u\ d\ t) + 0.5]\ d_0$$

s - distance between holes
P - force transmitted by the bolt
$f_u$ - ultimate tensile strength of plate or bolts
t - plate thickness
$d_0$ - hole diameter
$\gamma_{M2}$ - partial factor for resistance of net cross section at bolt holes
d - nominal diameter of the bolt

$\gamma_{M2}$ = 1.25 (Clause 7.3.5.1.1 of EN 50341-1)
$\gamma_{M2}$ may be amended in the National Normative Aspects or the Project Specification (See Annex D).

## EN 50341-3: National Normative Aspects

AT → EN 50341-3-1 – Clause 7.3.1/AT.1 – The minimum distances between centres of boreholes shall be not less than 2.5 times the diameter of the holes.

BE → EN 50341-3-2 – Clause 7.3.5/BE.1 – $\gamma_{M2} = 1.25$

DE → EN 50341-3-4 – Clause 4.3.11/DE.3 – $\gamma_{M2} = 1.25$

Clause 7.3.6.2.1/DE.1 – The minimum distances between centres of boreholes shall be not less than 2.5 times the diameter of the holes.

DK → EN 50341-3-5 – Clause J.11/DK.1 – The minimum distances between centres of boreholes shall be not less than 2.5 times the diameter of the bolts.

FR → EN 50341-3-8 – Clause 7/FR.1 – The Interministerial Decree prescribes the mechanical checks to be made during overhead transmission line design. The national safety factors $g_R$ for material properties depend on load cases and materials. Other values of $g_R = 1.80$ may be required in the Project Specification.

IT → EN 50341-3-13 – Clause 7.3.6/IT.1 – No partial safety factor for resistance $\gamma_M$ is required. No ultimate limit analysis is required (See Part 2.2 – General).

NL → EN 50341-3-15 – Clause 7.3.5.1.1/NL.1 – $\gamma_{M2} = 1.25$

NO → EN 50341-3-16 – Clause 7.3.5.1.1/NO.1 – $\gamma_{M2} = 1.25$

SE → EN 50341-3-18 – Clause 7.3.5.1.1/SE.1 – $\gamma_{M2} = 1.32$

CZ → EN 50341-3-19 – Clause 7.3.5.3.1/CZ.1 – $\gamma_{M2} = 1.30$

SI → EN 50341-3-21 – Clause 4.3.11/SI.3 – $\gamma_{M2} = 1.25$

Clause 7.3.6.2.1/SI.1 – See Clause 7.3.6.2.1/DE.1 above.

## ECCS 39 Recommendations (10.4)

$$s = (P / f_y\, t) + 0.5\, d$$

s – distance between holes
P - force transmitted by the bolt
$f_y$ - yield strength of the bolt
t - plate thickness
d - nominal diameter of the bolt

## Brazilian Industry Practice

In accordance with ASCE.
Refer to Tables 8.2 and 8.3 for the distances between holes (including tolerances).

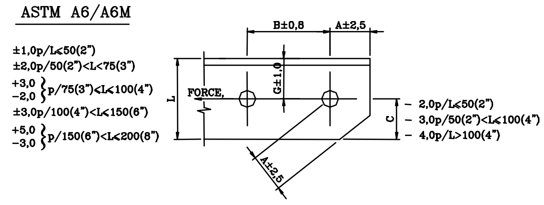
# Table 8.2 – Minimum hole and edge distances for bolts in inches

## Different allowable bearing stresses: steel mills and fabrication standard tolerances

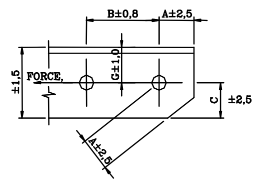
According to Brazilian practices

| Bolt Diameter | Thickness (mm) | A (mm) | | | B (mm) | | C Angles ≤ 50 mm ( 2” ) (mm) | | | C (mm) Angles > 50 mm ( 2” ) Plates Cold Formed Profiles | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | | fp = $1.08f_u$ | fp = $1.25f_u$ | fp = $1.50f_u$ | fp ≤ $1.25f_u$ | fp = $1.5f_u$ | fp = $1.08f_u$ | fp = $1.25f_u$ | fp = $1.50f_u$ | fp = $1.08f_u$ | fp = $1.25f_u$ | fp = $1.50f_u$ |
| | ≤ 10 | 19 | | | | | 16 | | | 18 | | |
| | 11.1 | 20 | | | | | 17 | | | 19 | | |
| 12.7 | 12 | 21 | 22 | 26 | 33 | 33 | 18 | 19 | 22 | 20 | 21 | 24 |
| ( 1/2” ) | 12.7 | 22 | | | | | 19 | | | 21 | | |
| | > 12.7 | 19 | | | | | 16 | | | 18 | | |
| | ≤ 13 | 24 | | | | | 20 | | | 22 | | |
| 15.9 | 14 | 25 | 27 | 32 | 38 | 39 | 21 | 23 | 27 | 23 | 25 | 29 |
| ( 5/8” ) | 15.9 | 27 | | | | | 23 | | | 25 | | |
| | > 15.9 | 24 | | | | | 20 | | | 22 | | |
| | ≤ 14 | 28 | | | | | 23 | | | 25 | | |
| | 15.9 | 28 | | | | | 24 | | | 26 | | |
| 19.1 | 18 | 30 | 31 | 37 | 44 | 47 | 26 | 27 | 32 | 28 | 29 | 34 |
| ( 3/4” ) | 19.1 | 31 | | | | | 27 | | | 29 | | |
| | > 19.1 | 28 | | | | | 23 | | | 25 | | |
| | ≤ 18 | 32 | | | | | 27 | | | 29 | | |
| | 19.1 | 33 | | | | | 28 | | | 30 | | |
| 22.2 | 20 | 34 | 36 | 43 | 49 | 55 | 29 | 31 | 36 | 31 | 33 | 38 |
| ( 7/8” ) | 22.2 | 36 | | | | | 31 | | | 33 | | |
| | > 22.2 | 32 | | | | | 27 | | | 29 | | |
| | ≤ 20 | 36 | | | | | 31 | | | 33 | | |
| | 22.2 | 38 | | | | | 32 | | | 34 | | |
| 25.4 | 24 | 40 | 41 | 49 | 55 | 62 | 34 | 35 | 41 | 36 | 37 | 43 |
| ( 1” ) | 25.4 | 41 | | | | | 35 | | | 37 | | |
| | > 25.4 | 36 | | | | | 31 | | | 33 | | |

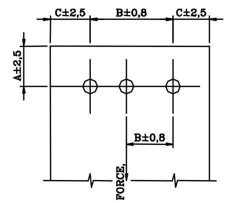
Hot–Rolled Angles




Cold Formed Angles




Plates

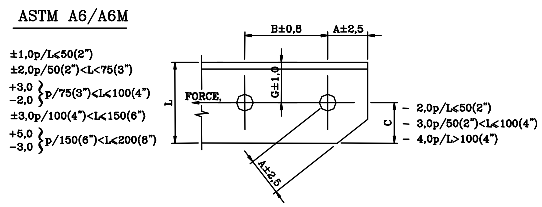


## Table 8.3 – Minimum hole and edge distances for metric bolts

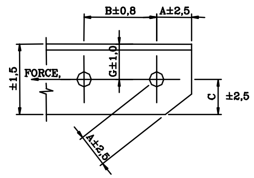
**Different allowable bearing stresses: steel mills and fabrication standard tolerances**
According to Brazilian practices

| Bolt Diameter | Thickness (mm) | A (mm) | | | B (mm) | | C Angles ≤ 50 mm ( 2” ) (mm) | | | C (mm) Angles > 50 mm ( 2” ) Plates Cold Formed Profiles | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | | fp = $1.08f_u$ | fp = $1.25f_u$ | fp = $1.50f_u$ | fp ≤ $1.25f_u$ | fp = $1.5f_u$ | fp = 1.08fu | fp = $1.25f_u$ | fp = $1.50f_u$ | fp = $1.08f_u$ | fp = $1.25f_u$ | fp = $1.50f_u$ |
| | ≤ 10 | 19 | | | | | 16 | | | 18 | | |
| M12 | 11 | 20 | | | | | 17 | | | 19 | | |
| | 12 | 21 | 21 | 25 | 32 | 32 | 18 | 18 | 21 | 20 | 20 | 23 |
| | > 12 | 19 | | | | | 16 | | | 18 | | |
| | ≤ 11 | 21 | | | | | 18 | | | 20 | | |
| | 12 | 22 | | | | | 19 | | | 21 | | |
| M14 | 13 | 23 | 24 | 28 | 35 | 35 | 19 | 20 | 24 | 21 | 22 | 26 |
| | 14 | 24 | | | | | 20 | | | 22 | | |
| | > 14 | 21 | | | | | 18 | | | 20 | | |
| | ≤ 13 | 24 | | | | | 20 | | | 22 | | |
| | 14 | 25 | | | | | 21 | | | 23 | | |
| M16 | 16 | 27 | 27 | 32 | 39 | 40 | 23 | 23 | 27 | 25 | 25 | 29 |
| | > 16 | 24 | | | | | 20 | | | 22 | | |
| | ≤ 16 | 29 | | | | | 25 | | | 27 | | |
| | 18 | 31 | | | | | 26 | | | 28 | | |
| M20 | 20 | 33 | 33 | 39 | 45 | 49 | 28 | 28 | 33 | 30 | 30 | 35 |
| | > 20 | 29 | | | | | 25 | | | 27 | | |
| | ≤ 18 | 34 | | | | | 29 | | | 31 | | |
| | 20 | 35 | | | | | 30 | | | 32 | | |
| M24 | 22 | 37 | 39 | 46 | 52 | 59 | 31 | 33 | 39 | 33 | 35 | 41 |
| | 24 | 39 | | | | | 33 | | | 35 | | |
| | > 24 | 34 | | | | | 29 | | | 31 | | |

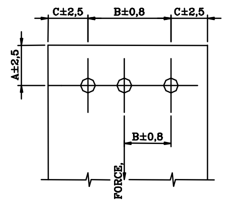
Hot-Rolled Angles




Cold Formed Angles




Plates



**Korean Industry Practice**

- Maximum: s ≤ 8 d
- Minimum: s ≥ 2.5 d
- Standard: s = 3d

**Table 8.4– Korean distances between holes**

| d | s (mm) Min. | Std. | Max. |
|---|---|---|---|
| M16 | 40 | 50 | 125 |
| M20 | 50 | 60 | 160 |
| M24 | 60 | 75 | 190 |

where: d = nominal diameter of the bolt

**Finnish Industry Practice**

No additional recommendation, but the practice is in leg connections s ≥ 3d and in other members s ≥ 2.5d.

**Icelandic Industry Practice**

No additional recommendation.

**Italian Industry Practice**

No additional recommendation.

### 8.1.3 Distance (e) between the hole and the end

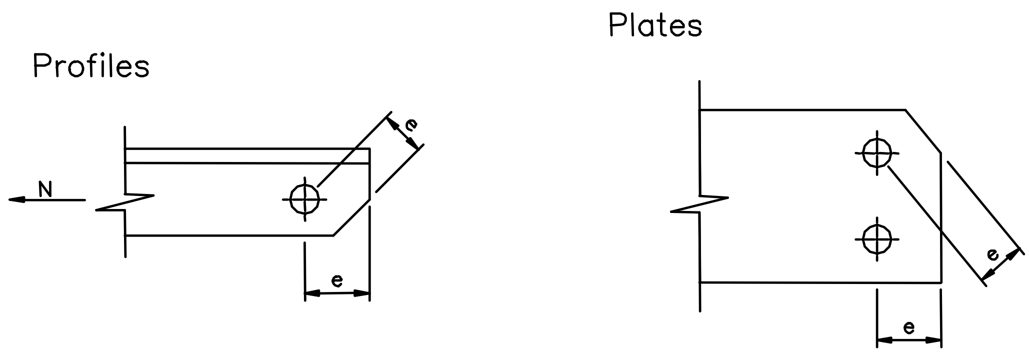



**Figure 8.5 – Hole distances to cut edge**

**ASCE 10-97 Standard**

Members carrying loads:

$$e \geq \frac{1.2\,P}{f_u \cdot t}$$

$e \geq 1.3\,d$

$e \geq t + d/2$ (for punched holes)

e – distance of the hole to the end or the cut edge of the profile

$f_u$ - ultimate tensile strength of the connected part
t - thickness of the most slender plate
d - nominal diameter of the bolt
P - force transmitted by the bolt

Redundant members:

$$e \geq 1.2\ d$$
$$e \geq t + d/2 \qquad \text{(for punched holes)}$$

Note: Maximum bearing stress $f_p$ to implicitly check out minimum distances:

$$P_{max} = f_p \cdot d \cdot t$$

$$e \geq \frac{1.2\ P_{max}}{f_u \cdot t} = \frac{1.2 \cdot f_p \cdot d \cdot t}{f_u \cdot t} = (1.2 \cdot f_p / f_u) \cdot d$$

and because $e \geq 1.3d$

$$1.2 \cdot f_p / f_u < 1.3 \text{ or}$$

$$f_p \leq 1.3/1.2\ f_u = 1.0833\ f_u$$

## EN 50341-1:2001 - European CENELEC Standard (J.11.2)

Members carrying loads

The biggest of:

$$e \geq \frac{P \cdot \gamma_{M2} \cdot d_0}{1.2 \cdot f_u \cdot d \cdot t}$$

$$e \geq \left( \frac{P \cdot \gamma_{m2}}{1.85 \cdot f_u \cdot d \cdot t} + 0.5 \right) \cdot d_0$$

e - distance of the hole to the end of the profile
$f_u$ - ultimate tensile strength of plate or bolt
t - plate thickness
$d_0$ - hole diameter
$\gamma_{M2}$ - partial factor for resistance of net cross section at bolt holes (see Item 8.1.2)
d - nominal diameter of the bolt
P - force transmitted by the bolt

## EN 50341-3: National Normative Aspects

AT → EN 50341-3-1 – Clause 7.3.1/AT.1 – The minimum distances between the centre of borehole and the end of profile shall be not less than 1.50 times the diameter of the boreholes.

DE → EN 50341-3-4 – Clause 7.3.6.2.1/DE.1 – The minimum values of the distances in direction of the force specified in Table 8.6 and which are measured from the centre of the hole shall be adhered to in any case. For tensile loaded components of the vertical truss faces of cross arms as well as for leg member joints the higher values shall be adhered to.

**Table 8.5 – Edge distances of bolts (DE & SI)**

| Dimension of bolt (mm) | M12 | M16 | M20 | M24 | M27 | M30 |
|---|---|---|---|---|---|---|
| Minimum distances in the force direction (mm) | 20 | 25 | 30 | 40 | 45 | 50 |

DK → EN 50341-3-5 – Clause J.11/DK.1 – The minimum distances between centres of boreholes and the ends shall be not less than 1.75 times the diameter of the bolts.

SI → EN 50341-3-21 – Clause 7.3.6.2.1/SI.1 – See Clause 7.3.6.2.1/DE.1 above.

**ECCS 39 Recommendations (10.3)**

Members carrying loads:

$$e \geq \frac{P}{2.0 \cdot f_y \cdot t} + 0.5\,d \qquad \text{for} \qquad e \leq 1.5\,d$$

$$e \geq \frac{P}{1.33 \cdot f_y \cdot t} \qquad \text{for} \qquad e \geq 1.5\,d$$

e - distance between the hole and the end or the cut edge of the profile
$f_y$ - yield strength of the bolt
t - plate thickness
d - nominal diameter of the bolt
P - force transmitted by the bolt
Not specified for redundant members

**Brazilian Industry Practice**

In accordance with ASCE 10-97.
See Tables 8.2 and 8.3 for the referred distances (including tolerances).

**Korean Industry Practice**

Leg members: $e \geq 2.0\,d$, $f \geq 1.5\,d$
Others: $e \geq 1.5\,d$, $f \geq 1.3\,d$
where: d - nominal diameter of the bolt
f - distance between the hole and the edge (See 8.1.4)

**Table 8.6 – Korean practice on bolted connection distances**

| | Leg members mm | | Other members mm | |
|---|---|---|---|---|
| d | e | f | e | f |
| M16 | 35 | 24 | 25 | 21 |
| M20 | 40 | 30 | 30 | 26 |
| M24 | 50 | 36 | 40 | 32 |

**Finnish Industry Practice**

No additional recommendation, but the practice is $e \geq 1.5d$.

**Icelandic Industry Practice**

Distance to clipped edge shall not be within an ellipse centred in the bolt hole and defined by major radius $e_1$ and minor radius $e_2$. Refer to Figure 8.6.

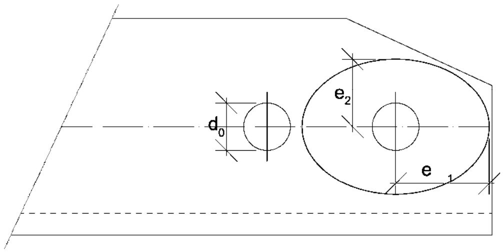



**Figure 8.6 – Iceland bolts distance to edge**

**Italian Industry Practice**

No additional recommendation.

### 8.1.4 Distance (f) between the hole and the edge

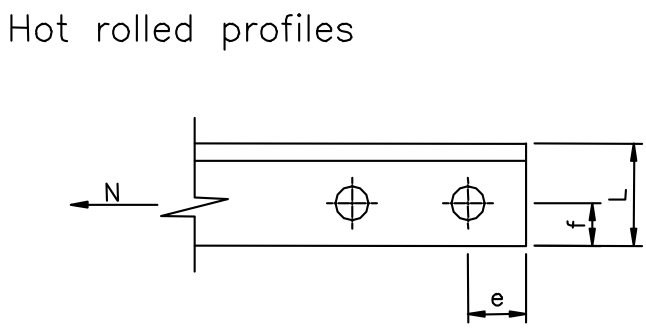



**Figure 8.7 – Distance between hole and edge**

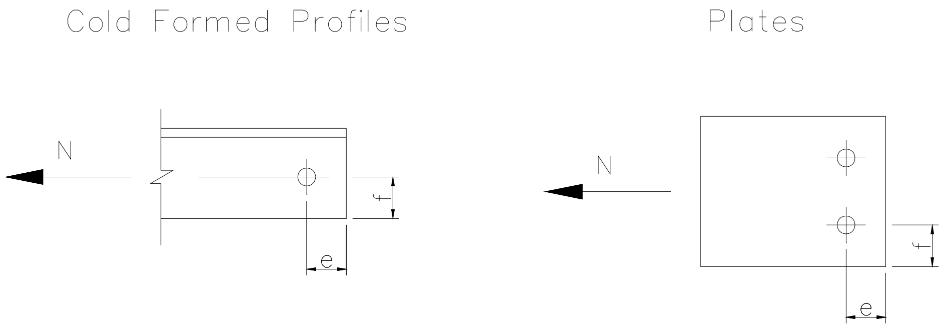



**Figure 8.8 – Hole distance to clipped edge**

**ASCE 10-97 Standard**

$$f > 0.85\ e$$

e - distance between the hole and the end
f - distance between the hole and the edge

**EN 50341-1:2001 - European CENELEC Standard (J.11.2)**

$$f = \left(\frac{P \cdot \gamma_{M2}}{2.3 \cdot f_u \cdot d \cdot t} + 0.5\right) \cdot d_0$$

f - distance between hole and edge
$f_u$ - ultimate tensile strength of plate or bolt
t - plate thickness
$d_0$ - hole diameter
$\gamma_{M2}$ - partial factor for resistance of net cross section at bolt holes (see Item 8.1.2)
d - nominal diameter of the bolt
P - force transmitted by the bolt

**EN 50341-3: National Normative Aspects**

AT → EN 50341-3-1 – Clause 7.3.1/AT.1 – The edge distances rectangular to the direction of the force shall be not less than 1.25 times of the diameter of the borehole.

DE →EN 50341-3-4 – Clause 7.3.6.2.1/DE.1 – The edge distances rectangular to the direction of the force shall be not less than 1.2 times of the diameter of the borehole.

DK → EN 50341-3-5 – Clause J.11/DK.1 – The minimum distances between centres of boreholes and the edge shall be not less than 1.3 times the diameter of the bolts.

SI → EN 50341-3-21 – Clause 7.3.6.2.1/SI.1 – See Clause 7.3.6.2.1/DE.1 above.

## ECCS 39 Recommendations (10.5)

$$f = \frac{P}{2.67 \cdot f_y \cdot t} + 0.5\,d$$

f - distance between the hole and the edge
$f_y$ - yield strength of the bolt
t - plate thickness
d - nominal diameter of the bolt
P - force transmitted by the bolt

## Brazilian Industry Practice

In accordance with ASCE 10-97.
See Tables 8.2 and 8.3 for the referred distances (including tolerances).

## Korean Industry Practice

Leg members: $e \geq 2.0$ d, $f \geq 1.5$ d
Others: $e \geq 1.5$ d, $f \geq 1.3$ d
where: d - nominal diameter of the bolt

**Table 8.7 – Korean practice on edge distances of bolted connections**

| | Leg members mm | | Other members mm | |
|---|---|---|---|---|
| **d** | **e** | **f** | **e** | **f** |
| M16 | 35 | 24 | 25 | 21 |
| M20 | 40 | 30 | 30 | 26 |
| M24 | 50 | 36 | 40 | 32 |

## Finnish Industry Practice

No additional recommendation, but $f \geq 1.5d$ is the practice.

## Icelandic Industry Practice

No additional recommendation.

## Italian Industry Practice

No additional recommendation.

**Figure 8.9 – Holes / edges distances**

## 8.2 Structural Design

### 8.2.1 Shear

**ASCE 10-97 Standard**

According to ASTM-A394 (1987):

**Type "0"**
Low or medium carbon steel, zinc-coated (hot dip):

$f_v$ = 380.5 N/mm$^2$ (for the thread)
$f_v$ = 316.5 N/mm$^2$ (for the body)

**Type "1", "2" and "3"**
$f_v$ = 513 N/mm$^2$ (thread and body)

$f_v$ - allowable shear stress

Bolts that have no specified shear strength:

$f_v = 0.62\ f_u$ (thread or body)

$f_u$ - ultimate tensile strength of the bolt

The minimum shear force shall be evaluated multiplying the effective area (root cross-section area at the thread or gross cross-section area at the body) by the corresponding allowable shear stress.

The cross-section area at the root thread is based on the core diameter (ANSI)

**Table 8.8 – ASCE 10-97 – Bolt's areas**

| d | Body area ($cm^2$) | Root thread area ($cm^2$) |
|---|---|---|
| 1/2" | 1.297 | 0.811 |
| 5/8" | 1.979 | 1.303 |
| 3/4" | 2.850 | 1.948 |
| 7/8" | 3.879 | 2.708 |
| 1" | 5.067 | 3.558 |
| M12 | 1.131 | 0.743 |
| M14 | 1.539 | 1.021 |
| M16 | 2.011 | 1.411 |
| M20 | 3.142 | 2.204 |
| M24 | 4.524 | 3.174 |

**<u>EN 50341-1:2001 - European CENELEC Standard (J.11.1)</u>**

If the shear plane passes through the unthreaded portion of the bolt:

$$F_{v,Rd} = 0.6\ f_u\ A/\gamma_{Mb}$$

If the shear plane passes through the threaded portion of the bolt for classes 4.6, 5.6, 6.6, 8.8:

$$F_{v,Rd} = 0.6\ f_u\ A_s/\gamma_{Mb}$$

If the shear plane passes through the threaded portion of the bolt for classes 4.8, 5.8, 6.8, 10.9:

$$F_{v,Rd} = 0.5\ f_u\ A_s/\gamma_{Mb}$$

A - cross section area of the bolt
$A_s$ - tensile stress area of the bolt
$F_{v,Rd}$ - shear resistance per shear plane
$\gamma_{Mb}$ - partial factor for resistance of bolted connections
$f_u$ - ultimate tensile strength of the bolt

$\gamma_{Mb}$ = 1.25 (Clause 7.3.6.1.1 of EN 50341-1)
$\gamma_{Mb}$ may be amended in the National Normative Aspects or the Project Specification (See Annex D).

## EN 50341-3: National Normative Aspects

AT → EN 50341-3-1 – Clause 7.3.1/AT.1 – $\gamma_{Mb} = 1.25$

BE → EN 50341-3-2 – Clause 7.3.6/BE.1 – $\gamma_{Mb} = 1.25$

DE → EN 50341-3-4 – Clause 4.3.11/DE.3 – $\gamma_{Mb} = 1.25$

GB → EN 50341-3-9 – Clause 7.1/GB.1 – Steelwork bolted connections shall be designed to the tensile strength, $R_m$ detailed in ISO 898-1.

IT → EN 50341-3-13 – Clause 7.3.6/IT.1 – The allowable shear stress, not to be exceeded under the given normal loading conditions, without partial safety factor, shall be 30 % of ultimate tensile strength.

NL → EN 50341-3-15 – Clause 7.3.6.1.1/NL.1 – $\gamma_{Mb} = 1.25$

Clause 7.3.6.2.2/NL.1 - The design resistance of bolts in shear is given in the Eurocode.

NO → EN 50341-3-16 – Clause 7.3.6.1.1/NO.1 – $\gamma_{Mb} = 1.25$

SE → EN 50341-3-18 – Clause 7.3.6.1.1/SE.1 – $\gamma_{Mb} = 1.32$

CZ → EN 50341-3-19 – Clause 7.3.5.3.1/CZ.1 – $\gamma_{Mb} = 1.45$

SI → EN 50341-3-21 – Clause 4.3.11/SI.3 – $\gamma_{Mb} = 1.25$

## ECCS 39 Recommendations (10.1)

The lesser than:

$$P \leq 0.69\, f_u \cdot A$$
$$P \leq 0.95\, f_y \cdot A$$

$f_u$ - ultimate tensile stress of the bolt
$f_y$ - yield strength of the bolt
A - cross section area of the bolt shank in shear plane
P - maximum shear force transmitted by the bolt

Not specified for the cross-section area at the thread

## Brazilian Industry Practice

In accordance with ASCE 10-97

## Korean Industry Practice

Definitions

- $d_b$ - diameter for bearing
- $d_g$ - diameter of thread
- p - pitch
- $A_s$ - net area of the bolt in shear or tension
- $f_{as}$ - allowable shear stress
- $f_{ab}$ - allowable bearing stress
- $f_{at}$ - allowable tensile stress

- $f_y$ - yield stress of the bolt
- $f_u$ - ultimate tensile strength

* $d_b = (d_e + d_g - 0.866025\ p/6) \cdot 0.5$
* $A_s = \pi/4 \cdot \{(d_e + d_g - 0.866025\ p/6) \cdot 0.5\}^2$
* $f_{as} = \min(f_y,\ 0.7\ f_u) / 1.5 / 1.732051$
* $f_{ab} = f_y \cdot 1.1$
* $f_{at} = \min(f_y,\ 0.7\ f_u) / 1.5$

$A_s \cdot f_{as} \cdot Q_s$ / one bolt

where $Q_s = 1$ for Lap-joint
$Q_s = 2$ for Butt-joint

Allowable shear stress $f_{as}$

| d | $A_s$ cm$^2$ | $f_u$ N/mm$^2$ | $0.7f_u$ N/mm$^2$ | $f_y$ N/mm$^2$ | $f_{as}$ N/mm$^2$ |
|---|---|---|---|---|---|
| M16 | 1.57 | 520 | 364 | 420 | 137 |
| M20 | 2.45 | 520 | 364 | 420 | 137 |
| M24 | 3.53 | 830 | 581 | 660 | 221 |

* M16 & M20 : Gr. 5.8, M24 : Gr. 8.8

**Finnish Industry Practice**

No additional recommendation

**Icelandic Industry Practice**

No additional recommendation

**Italian Industry Practice**

No additional recommendation

**Figure 8.10 - Typical galvanized towers bolts**

### 8.2.2 Tension

**ASCE 10-97 Standard**

According to ASTM-A394 (1987):

**Type "0"**

Low or medium carbon steel, zinc-coated (hot dip):

$f_v$ = 316.5 N/mm$^2$ (for the body)

**Type "1" , "2" and "3"**

$f_v$ = 513 N/mm$^2$ (thread and body)

$f_v$ - allowable shear stress

Bolts that have no specified proof-load stress

$f_t = 0.6\ f_u$ over the net area ($A_s$) of the bolt

$f_u$ - ultimate tensile strength of the bolt

Net stress areas for bolts in tension:

Bolts in inches: $A_s = (\pi/4)\ [d - (0.974/n_t)]^2$

Metric bolts : $A_s = (\pi/4)\ [d - 0.9382\ p\ ]^2$

where: d - nominal diameter of the bolt
$n_t$ - number of threads per inch
p – pitch of thread

**Table 8.9 – Net areas for tension bolts**

| **d** | **$n_t$** | **$A_s$ ($cm^2$)** | **d** | **p (mm)** | **$A_s$ ($cm^2$)** |
|---|---|---|---|---|---|
| 1/2” | 13 | 0.915 | M12 | 1.75 | 0.843 |
| 5/8” | 11 | 1.450 | M14 | 2.00 | 1.154 |
| 3/4” | 10 | 2.155 | M16 | 2.00 | 1.567 |
| 7/8” | 9 | 2.979 | M20 | 2.50 | 2.448 |
| 1” | 8 | 3.908 | M24 | 3.00 | 3.525 |

**EN 50341-1:2001 - European CENELEC Standard (J.11.3)**

$$F_{t,Rd} = 0.9 \cdot f_u \cdot A_s / \gamma_{Mb}$$

$A_s$ - net tensile stress area of the bolt
$F_{t,Rd}$ – tension resistance per bolt
$\gamma_{Mb}$ - partial factor for resistance of bolted connections (see Item 8.2.1)
$f_u$ - ultimate tensile strength of the bolt

**EN 50341-3: National Normative Aspects**

IT → EN 50341-3-13 – Clause 7.3.6/IT.1 – The allowable tension stress, not to be exceeded under the given normal loading conditions, without partial safety factor, shall be of the ultimate tensile strength.

NL → EN 50341-3-15 – Clause 7.3.6.2.2/NL.1 – The design resistance of bolts in tension is given in the Eurocode.

**ECCS 39 Recommendations**
Not specified

**Brazilian Industry Practice**
In accordance with ASCE 10-97

**Korean Industry Practice**
For all the definitions see item 8.2.1.

$A_s \cdot f_{at}$ / one bolt

**Table 8.10 – Korean practice - Bolt tensions**

$f_{at} = \min(f_y, 0.7\ f_u) / 1.5$
(allowable tensile stress ; $f_{at}$)

| d | $A_s$ cm$^2$ | $f_u$ N/mm$^2$ | $0.7\ f_u$ N/mm$^2$ | $f_y$ N/mm$^2$ | $f_{at}$ N/mm$^2$ |
|---|---|---|---|---|---|
| M16 | 1.57 | 520 | 364 | 420 | 240 |
| M20 | 2.45 | 520 | 364 | 420 | 240 |
| M24 | 3.53 | 830 | 581 | 660 | 382 |

* M16 & M20 : Gr. 5.8,      M24 : Gr. 8.8

**Finnish Industry Practice**

No additional recommendation.

**Icelandic Industry Practice**

No additional recommendation

**Italian Industry Practice**

No additional recommendation

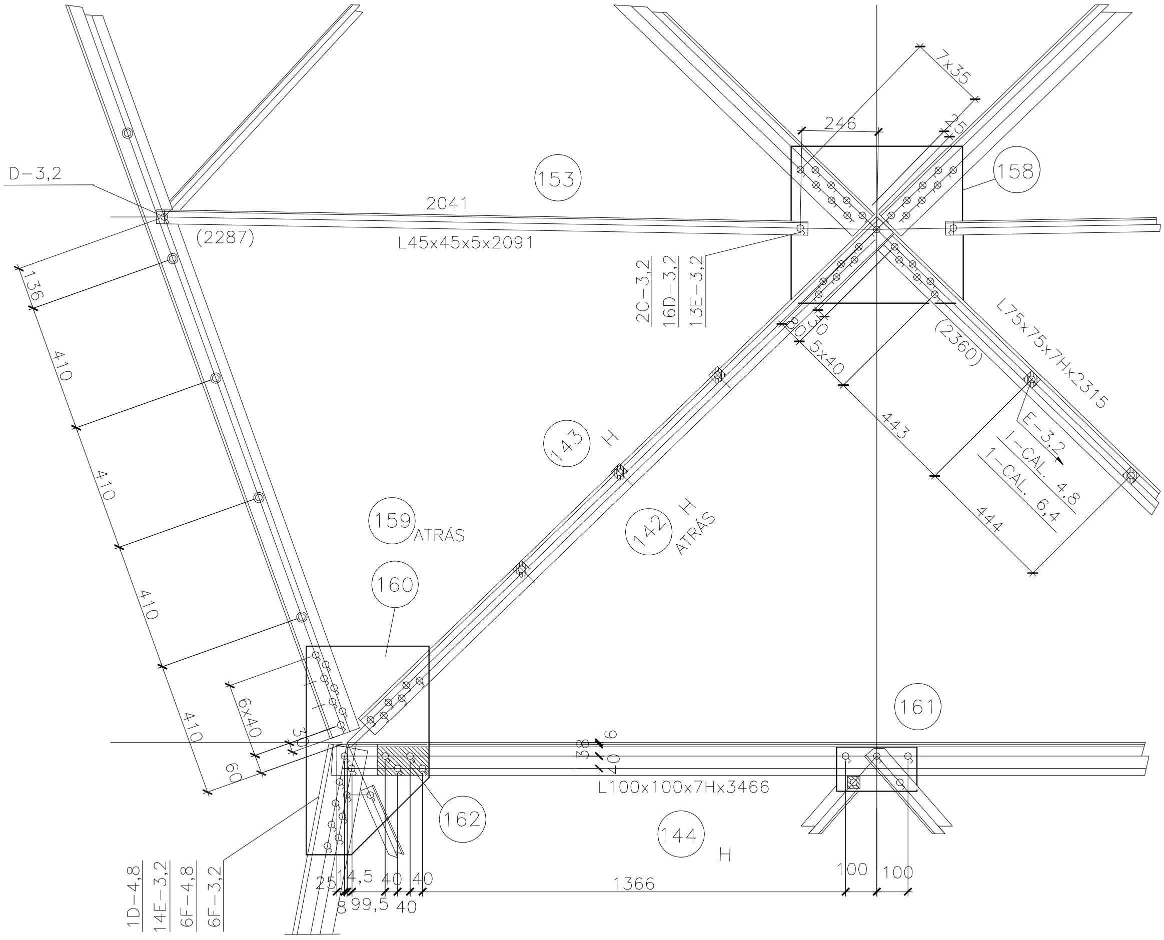



**Figure 8.11 – Typical lattice tower detailing**

### 8.2.3 Combined Shear and Tension

**ASCE 10-97 Standard**

$$f_t(v) = f_t \left[1 - (f_{cv}/f_v)^2\right]^{1/2}$$

where:

$f_t$ - design tensile stress under tension only (item 8.2.2)
$f_v$ - design shear stress under shear only (item 8.2.1)
$f_{cv}$ - computed shear stress on effective area (thread or body)
$f_t(v)$ - design tensile stress when bolts are subject to combined shear and tension. The combined tensile and shear stresses shall be taken at the same cross section in the bolt, either in the threaded or the unthreaded portion.

**EN 50341-1:2001 - European CENELEC Standard**

Not specified

**ECCS 39 Recommendations**

Not specified

**Brazilian Industry Practice**

In accordance with ASCE 10-97, taking into account maximum assembly torques (tightening of the bolts).

Allowable shear stresses for maximum assembly torques are in accordance with Table 8.15.

**Table 8.11 – Shear and Tension Bolts - Brazilian Practice**

| Bolt | Section | $f_t$ (for max torque) | $f_{vt}$ [N/mm$^2$] |
|---|---|---|---|
| NBR-8855 Classe 5.8 and ASTM A394 Type “0” | Body<br>Thread | $0.34\ f_u$<br>$0.45\ f_u$ | 260.7<br>252.8 |
| NBR-8855 Classe 8.8 and ASTM A394 Type “1” | Body<br>Thread | $0.21\ f_u$<br>$0.28\ f_u$ | 480.6<br>455.3 |

$f_{vt}$ - Allowable shear stresses for maximum assembly torques (N/mm$^2$)
$f_t$ - Tension stress for maximum assembly torques (N/mm$^2$)

**Korean Industry Practice**

No recommendation

**Finnish Industry Practice**

$$\left(\frac{\sigma}{f_t}\right)^2 + \left(\frac{\tau}{f_v}\right)^2 \leq 1,$$

Where: $\sigma$ is tensile stress and $\tau$ is shear stress.

**Icelandic Industry Practice**

No recommendation

**Italian Industry Practice**

No recommendation

### 8.2.4 Bearing

**ASCE 10-97 Standard**

$$f_p \leq 1.5\, f_u$$

$f_u$ - ultimate tensile strength of plate or bolt
$f_p$ - ultimate bearing stress

**EN 50341-1:2001 - European CENELEC Standard (J.11.2)**

$$F_{b,Rd} = \alpha\, f_u\, d\, t / \gamma_{M2}$$

where: $\alpha$ is the smallest of:

$1.20\,(e_1/d_o)$ $\quad$ $1.85\,(e_1/d_o - 0.5)$ $\quad$ $0.96\,(P_1/d_o - 0.5)$ $\quad$ $2.3\,(e_2/d_o - 0.5)$

$e_1$ - end distance from centre of hole to adjacent end in angle
$e_2$ - edge distance from centre of hole to adjacent edge in angle
$d_0$ - hole diameter
$P_1$ - spacing of two holes in the direction of load transfer
$f_u$ - ultimate tensile strength of plate or bolt
$F_{b,Rd}$ - bearing resistance per bolt
$\gamma_{M2}$ - partial factor for resistance for net cross section at bolt holes (See Item 8.1.2)

**EN 50341-3: National Normative Aspects**

DE → EN 50341-3-4 – Clause J.11/DE.1 – The bearing capacity $F_{b,Rd}$ calculated in accordance with EN 50341-1 shall be reduced by the factor 0.80.

DK → EN 50341-3-5 – Clause J.11/DK.1 – The corresponding bearing resistance factor $\alpha$, for M16 and M20, are indicated in the Table 8.16 below.

**Table 8.12 – α-factors for bearing stresses**

| **Bolt hole, do** | M16 | M20 |
|---|---|---|
| d + 1.5 mm | 1.58 | 1.63 |
| d + 2.0 mm | 1.51 | 1.57 |

IT → EN 50341-3-13 – Clause 7.3.6/IT.1 – The allowable bearing stress, not to be exceeded under the given normal loading conditions, without partial safety factor, shall be 240 % of allowable max compression stress of the material, for $\lambda \leq 15$.

NL → EN 50341-3-15 – Clause 7.3.6.2.2/NL.1 - The design resistance of bolts in bearing is given in the Eurocode.

**ECCS 39 Recommendations (10.2)**

$$f_p \leq 2.0\ f_y$$

$f_p$ – ultimate bearing stress
$f_y$ - yield strength

The ultimate bearing stress is normally related to the acceptable amount of deformation accepted in the hole, and while values up to three times yield strength may be used without failure, it is usual to restrict the stress to twice yield strength value to reduce the deformation into acceptable limits.

**Brazilian Industry Practice**

$$1.0833\ f_u \leq f_p \leq 1.5\ f_u$$

Distances hole to hole and hole to edge must be compatible with the adopted bearing stress.

Note - For those distances see Tables 8.2 and 8.3 where minimum distances are given for $f_p = 1.0833\ f_u$ as well as distances for allowable bearing stresses $f_p = 1.25\ f_u$ and $f_p = 1.50\ f_u$.

**Korean Industry Practice**

For all the definitions see item 8.2.1

$d \cdot t \cdot f_{ab}$ / one bolt

where: t - thickness of member
$f_{ab1}, f_{ab2} = f_y \cdot 1.1$
$f_{ab} = \min(f_{ab1}, f_{ab2})$

(allowable bearing stress ; $f_{ab}$ )

| Bolt | $f_y$ N/mm$^2$ | $f_{ab1}$ N/mm$^2$ | Member | $f_y$ N/mm$^2$ | $f_{ab2}$ N/mm$^2$ |
|---|---|---|---|---|---|
| M16 | 420 | 460 | SS 400 | 245 | 270 |
| M20 | 420 | 460 | SWS490 | 315 | 345 |
| M24 | 660 | 725 | SS 540 | 390 | 430 |

* M16 & M20 : Gr. 5.8,  M24 : Gr. 8.8

**Finnish Industry Practice**

No additional recommendation

**Icelandic Industry Practice**

EN 50341-1 is adopted, but additional requirements are made to limit deformation of bolt holes.

**Table 8.13 –Holes deformation - Icelandic practice**

| Category | $e_1$ | $e_2$ | p | $F_{b,Rd}$ |
|---|---|---|---|---|
| Minimum distance | 1.5 $d_0$ | 1.2 $d_0$ | 2.5 $d_0$ | 1.5 $f_y/\gamma_{M1}$ |
| General distance | 1.8 $d_0$ | 1.4 $d_0$ | 2.75 $d_0$ | 1.8 $f_y/\gamma_{M1}$ |
| Increased distance | 2.0 $d_0$ | 1.5 $d_0$ | 3.0 $d_0$ | 2.0 $f_y/\gamma_{M1}$ |

where: $\gamma_{M1}$ - partial factor for resistance for net section at bolt holes (See Item 8.1.2)

**Italian Industry Practice**

No additional recommendation

# 9 Design of tension members

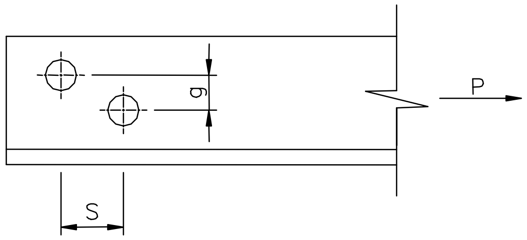



**Figure 9.1 – Tension members design**

s - distance between holes in the direction parallel to the direction of the force
g - distance between holes in the direction perpendicular to the direction of the force

**ASCE 10-97 Standard**

– Concentric loads

$$f_t = P / A_n \leq f_y$$

$f_t$ - tensile stress on net area
$f_y$ - yield strength of the material
P - force transmitted by the bolt

$$A_n = A_g - n\, d_0\, t + [\sum(s^2 / 4\, g)]\, t$$

s - distance between holes in the direction parallel to the axial force
g - distance between holes in the direction perpendicular to the axial force
$A_n$ - net cross section area
$A_g$ - gross cross section area
n - number of holes
t - plate thickness
$d_0$ - hole diameter

– Eccentric loads: angle members connected by one leg

$$f_t = P / A_e \leq f_y$$

$$A_e = 0.9\, A_n$$

$A_e$ - effective cross section area

For unequal leg angles, connected by the shorter leg the free leg shall be considered as having the same width as the shorter leg

– Eccentric loads: other sections
Members shall be proportioned for axial tension and bending

– Threaded rods :

$$f_t = P / A_s \le f_y$$

$A_s$ - core section at the thread, as defined under item 8.2.2.

**EN 50341-1:2001 - European CENELEC Standard (J.4.1)**

– Concentric loads: angles connected by two legs

$$f_t = P \cdot \gamma_{m2} / 0.9 \cdot A_n \le f_u$$

$f_t$ - tension stress on net area
$f_u$ - ultimate tensile strength
P - axial force

$$A_n = A_g - [\, n\, d_0 + \sum(s^2 / 4\, g)]\, t$$

s - distance between holes in the direction parallel to the direction of the force
g - distance between holes in the direction perpendicular to the direction of the force
$A_g$ - gross cross section area
$A_n$ - net cross section area
t - plate thickness
n - number of holes
$d_0$ - hole diameter

$\gamma_{M2}$ - partial factor for resistance of net section at bolt holes (see 8.1.2)

– Eccentric loads: angle members connected by one leg
  – with one bolt:

$$f_t = P \cdot \gamma_{M2} / (b_1 - d_0)\, t \le f_u$$

Requirements (Clause J.11.2) for bolted connection also limit the capacity of angle with single bolt to

$$f_t = P \cdot \gamma_{M2} / (2.3\, (e_2 / d_0 – 0.5))\, t \le f_u$$

  – with two or more bolts:

$$f_t = P \cdot \gamma_{M2} / (b_1 - d_0 + b_2/2)\, t \le f_u$$

$b_1$ - width of connected leg
$b_2$ - width of free leg
t - plate thickness
$d_0$ - hole diameter
$\gamma_{M2}$ - partial factor for resistance for net section at bolt holes (See 8.1.2)

**EN 50341-3: National Normative Aspects**

DE → EN 50341-3-4 – Clause J.3.2(2)/DE.1 – The net cross section of an angle section or a cross-sectional part under tensile load is the smallest value which results from

checking of potential lines of breaking and is calculated from $A_n = A_g - \Delta A$, where $\Delta A$ is the sum of all hole areas along the breaking line checked.
Clause J.4.1/DE.1 – The design tension resistance of angle sections calculated according to EN 50341-1 shall be reduced by 10%.

IT → EN 50341-3-13 – Clause 7.3.5/IT.1 – The stress in tension members shall be checked on net section, with the allowable stress shown for $\lambda \leq 15$ according to Tables I and II of EN 50341-3-13, Clause J.5/IT.1. See Part 2.2 - General.

NL →EN 50341-3-15 – Clause 7.3.5.3/NL.2 – The resistance of cross sections of members for axial tension shall be determined in accordance with the requirements of EN 1993-1-8, 6.2.3. For angles connected through one leg, see EN 1993-1-1, 3.6.3.

**ECCS 39 Recommendations (9)**

– Concentric loads: angle members connected by two legs

$$f_t = P \cdot / A_n \leq f_y$$

$f_y$ - yield strength of material

$$A_n = A_g - n\, d_0\, t$$

$A_g$ - gross area
$A_n$ - net area

– Eccentric loads : angle members connected by one leg

$$f_t = P / A_e \leq f_y$$

$$A_e = (A_g/2) - n\, d_0\, t + (A_g/4)$$

$A_e$ - effective area

**Brazilian Industry Practice**

In accordance with ASCE 10-97 taking into account the following tolerances for hole diameters:

Imperial bolts: $d_0 = d + 1/8''$
Metric bolts: $d_0 = d + 3$ mm

$d_0$ - hole diameter
d - nominal diameter of the bolt

**Korean Industry Practice**

– Concentric loads

$$f_t = P / A_n \leq f_{at} = f_y / 1.5$$

$f_{at}$ - allowable tensile stress of material
$f_y$ - yield stress of material

$$A_n = A_g - [\, n \cdot d_0 + \sum(s^2 / 4\, g)] \cdot t = A_g - (R+1)\, d_0 \cdot t$$

s - distance between holes in the direction parallel to the direction of the force
g - distance between holes in the direction perpendicular to the direction of the force
n - number of holes
t - plate thickness
$d_0$ - hole diameter (nominal diameter + 1.5 or 2.0mm)
R - quantity of row-line

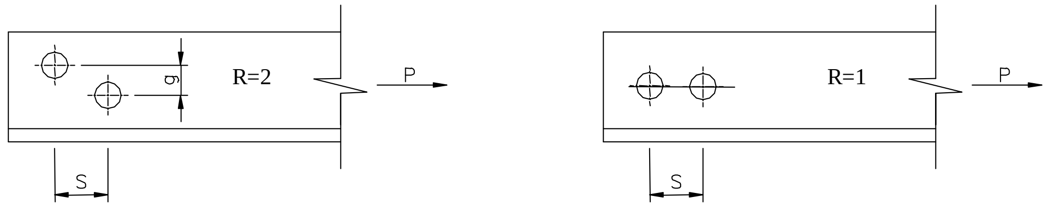



**Figure 9.2 – Distances between holes – Korean practices**

– <u>Eccentric loads: angle members connected by one leg</u>

$$f_t = P / A_e \leq f_{at} = f_y / 1.5$$

$f_{at}$ - allowable tensile stress of material
$f_y$ - yield strength of material

$$A_e = 0.75 \cdot A_g - n \cdot d_0 \cdot t = 0.75 \cdot A_g - R \cdot d_0 \cdot t$$

$A_e$ - effective area
n - number of holes
t - plate thickness
$d_0$ - hole diameter (nominal diameter + 1.5 or 2.0 mm)
R - quantity of row-line

– <u>Threaded rods</u>
No recommendation

**<u>Finnish Industry Practice</u>**
No additional recommendation.

**<u>Icelandic Industry Practice</u>**
No additional recommendation

**<u>Italian Industry Practice</u>**
No additional recommendation

# 10 Design of compression members

## 10.1 Allowable compression stress

**<u>ASCE 10-97 Standard</u>**

Allowable compression stress on the gross cross-sectional area ($N/mm^2$):

$$f_a = [\,1 - 0.5\,((kL/i) / C_c)^2\,]\, f_y \qquad \text{if } (kL/i) \le C_c$$

$$f_a = \pi^2 E / (kL/i)^2 \qquad \text{if } (kL/i) > C_c$$

$$\text{with } C_c = [(2\pi^2 E) / f_y]^{1/2}$$

where:

- $f_y$ - yield strength ($N/mm^2$) (See also Clause 10.2)
- $E$ - modulus of elasticity ($N/mm^2$)
- $L$ - unbraced length
- $i$ - radius of gyration corresponding to the buckling axis
- $k$ - effective buckling length coefficient (See Clauses 11.1 and 11.2)

**<u>EN 50341-1:2001 - European CENELEC Standard (J.5.1.1)</u>**

The design buckling resistance of the compression member $N_{R,d}$ is defined by:

$$N_{R,d} \le \chi \cdot A_{eff} \cdot f_y / \gamma_{M1}$$

- $A_{eff}$ - effective cross section area when subjected to uniform compression (See also 10.2)
- $f_y$ - yield strength
- $\chi$ - reduction factor depending on the slenderness ratio, the material properties and the buckling curve (or the imperfection factor) (see EN 1993-1-1 below)
- $\gamma_{M1}$ - partial factor for member resistance to buckling (or tension or in bending)

$\gamma_{M1}$ = 1.10 (see Clause 7.3.5.1.1 of EN 50341-1)

$\gamma_{M1}$ may be amended in the National Normative Aspects or the Project Specification (See Annex D).

The tower design shall be made by two methods:

- by calculation only or
- by calculation validated by a full scale loading test.

- If the design is done by **calculation only**, the imperfection factor $\alpha$ = 0.49 (buckling curve c). In this case the slenderness ratio for the different bracing types may be multiplied with a buckling length factor k given in the NNAs to take account of the influence of the end connections (See Clause 11.2).

- If the design is calculated and **validated by a full scale loading test**, then the imperfection factor $\alpha$ = 0.34 (buckling curve b). In this case the slenderness ratio for the different bracing types may <u>not</u> be multiplied with a buckling length factor (See Clause 11.2). The non-dimensional slenderness $\bar{\lambda}$ for the relevant buckling load is replaced by the effective slenderness $\bar{\lambda}_{eff}$ (See Clause 11.1).

**EN 1993-1-1 (Clause 6.3.1.2)**

EN 1993-1-1 gives the value of the reduction factor χ for the relevant buckling curve according to the imperfection factor α:

$$\chi = \frac{1}{\Phi + (\Phi^2 - \bar{\lambda}^2)^{0.5}} \qquad \text{but } \chi \le 1$$

$$\Phi = 0.5 \cdot (1 + \alpha \cdot (\bar{\lambda} - 0.2) + \bar{\lambda}^2)$$

$$\bar{\lambda} = \frac{\lambda}{\pi}\sqrt{\frac{f_y \cdot A_{eff}}{E \cdot A_g}}$$

where:

$A_{eff}$ - effective cross section area for uniform compression (See 10.2)

$\bar{\lambda}$ - non dimensional slenderness ratio for the relevant buckling load (See 11.1)

λ - slenderness ratio (See 11.2)

α - imperfection factor according to Table 10.1

**Table 10.1 – Buckling curves according to EN 1993-1-1**

| Buckling curve | $a_0$ | a | b | c | d |
|---|---|---|---|---|---|
| Imperfection factor α | 0.13 | 0.21 | 0.34 | 0.49 | 0.76 |

According to Table 6.2 “Selection of buckling curve for a cross section” of EN 1993-1-1 the appropriate buckling curve for angles (L-profiles) is b.

**EN 50341-3: National Normative Aspects**

BE → EN 50341-3-2 – Clause 7.3.5/BE.1 – $\gamma_{M1}$ = 1.00

Clause J.2.3/BE.1 – According to the Belgian Regulation the design method of ECCS 39 has to be applied for the calculation of the net and effective cross sections and the effective slenderness ratios (See Part 2.2 – General).

DE →EN 50341-3-4 – Clause 4.3.11/DE.1 – $\gamma_{M1}$ = 1.10

Clause J.5.1/DE.1 – In Germany the design formulae for the alternative “by calculation only” shall be used even if loading tests are carried out. Loading tests are valid only for the tested individual tower type and its particular extension. An assignment of the test results to other tower types within the tower family or to other extensions of the tower type is not permitted.

DK → EN 50341-3-5 – Clause J.5.1.a/DK.1 – The appropriate buckling curve shall be the curve c with imperfection factor α = 0.34.

FR → EN 50341-3-8 – Clause 7/FR.1 – The Interministerial Decree prescribes the mechanical checks to be made during overhead transmission line design. The national safety factors $g_R$ for material properties depend on load cases and materials. Other values than $g_R$ = 1.80 may be required in the Project Specification.

GB → EN 50341-3-9 – Clause 7.3/GB.1.a – Members and connections shall be designed in accordance with ECCS 39, with some modification (See Part 2.2 – General).

IT → EN 50341-3-13 – IT → EN 50341-3-13 – Clause 7.3.6/IT.1 – No partial safety factor for resistance $\gamma_M$ is required. No ultimate limit analysis is required (See Part 2.2 – General).

Clause 7.3.5/IT.1 – The stress in compressed member with $\lambda > 20$ shall be checked on gross section according to Tables I and II of EN 50341-3-13, Clause J.5/IT.1. See Part 2.2 - General.

NL →EN 50341-3-15 – Clause 7.3.5.1.1/NL.1 – $\gamma_{M1}$ = 1.00

Clause 7.3.5.3/NL.2 – The resistance of cross sections of members for axial compression shall be determined in accordance with the requirements of Clause 6.2.4 of ENV 1993-1-1.

NO →EN 50341-3-16 – Clause 7.3.5.1.1/NO.1 – $\gamma_{M1}$ = 1.10

SE → EN 50341-3-18 – Clause 7.3.5.1.1/SE.1 – $\gamma_{M1}$ = 1.15.

CZ → EN 50341-3-19 – Clause 7.3.5.3.1/CZ.1 – $\gamma_{M1}$ = 1.15

SI → EN 50341-3-21 – Clause 4.3.11/SI.3 – $\gamma_{M1}$ = 1.00

**ECCS 39 Recommendations (3)**

Allowable compression stress on the gross cross-sectional area:

$$f_a = \overline{\Lambda} \cdot f_y$$

$f_y$ - yield strength
$\overline{\Lambda}$ - modified non dimensional slenderness ratio (See Item 11.1)
$\Lambda$ - non dimensional slenderness ratio defined by

$$\Lambda = \frac{\lambda}{(\pi\sqrt{\frac{E}{\overline{\sigma}}})}$$

$$\lambda = kL / i$$

$\overline{\sigma}$ - actual or conventional yield strength (See 10.2)
L - unbraced length
i - radius of gyration corresponding to the buckling axis
k - effective buckling length coefficient (see Item 11.2)

The buckling curve computed taking into account only an initial sinusoidal deflection of 1/1000 the column length and no residual stresses fits very well with ECCS curve $a_0$. Therefore it has been adopted as basic buckling curve. Moreover it fits very well with the results collected by utilities from destructive tests of transmission towers.

**Brazilian Industry Practice**

In accordance with ASCE 10-97.

**Figure 10.1 – Diagonals buckling during test.**
**See the bracing provided by the tension diagonals.**

**Korean Industry Practice**

Allowable compression stress on the gross cross-sectional area ($N/mm^2$):

$$f_{ac} = f_{ao} - K_1 \cdot (kL/i)/100 - K_2 \cdot ((kL/i)/100)^2 \qquad \text{if } (L_k/i) \leq C_c$$

$$f_{ac} = 95/ ((kL/i)/100)^2 \qquad \text{if } (L_k/i) \geq C_c$$

$$\text{with } C_c = [(1.5\pi^2 E) /(2.2 \cdot K_o \cdot f_y)]^{1/2}$$

where:

$f_y$ - yield strength
$f_{ac}$ - allowable compression stress
$E$ - modulus of elasticity
$L_k$ - effective buckling length

$L_k = L \cdot k$

L - unbraced length (buckling length)

i - radius of gyration corresponding to the buckling axis

k - effective buckling length coefficient

- Leg members $k = 0.9$
- Other compression members:
  - with one bolt $k = 1.0$
  - with two or more bolts $k = 0.8$

– Leg members (single angle): $K_o = 0.5$

**Table 10.2 – Compression stress coefficient $K_o$ = 0,5 - Korean practices**

| Material | $f_y$ ($N/mm^2$) | $C_c$ | $f_{ao}$ ($N/mm^2$) | $K_1$ | $K_2$ |
|---|---|---|---|---|---|
| SS400 | 240 | 110 | 150 | 2.1 | 57.2 |
| SS400 | 250 | 105 | 155 | 2.3 | 60.2 |
| SS540 | 400 | 85 | 250 | 4.6 | 158.6 |
| SS540 | 410 | 85 | 255 | 4.7 | 165.4 |

– Leg members (Double angles; back to back) or pipe: $K_o = 0.6$

**Table 10.3 – Compression stress coefficient $K_o$ = 0.6 - Korean practices**

| Material | $f_y$ ($N/mm^2$) | $C_c$ | $f_{ao}$ ($N/mm^2$) | $K_1$ | $K_2$ |
|---|---|---|---|---|---|
| SS400 | 240 | 100 | 160 | 0 | 65.0 |
| SS400 | 250 | 100 | 165 | 0 | 70.0 |
| SS540 | 400 | 75 | 265 | 0 | 170.9 |
| SS540 | 410 | 75 | 270 | 0 | 129.8 |

– Other compression members (Eccentric loads): $K_o = 0.3$

**Table 10.4 – Compression stress coefficient $K_o$ = 0.3 - Korean practices**

| Material | $f_y$ ($N/mm^2$) | $C_c$ | $f_{ao}$ ($N/mm^2$) | $K_1$ | $K_2$ | Limit $f_{ac}$ ($N/mm^2$) |
|---|---|---|---|---|---|---|
| SS400 | 2400 | 140 | 1500 | 725 | 0 | 960 |
| SS400 | 2500 | 135 | 1550 | 762 | 0 | 1000 |
| SS540 | 4000 | 110 | 2500 | 1559 | 0 | 1600 |
| SS540 | 4100 | 105 | 2550 | 1608 | 0 | 1640 |

**Finnish Industry Practice**

No additional recommendation

**Icelandic Industry Practice**

No additional recommendation

**Italian Industry Practice**

No additional recommendation

**Figure 10.2 – Feet buckling during tests**

## 10.2 Local and torsional Buckling

For hot rolled steel members with the types of cross-section commonly used for compression members, the relevant buckling mode is generally *flexural* buckling. In some cases *torsional*, *flexural-torsional* or *local buckling modes may govern.* Local bucking and purely torsional bucking are identical if the angle has equal legs and is simply supported and free to warp at each end; furthermore, the critical stress for torsional-flexural bucking is only slightly smaller than the critical stress for purely flexural buckling, and for this reason such members have been customarily checked only for flexural and local buckling.

- For ASCE 10-97 $f_y$ from Clause 10.1 shall be replaced with $f_{cr}$.
- For EN 50341-1 b and A from Clause 10.1 shall be replaced with $b_{eff}$ and $A_{eff}$.
- For ECCS 39 $f_y$ from Clause 10.1 shall be replaced with $f_{cr}$.

## ASCE 10-97 Standard

$$(w/t)_{lim} = 670.8 / (f_y)^{1/2}$$

If $(w/t) < (w/t)_{lim}$ then the previous formulas for $f_a$ apply without any change.

If $670.8 / (f_y)^{1/2} \leq (w/t) \leq 1207.4 / (f_y)^{1/2}$

then the allowable compression stress $f_a$ shall be the value given by the previous expressions with $f_y$ replaced by $f_{cr}$ given by

$$f_{cr} = \{[\ 1.677 - 0.677\ [(w/t) / (w/t)_{lim}]\}\ f_y$$

If $(w/t) > 1207.4 / (f_y)^{1/2}$

$$f_{cr} = 668086 / (w/t)^2$$

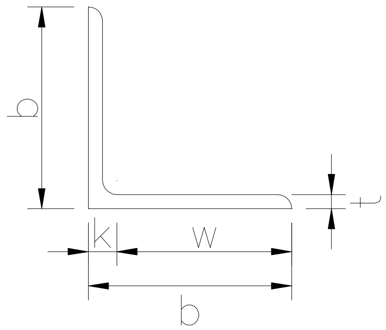



**Figure 10.3 – Determination of w/t ratios**

where:
- w - flat width of the member
- t - thickness of the member
- b - nominal width of the member
- R - rolling radius
  - $k = R + t$
  - $w = b - k$

Note: The provisions of this section are only applicable for 90° angles.

## EN 50341-1:2001 - European CENELEC Standard (J.2.3 & J.5.1.2)

The effective cross-section properties ($A_{eff}$ and $W_{eff}$) shall be based on the effective width $b_{eff} = \rho \cdot b$ of the leg. In case of an angle connected by one leg, the reduction applies only to the connected leg, due to the free leg being partially in tension.

The effective cross section, instead of gross cross section, generally do not need to be considered in the global elastic analysis.

$$\lambda_p = b / t$$

$$\overline{\lambda}_p = \lambda_p / (28.4\, \varepsilon \sqrt{K_\sigma})$$

with:

$K_\sigma = 0.43$

$$\varepsilon = \sqrt{(235 / f_y)}$$

$f_y$ – yield strength (expressed in MPa)
b – nominal width
t – thickness
ρ – reduction factor as follows

– <u>For rolled angles</u>

| | | | |
|---|---|---|---|
| $\rho = 1$ | if | | $\overline{\lambda}_p \leq 0.910$ |
| $\rho = 2 - \overline{\lambda}_p/0.910$ | if | $0.910 <$ | $\overline{\lambda}_p \leq 1.213$ |
| $\rho = 0.98 / \overline{\lambda}_p^{\,2}$ | if | | $\overline{\lambda}_p > 1.213$ |

– <u>For cold formed angles</u>

| | | | |
|---|---|---|---|
| $\rho = 1$ | if | | $\overline{\lambda}_p \leq 0.809$ |
| $\rho = (5 - \overline{\lambda}_p/0.404) / 3$ | if | $0.809 <$ | $\overline{\lambda}_p \leq 1.213$ |
| $\rho = 0.98/ \overline{\lambda}_p^{\,2}$ | if | | $\overline{\lambda}_p > 1.213$ |

It should be noted that EN 50341-1 defines (Clause J.5.1.2) that slenderness ratio for flexural torsional buckling should be calculated by an accepted formula and then be used in the calculation of the reduction factor χ. An approximation for that could be given by:

$$\overline{\lambda} = \frac{5}{\pi}\frac{b}{t}\sqrt{\frac{f_y \cdot A_{eff}}{E \cdot A}} \qquad \text{i.e. torsional buckling.}$$

$A_{eff}$ - effective cross section area when subjected to uniform compression

**<u>EN 50341-3: National Normative Aspects</u>**

BE → EN 50341-3-2 – Clause J.5.1.1/BE.2 – Calculation according to ECCS 39.

DE →EN 50341-3-4 – Clause J.5.1.2/DE.1 – For cruciform angle sections, torsional buckling shall be verified whereby the slenderness ratio may be calculated approximately using the formula:

$$\overline{\lambda} = 1{,}61\ \sqrt{\frac{I_P}{I_T}\frac{f_y}{E}\frac{A_{eff}}{A}}\ ,$$

where

- $I_P$ is the polar moment of inertia,
- $I_T$ is the torsional moment of inertia.

DK → EN 50341-3-5 – Clause J.2.3/DK.1 –The calculation of the effective cross section is applicable to class 3 and 4 elements. However, for class 3 elements, the effective widths $b_{eff}$ of the angles are identical to the nominal widths.

FI → EN 50341-3-7 – Clause J.5.1.2/FI.1 – The profiles of open cross sections shall be checked against torsional flexural buckling according to the standard ANSI/ASCE 10-90 or the recommendation ECCS 39. This is because of the improper specification and insufficient formulation of EN 50341-1 for this subject.

SE → EN 50341-3-18 – Clause 7.3.1/SE.2 – Resistance of members shall consider the thickness ratio b/t in accordance with rules in the Swedish Building and Design Regulation. For a hot-rolled, 90° angle bar the limit for unreduced area shall be in accordance with the formula $b / t \leq 260 / f_y^{1/2}$. For cold-formed bars the buckling calculation shall consider torsional-flexural buckling, see the Swedish Building and Design Regulation or Project Specification. For the upper limit of ratio b/t, see the Project Specification.

**ECCS 39 Recommendations (3)**

For taking into account in a simple and practical way the influence of local and torsional buckling on the overall buckling strength of rolled and cold formed angles, the actual or conventional yield stress, $\bar{\sigma}$, given by the following formulae must be adopted for the transformation of the non dimensional buckling curve $a_0$ adopted by ECCS 39.

- Equal leg rolled angle:

For $b/t < (b/t)_{lim} = 0.567\sqrt{E/f_y} = 260/\sqrt{f_y}$

$$\bar{\sigma} = f_y$$

For $(b/t)_{lim} < b/t < (4/3)(b/t)_{lim}$

$$\bar{\sigma} = f_y\left[2 - (b/t)/(b/t)_{\lim}\right]$$

For $b/t > (4/3)(b/t)_{lim}$

$$\bar{\sigma} = \pi^2 E/(5.1 \times b/t)^2 = 80000/(b/t)^2$$

- Equal leg cold formed angle:

For $b/t < (b/t)_{lim} = 0.503\sqrt{E/f_y} = 231/\sqrt{f_y}$

$$\bar{\sigma} = f_y$$

For $(b/t)_{lim} < b/t < (3/2)(b/t)_{lim}$

$$\bar{\sigma} = f_y\left[5/3 - (2/3)(b/t)/(b/t)_{\lim}\right]$$

For $b/t > (3/2)(b/t)_{lim}$

$$\bar{\sigma} = \pi^2 E/(5.1 \times b/t)^2 = 80000/(b/t)^2$$

$\bar{\sigma}$ - conventional yield strength
b - nominal width of the angle leg
t - plate thickness

**Brazilian Industry Practice**

In accordance with ASCE10-97

**Korean Industry Practice**

Not specified

**Finnish Industry Practice**

No additional recommendation

**Icelandic Industry Practice**

No additional recommendation

**Italian Industry Practice**

No additional recommendation

# Part 2.2 – Typical design practices for lattice steel towers

## Initial design considerations

**<u>EN 50341-1 – European CENELEC Standard (7.3)</u>**
The requirements of EN 1993-1-1 shall be complied with, except where otherwise specified in EN 50341-1 and the National Normative Aspects of EN 1993-1-3.

The requirements for lattice steel towers generally refer to angle members.

Those towers shall be so proportioned that the basic design requirements for the ultimate limit state are satisfied.

Lattice steel towers are normally considered as pin jointed truss structures.

The internal forces and moments in a statically indeterminate structure shall be determined using elastic global analysis. The elastic global analysis shall be based on the assumption that the stress-strain behaviour of the material is linear, whatever the stress level.

For the determination of the internal forces and moments:

- the first order theory, taking into account the initial geometry of the structure, is used for the global analysis of self-supporting lattice towers;
- the second order theory, taking into account the influence of the deformation of the structure, is used for guyed structures, at least as they are not embedded with pre-tensioned guys (See Clause 17).

It shall be verified that bracing systems have adequate stiffness to prevent local instability of any parts. Bending moments due to normal eccentricities are treated in the selection of buckling cases. If the continuity of a member is considered, the consequent secondary bending stresses may generally be neglected.

**<u>EN 50341-3: National Normative Aspects</u>**

BE→ EN 50341-3-2 – Clause J.5.1.1/BE.1 & 2 – Members and connections shall be designed in accordance with ECCS 39, modified following Clauses 11.2 (buckling length), 11.3 (maximum slenderness ratios) and 12 (redundant members).

CH→ EN 50341-3-3 – Clause 7.3.1/CH.1 – The prescriptions postulated by the Swiss Society of Engineers and Architects (SIA) in the Swiss National Application Document SIA 460.030 and SIA 460.031 have to be fulfilled additionally for lattice steel tower design.

ES → EN 50341-3-6 – Clause 7/ES.1 – The Clause 7 “Supports” of EN 50341-1 applies without change.

GB→ EN 50341-3-9 – Clause 7.3/GB.1 – Members and connections shall be designed in accordance with ECCS 39, modified following Clauses 11.1 ($\overline{\Lambda}$), 11.3 (maximum slenderness ratios) and 12 (redundant members).

GR → EN 50341-3-10 – Clause 7/GR.1 – The Clause 7 “Supports” of EN 50341-1 applies without change.

IT → EN 50341-3-13 – Clause 7.3.5/IT.1 – No ultimate limit analysis is required. The steel structure is so designed that the allowable stress values are not exceeded under the given loads, without safety factor. The allowable stress values for various steel types and for normal loading conditions are given in Tables I and II of EN 50341-3-13,

Clause J.5/IT.1 – The stress in tension members shall be checked on net section, with the allowable stress shown for $\lambda \leq 15$. The stress in compressed member with $\lambda > 20$ shall be checked on gross section.

NO→ EN 50341-3-16 – Clause 7.3.5.4/NO.1 – Unless otherwise stated in the Project Specification the resistance of cross sections against tension, compression and bending, and the buckling resistance of members shall be determined in accordance with normative annex J to Clause 7 "Supports" of EN 50341-1.

SE → EN 50341-3-18 – Clause 7.1/SE.1 – According to the Swedish Building and Design Regulation two partial factors $\gamma_M$ and $\gamma_n$ at the characteristic value of the material property have to be considered for the calculation of the design value of the material strength. In ultimate limit states $\gamma_n = 1.10$ (Safety Class 2). The Eurocode Standards shall be used with the Swedish National Application Documents.

Clause 7.3.1/SE.1 & J/SE.1 – If the Swedish Building and Design Regulation is used as an alternative instead of EN 1993-1-1, then only Clauses 11.2 (buckling length) and 12 (redundant members) shall be considered.

PL→ EN 50341-3-22 – Clause 7.3.1/PL.1 – When designing lattice steel towers, the requirements of EN 1993-1-1 and PN-EN 1993-3-1 shall apply.

# 11 Design of buckling lengths

Clause 11.1 modifies the buckling length to take account of end restraints (discontinuity or limited number of bolts) and eccentricities:

- For ASCE FFor ASCE 10-97 L/i is replaced with kL/i;
- For EN 50341_1 $\bar{\lambda}$ is replaced with $\bar{\lambda}_{eff}$ (only for calculation validated by documented full scale loading tests);
- For ECCS 39 $\Lambda$ is replaced with $\bar{\Lambda}$.

Clause 11.2 provides the various slenderness ratios $\lambda$ = kL/i to consider for all types of bracings (single, cross and multiple bracing with or without redundant members, K-bracing, etc.) as well as the appropriate buckling length and buckling axis.

Finally Clause 11.3 provides the maximum values recommended for L/i.

Distinction has been made between:

- Legs and chords;
- Primary bracing members;
- Secondary bracing members or redundant members (See Clause 12 for their design).

## 11.1 Effective buckling lengths for end restraints and eccentricities

**<u>ASCE 10-97 Standard</u>**

**a) Primary bracing members (excluding main legs, chords and redundant members)**

Curve 1 - members with concentric load at both ends of the unsupported panel

kL/i = L/i $0 \leq$ L/i $\leq$ 120 (according to ASCE Eq. 3.7-5)

Curve 2 - members with concentric load at one end and normal framing eccentricity at the other end of the unsupported panel

kL/i = 30 + 0.75 (L/i) $0 \leq$ L/i $\leq$ 120 (according to ASCE Eq. 3.7-6)

Curve 3 - members with normal framing eccentricities at both ends of the unsupported panel

kL/i = 60 + 0.5 (L/i) $0 \leq$ L/i $\leq$ 120 (according to ASCE Eq. 3.7-7)

Curve 4 - members unrestrained against rotation at both ends of the unsupported panel

kL/i = L/i $120 \leq$ L/i $\leq$ 200 (according to ASCE Eq. 3.7-8)

Curve 5 - members partially restrained against rotation at one end of the unsupported panel

kL/i = 28.6 + 0.762 (L/i) $120 \leq$ L/i $\leq$ 225 (*) (according to ASCE Eq. 3.7-9)

Curve 6 - members partially restrained against rotation at both ends of the unsupported panel kL/i = 46.2 + 0.615 (L/i) 120 ≤ L/i ≤ 250 (*)(according to ASCE Eq. 3.7-10)

*(*) To qualify a member for partial joint restraint the following rules are recommended (BPA criteria) :*

*a - the member shall be connected with at least two bolts*

*b - the connection shall be detailed to minimize eccentricity*

*c - the relative stiffness (I/L) of the member must be appreciably less than one or more other members attached to the same joint. This is required for each plane of buckling being considered. To provide restraint the joint itself must be relatively stiffer than the member to which it is providing restraint. The stiffness of the joint is a function of the stiffness of the other members attached to the joint.*

**b) Redundant Members**

kL/i = L/i 0 ≤ L/i ≤ 120 (according to ASCE Eq. 3.7-11)

Curve 4 - members unrestrained against rotation at both ends of the unsupported panel

kL/i = L/i 120 ≤ L/i ≤ 250 (according to ASCE Eq. 3.7-12)

Curve 5 - members partially restrained against rotation at one end of the unsupported panel

kL/i = 28.6 + 0.762 (L/i) 120 ≤ L/i ≤ 290 (according to ASCE Eq. 3.7-13)

Curve 6 - members partially restrained against rotation at both ends of the unsupported panel

kL/i = 46.2 + 0.615 (L/i) 120 ≤ L/i ≤ 330 (according to ASCE Eq. 3.7-14)

**<u>EN 50341-1:2001 - European CENELEC Standard (J.8 & J.9)</u>**

If the design is done by calculation validated by documented full scale loading tests, $\overline{\lambda}$ has to be replaced with $\overline{\lambda}_{eff}$ .

The non dimensional slenderness $\overline{\lambda}$ for the relevant buckling load is (see Item 10.1)

$$\overline{\lambda} = \frac{\lambda}{\pi} \times \sqrt{\frac{f_y \times A_{eff}}{E \times A_g}}$$

where:

$A_{eff}$ - effective area of the cross-section when subject to uniform compression (see Item 10.2)

For the design validated by full scale loading tower tests, the non-dimensional slenderness $\overline{\lambda}$ is replaced by the effective slenderness ratio $\overline{\lambda}_{eff}$, which shall be calculated as follows:

If: $\Lambda \leq \sqrt{2}$

Eccentricity at one end only:

Buckling about the vv axis $\overline{\lambda}_{eff} = 0.02 + 0.88\overline{\lambda}$

Buckling about the xx-yy axis $\overline{\lambda}_{eff} = 0.30 + 0.68\overline{\lambda}$

Eccentricity at both ends:

Buckling about the vv axis $\bar{\lambda}_{eff} = 0.30 + 0.68\bar{\lambda}$

Buckling about the xx-yy axis $\bar{\lambda}_{eff} = 0.52 + 0.68\bar{\lambda}$

If: √2 < Λ

Member continuous both ends:

Buckling about the vv, xx, yy axis $\bar{\lambda}_{eff} = e^{(1.747\bar{\lambda} - 1.98)}$ for $0.2 < \bar{\lambda} < 1.035$

$\bar{\lambda}_{eff} = 1.091\bar{\lambda} - 0.287$ for $\bar{\lambda} > 1.035$

Member continuous one end:

Buckling about the vv, xx, yy axis
with 2 or more bolts at connected end $\bar{\lambda}_{eff} = 0.30 + 0.68\bar{\lambda}$

Single bolt at connected end $\bar{\lambda}_{eff} = e^{(1.747\bar{\lambda} - 1.98)}$ for $0.2 < \bar{\lambda} < 1.035$

$\bar{\lambda}_{eff} = 1.091\bar{\lambda} - 0.287$ for $\bar{\lambda} > 1.035$

Discontinuous member:

a) 2 or more bolts each end

Buckling about the vv axis $\bar{\lambda}_{eff} = 0.30 + 0.68\bar{\lambda}$

Buckling about the xx-yy axis $\bar{\lambda}_{eff} = 0.52 + 0.68\bar{\lambda}$

b) single bolt both ends

Buckling about the vv axis $\bar{\lambda}_{eff} = e^{(1.747\bar{\lambda} - 1.98)}$ for $0.2 < \bar{\lambda} < 1.035$

$\bar{\lambda}_{eff} = 1.091\bar{\lambda} - 0.287$ for $\bar{\lambda} > 1.035$

Buckling about the xx-yy axis $\bar{\lambda}_{eff} = 0.16 + 0.94\bar{\lambda}$

The appropriate buckling case can also be selected from the following Table 11.1:

**Table 11.1 – Buckling cases according to EN 50341-1**

| Buckling cases | | | | | | |
|---|---|---|---|---|---|---|
| | Buckling axis | Slenderness condition $\bar{\lambda}$ | Load eccentricity condition | Member continuity condition (1) | Number of bolts at non-continuous end | N° case |
| Leg member | VV | > or < √2 | Leg with symmetrical bracing (Fig. 11.4.a & b) | | | 1 |
| | YY or ZZ | > or < √2 | Leg with intermediate transverse support (Fig. 11.4.c) | | | 1 |
| | | > or < √2 | Leg with staggered bracing (Fig. 11.4.d) | | | 2 |
| Bracing member | VV | < √2 | 1 end | - | - | 3 |
| | | < √2 | 2 ends | - | - | 4 |
| | | < √2 | - | 2 ends | - | 1 |
| | | < √2 | - | 1 end | 2 bolts | 4 |
| | | < √2 | - | 1 end | 1 bolt | 1 |
| | | < √2 | - | 0 end | 2 bolts | 4 |
| | | < √2 | - | 0 end | 1 bolt | 1 |
| | YY or ZZ | < √2 | 1 end | - | - | 4 |
| | | < √2 | 2 ends | - | - | 5 |
| | | < √2 | - | 2 ends | - | 1 |
| | | < √2 | - | 1 end | 2 bolts | 4 |
| | | < √2 | - | 1 end | 1 bolt | 1 |
| | | < √2 | - | 0 end | 2 bolts | 5 |
| | | < √2 | - | 0 end | 1 bolt | 6 |

(1) - Member continuity conditions are:

2 ends = the member is continuous at both ends

1 end = the member is continuous at one end only

0 end = single span member

A bracing member fixed to both legs is considered as leg member

A bracing member fixed by welding is considered as connected by two bolts

with:

Case 1 $\bar{\lambda}_{eff} = e^{(1.747\bar{\lambda}-1.98)}$ for $0.2 < \bar{\lambda} < 1.035$

$\bar{\lambda}_{eff} = 1.091\bar{\lambda} - 0.287$ for $\bar{\lambda} > 1.035$

Case 2 $\bar{\lambda}_{eff} =$ as case 1 with $\bar{\lambda} = 1.2$ times $\bar{\lambda}$ of case 1

Case 3 $\bar{\lambda}_{eff} = 0.02 + 0.88\bar{\lambda}$

Case 4 $\bar{\lambda}_{eff} = 0.30 + 0.68\bar{\lambda}$

Case 5 $\bar{\lambda}_{eff} = 0.52 + 0.68\bar{\lambda}$

Case 6 $\bar{\lambda}_{eff} = 0.16 + 0.94\bar{\lambda}$

**<u>EN 50341-3: National Normative Aspects</u>**

BE → EN 50341-3-2 – Clause J.5.1.1/BE.2 – Calculation according to ECCS 39.

DE → EN 50341-3-4 – Clause 7.3.5.2.1/DE.2 – The eccentricity of the connections of members at nodes shall be kept as small as possible. For leg members of lattice steel towers the eccentricity at joints may be disregarded provided that the centroidal axis of the joint area is calculated in an average position. In case of compression bracing members of lattice steel towers consisting of one single angle (for example members between leg members or between chords) being connected by only one of the angle legs the eccentricity of load application may be disregarded.

DK → EN 50341-3-5 – Clause J.5.1.c/DK.1 – If the design of the tower family is supported by a documented full scale loading test, the non-dimensional slenderness $\bar{\lambda}$ for the relevant buckling load in the EN 1993-1-1 equation is replaced by the effective slenderness $\bar{\lambda}_{eff}$.

FI → EN 50341-3-17 – Clause J.6/FI.1 – The buckling length factors shall be taken from the Eurocode.

GB → EN 50341-3-9 – Clause J.3/GB.1.a – The non dimensional slenderness ratio $\Lambda$ shall be in accordance with ECCS 39, modified as follows:

If: $\Lambda \leq \sqrt{2}$

No eccentricity at both ends; supported with asymmetric bracing:

Buckling about the xx-yy axis $\quad \bar{\Lambda} = 0.03 + 0.6464\ \Lambda$

Discontinuity at one end only and connected with a single bolt at the connected end:

Buckling about the vv axis $\quad \bar{\Lambda} = 0.25 + 0.8232\ \Lambda$

Buckling about the xx-yy axis $\quad \bar{\Lambda} = 0.50 + 0.6464\ \Lambda$

Discontinuity at both ends only and connected with a single bolt at each end:

Buckling about the xx-yy axis $\quad \bar{\Lambda} = 0.707 + 0.6464\ \Lambda$

If: $\sqrt{2} < \Lambda$

No eccentricity at both ends; supported with asymmetric bracing or
Discontinuity at both ends only and connected with a single bolt at each end:

Buckling about the xx-yy axis $\quad \bar{\Lambda} = 0.40 + 0.8635\ \Lambda$

NL → EN 50341-3-15 – Clause 7.3.5.3/NL.9 – If bracing members are sufficiently supported by leg members and connected with at least two bolts, the eccentricities do not have to be taken in account in the stability check, the effective slenderness may be taken as:

$\lambda_{eff} = 0{,}35 + 0{,}7\ \lambda_{vv}$, for the minimum axis,

$\lambda_{eff} = 0{,}50 + 0{,}7\ \lambda_{yy}$, for the rectangular axis.

If the bracing members are connected with one bolt for the stability check related to the rectangular axis the eccentricities shall be taken in account.

SE → EN 50341-3-18 – Clause 7.3.1/SE.2 – Calculation of buckling resistance shall consider essential eccentricity in the attachment of the member or bending moment in accordance with rules in the Swedish Building and Design Regulation.

Clause J.6.3.1/SE.1.2 – Amendment: For an angle bar connected in only one flange the effective slenderness ratio $\bar{\lambda}_{eff}$ shall be calculated as follows:

$\bar{\lambda}_{eff} = 0{,}60 + 0{,}57 \cdot \bar{\lambda}$ for $\bar{\lambda} < 1{,}4$

$\bar{\lambda}_{eff} = \bar{\lambda}$ for $\bar{\lambda} \geq 1{,}4$

SI → EN 50341-3-21 – Clause 7.3.5.2.1/SI.2 – See Clause 7.3.5.2.1/DE.2 above.

**<u>ECCS 39 Recommendations (5.4)</u>**

For low slenderness ratios ($\Lambda \leq \sqrt{2}$) the eccentricity of axial loads the buckling strength has to be penalized. At higher slenderness ratios ($\sqrt{2} < \Lambda < 3$) the effect of the end restraint has a beneficial effect on the buckling strength.

Correction formulae for the non dimensional slenderness ratio Λ (see Item 10.1) of web members:

If: $\Lambda \leq \sqrt{2}$

Eccentricity at one end only:

| | |
|---|---|
| Buckling about the vv axis | $\bar{\Lambda} = 0.250 + 0.8232\ \Lambda$ |
| Buckling about the xx-yy axis | $\bar{\Lambda} = 0.500 + 0.6464\ \Lambda$ |

Eccentricity at both ends:

| | |
|---|---|
| Buckling about the vv axis | $\bar{\Lambda} = 0.500 + 0.6464\ \Lambda$ |
| Buckling about the xx-yy axis | $\bar{\Lambda} = 0.707 + 0.6464\ \Lambda$ |

These modified slenderness ratios assume an end fixity resulting from 2 or more bolts at the end connections. For single bolted members the capacity of the member is normally governed by the capacity of bolted connection.

If: $\sqrt{2} < \Lambda < 3$

Member continuous both ends:

| | |
|---|---|
| Buckling about the vv axis | $\bar{\Lambda} = 0.500 + 0.6464\ \Lambda$ |
| Buckling about the xx-yy axis | $\bar{\Lambda} = \Lambda$ |

Member continuous one end:

| | |
|---|---|
| Buckling about the vv, xx, yy axis | |
| with 2 or more bolts at connected end | $\bar{\Lambda} = 0.500 + 0.6464\ \Lambda$ |
| Single bolt at connected end | $\bar{\Lambda} = \Lambda$ |

Discontinuous member:

a) 2 or more bolts each end

| | |
|---|---|
| Buckling about the vv axis | $\bar{\Lambda} = 0.500 + 0.6464\ \Lambda$ |
| Buckling about the xx-yy axis | $\bar{\Lambda} = 0.707 + 0.6464\ \Lambda$ |

b) single bolt both ends

| | |
|---|---|
| Buckling about the vv axis | $\bar{\Lambda} = \Lambda$ |
| Buckling about the xx-yy axis | $\bar{\Lambda} = 0.400 + 0.8635\ \Lambda$ |

This increment of the buckling load is obtained only if the end connections have two or more bolts and the leg is stiffer than the connected web member.

**<u>Brazilian Industry Practice</u>**

In accordance with ASCE 10-97

**<u>Korean Industry Practice</u>**

Primary Bracing Members: Both bolts positions should be located at 5% length of whole member length from both ends respectively.

Redundant Members: Bolts positions should be located at 10 % length of whole member length from both ends respectively.

**<u>Finnish Industry Practice</u>**

No additional recommendation

**<u>Icelandic Industry Practice</u>**

No additional recommendation

**<u>Italian Industry Practice</u>**

No additional recommendation

### 11.2 Buckling length for different bracing types

**<u>ASCE 10-97 Standard</u>**

**Main members bolted in both faces at connections**

$kL/i = L/i$ $\qquad 0 \le L/i \le 150$ (according to ASCE Eq. 3.7-4)

For main members of equal angles, having no change in member force between panels, designed with staggered bracing, the controlling L/i ratio shall be as shown in the Figures 11.1, 11.2 and 11.3 here below:

**(a) Leg Members Controlled by $(2L/3)/i_{zz}$**

Leg members shall be supported in both faces at the same elevation level every four panels

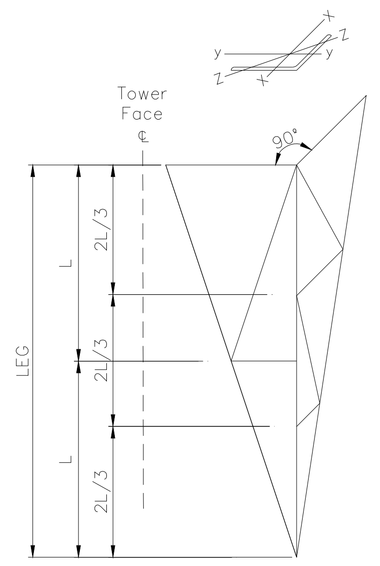



**Figure 11.1 – Leg members effective buckling lengths (a)**

**(b) Leg Members Controlled by $(1.2L)/i_{xx}$**

Leg members shall be supported in both faces at the same elevation level every four panels

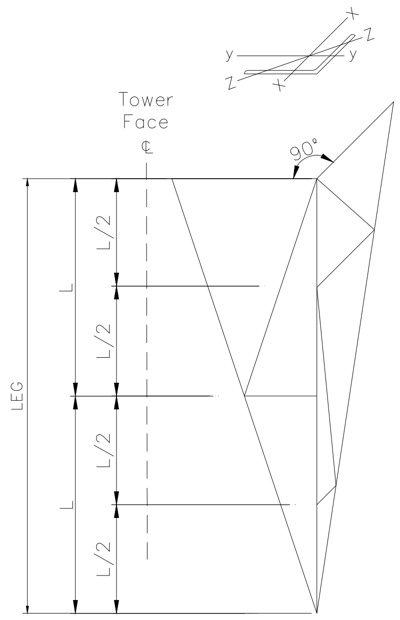



**Figure 11.2 – Leg members effective buckling lengths (b)**

**c) Leg Members Controlled by $(1.2L)/i_{xx}$**

For these configurations some rolling of the leg will occur. Eccentricities at leg splices shall be minimised. The thicker leg sections shall be properly butt spliced. The controlling L/r values shown shall be used with k =1.

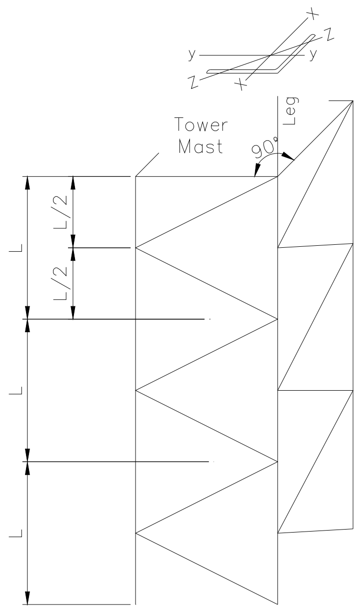



**Figure 11.3 – Leg of tower mast effective buckling lengths**

**As per appendix B:**

**d) Leg Members with Symmetrical Bracing**

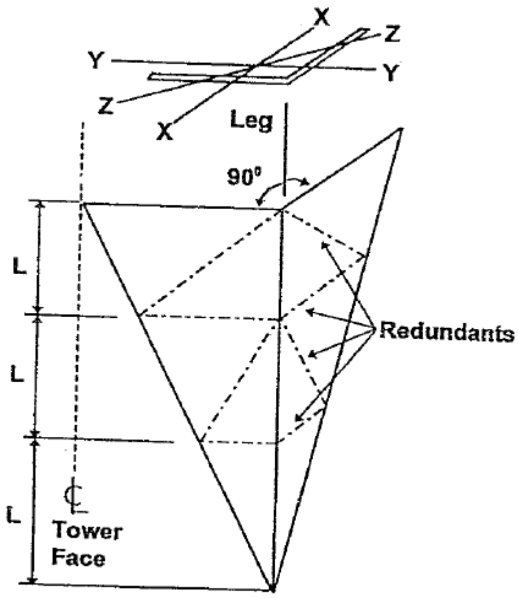



**Figure 11.4 – Leg Members with Symmetrical Bracing**

Leg Requirements:
L/rzz critical factor, Concentric loading, Eq. 3.7-4.
Eccentricities at leg splices should be minimized.
Thicker leg sections should be properly butt-spliced.
This method of support minimizes rolling of the angle under load.

**Diagonal and Bracing Members**

**As per appendix B:**

**e) Effect of end Connections on Member Capacity Single Angle Detail**

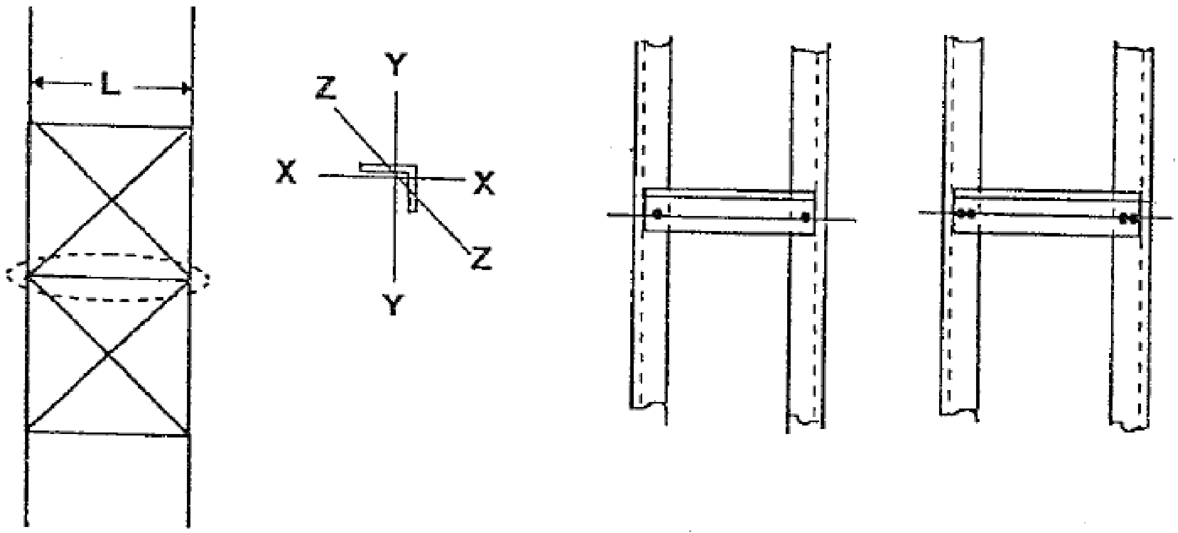



**Figure 11.5 – Effect of end Connections on Member Capacity Single Angle Detail**

**Tension – Only System with Compression Struts.**
$L/r_{zz}$ Critical Factor
Eccentricity in critical axis.
$L/r_{zz}$ = from 0 to 120
(Eq. 3.7-7).

**Single Bolt Connection; No Restraint at Ends.**
$L/r_{zz}$ = from 120 to 250 (Eq. 3.7-8).

**Multiple Bolt Connection; Partial Restraint at Both Ends.**
$L/r_{zz}$ = from 120 to 250 (Eq. 3.7-10).

**f) Concentric Loading Two Angles Member**

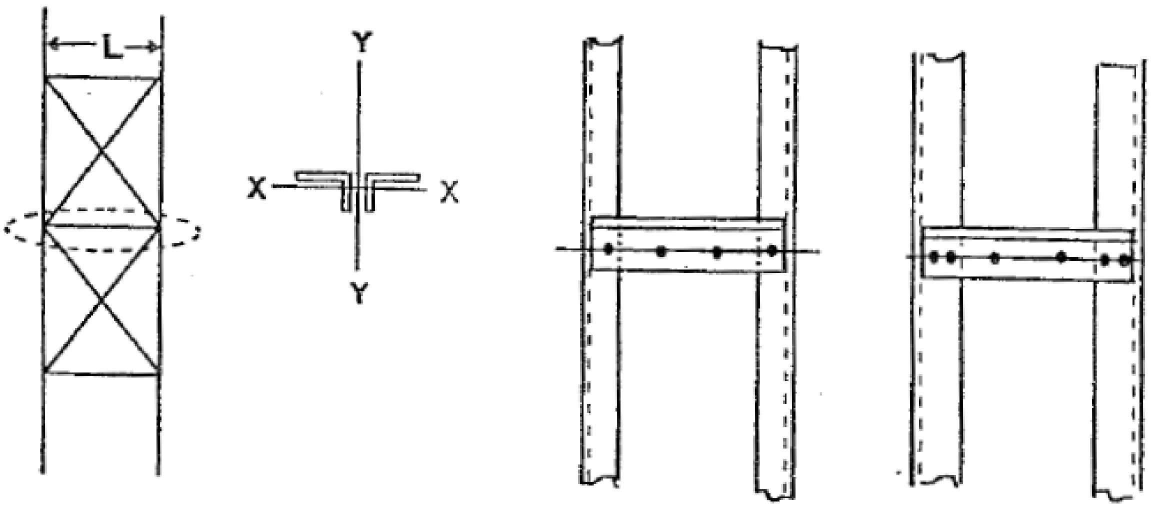
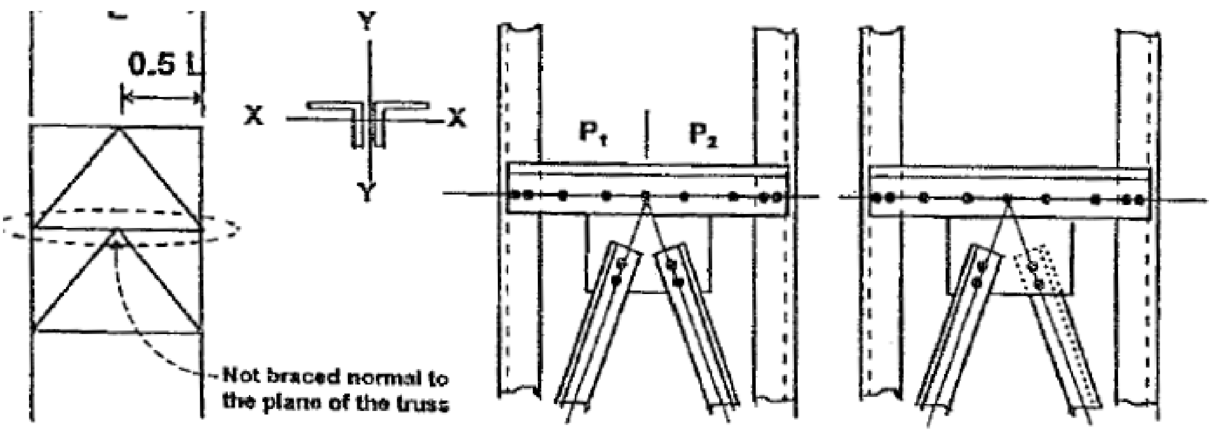



**Figure 11.6 – Concentric Loading Two Angles Member**

| | | |
|---|---|---|
| **Tension – Only System with Compression Struts.** | **Single Bolt Connection; No Restraint at Ends.** | **Multiple Bolt Connection; Partial Restraint at Both Ends.** |
| $L/r_{xx}$ or $L/r_{yy} =$ Critical Factor Concentric Loading.<br>$L/r_{xx}$ or $L/r_{yy} =$ fron 0 to 120 (Eq. 3.7-5). | $L/r_{xx}$ or $L/r_{yy} =$ = from 120 to 200 (Eq. 5.7-8). | $L/r_{xx}$ or $L/r_{yy} =$ = from 120 to 250 (Eq. 5.7-10). |




(a) (b)

**Figure 11.7 – K-Bracing Two Angles Member**

**Bracing Requirements**

| | | |
|---|---|---|
| **Tension – Compression System with Compression Struts.** | **Concentric Load at Ends, Eccentric Loading at Intermediate in Both Directions, (a).** | **Concentric Loading at Ends and Intermediate, (b).** |

<u>Constant Load (P1=P2) in Double Angle</u>

| | | |
|---|---|---|
| Multiple bolt connections 0.5 $L/r_{xx}$ or $L/r_{yy} =$ Critical Factor. | 0.5 $L/r_{xx}$ (Eq. 3.7-6) or $L/r_{yy}$ = (Eq. 3.7-5) from 0 to 120. | 0.5 $L/r_{xx}$ or $L/r_{yy}$ from 0 to 120 (Eq. 3.7-5). |

Partial restraint at one end.
$0.5\ L/r_{xx}$ from 120 to 225 (Eq. 3.7-9).
See Statement in Section C3.7.4(c).

Partial restraint at both ends.
$L/r_{yy}$ from 120 to 250 (Eq. 3.7-10).

<u>Varying Load ($P_1$ not equal to $P_2$) in Equal Length Subpanel of Double Angle</u>

Check member for forces $P_1$ for in-plane buckling using $0.5\ L/r_{xx,}$ as given previously.

Check out-of-plane buckling on the y-y axis of the angle over the length L, as follows.

$KL/r_{yy,} = K(K'L/r_{yy,})$ where K' shall be determined from:

1. When $P_1$ and $P_2$ are compression forces use Eq. 3.7-15a

   $K' = 0.75 + 0.25\ (P_2/P_1)$, where $P_1 > P_2$

2. When either $P_1$ or $P_2$ is a tension force use Eq. 3.7-15b

   $K' = 0.75 - 0.25\ (P_2/P_1)$, where $P_2$ is the tension force.

**h) Effect of Subdivides Panels and End Connects**

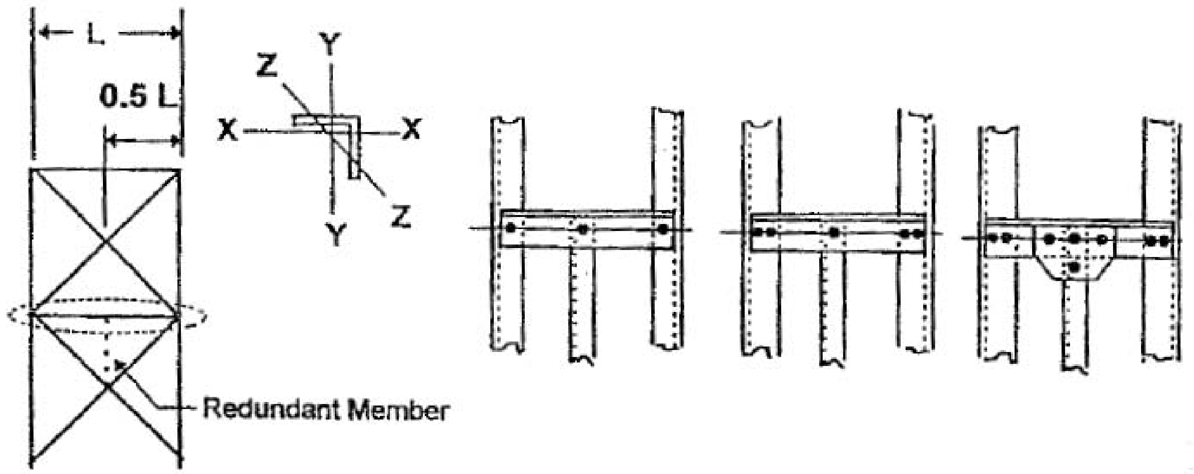



**Figure 11.8 – Effect of Subdivides Panels and End Connects**

| | | | |
|---|---|---|---|
| **Tension – Only System with Compression Struts.**<br>$0.5L/r_{zz}$ or $L/r_{yy}$ = critical factor. Eccentricity in critical axis.<br>$0.5L/r_{zz}$ (Eq. 3.7-6) or $L/r_{yy}$ (Eq. 3.7-7) from 0 to 120. | **Single Bolt Connection; No Restraint at Ends or Intermediate.**<br>$0.5L/r_{zz}$ or $L/r_{yy}$ from 120 to 200 (Eq. 3.7-8). | **Multiple Bolt Connection at Ends; Single Bolt Connection at Intermediate Point. Partial Restraint at One End. No Restraint at Intermediate.**<br>$0.5L/r_{zz}$ from 120 to 225 (Eq. 3.7-9). | **Multiple Bolt Connection; Partial Restraint at Ends and Intermediate.**<br>$0.5L/r_{zz}$ or $L/r_{yy}$ from 120 to 250 (Eq. 3.7-10). |

### i) Concentric Loading, Two Angle Member, Subdivided Panels

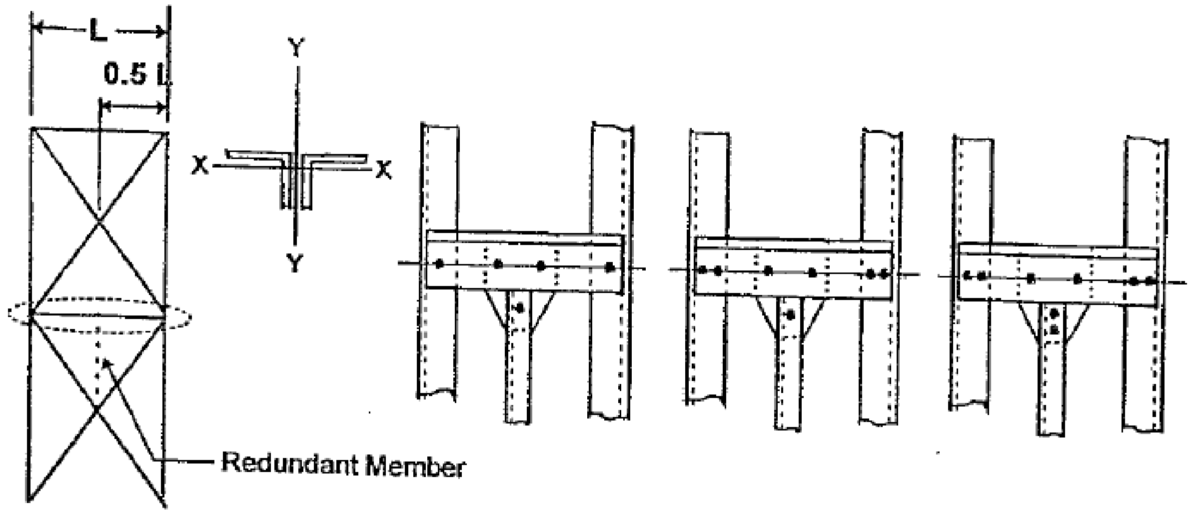



**Figure 11.9 – Concentric Loading, Two Angle Member, Subdivided Panels**

| **Tension – Only System with Compression Struts.** | **Single Bolt Connection; No Restraint at Ends or Intermediate.** | **Multiple Bolt Connection at Ends; Single Bolt Connection at Intermediate Point. Partial Restraint at One End. No Restraint at Intermediate.** | **Multiple Bolt Connection; Partial Restraint at Ends and Intermediate.** |
|---|---|---|---|
| | | **See statement in Section C3.7.4 concerning partial restraint** | |
| $0.5L/r_{zz}$ or $L/r_{yy}$ = critical factor. Concentric loading $0.5L/r_{xx}$ or $L/r_{yy}$ from 0 to 120. (Eq. 3.7-5) | $0.5L/r_{xx}$ or $L/r_{yy}$ from 120 to 200 (Eq. 3.7-8). | $0.5L/r_{xx}$ from 120 to 225 (Eq. 3.7-9).<br><br>**Partial restraint at both ends.**<br><br>$L/r_{yy}$ from 120 to 250 (Eq. 3.7-10). | $0.5L/r_{xx}$ or $L/r_{yy}$ from 120 to 250 (Eq. 3.7-10). |

### j) X-Brace Systems with no Intermediate Redundant Support

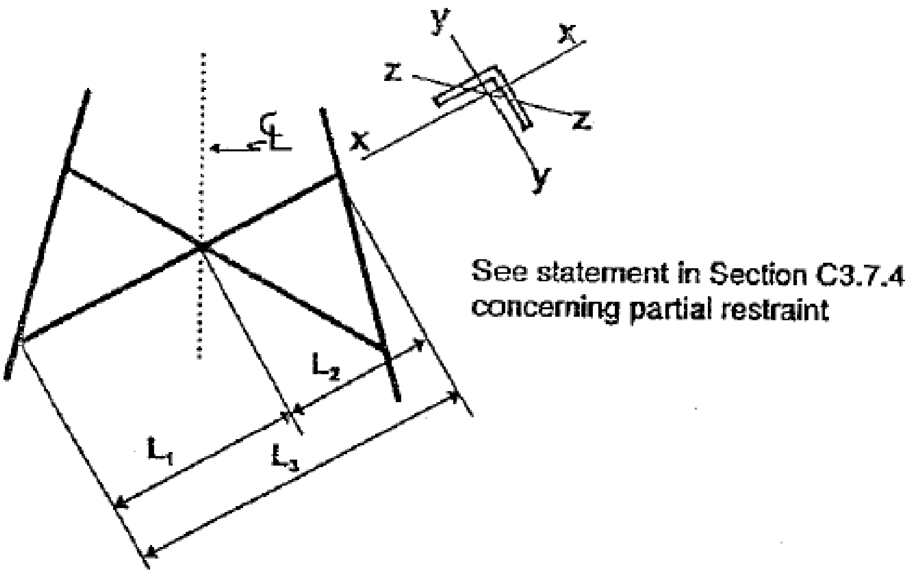



**Figure 11.10 – X-Brace Systems with no Intermediate Redundant Support**

1. Tension/Compression system with members connected at the crossover point. If the member in tension has a force of not less than 20% of the force in the compression member, the crossover point provides support resisting out-of-plane buckling.

$$L_1 \geq L_2$$

For L/r from 0 to 120.

Critical factor $\mathbf{L/r_{zz}}$ Use Eq. 3.7-6.

For L/r from 120 to 200.

Critical factor $\mathbf{L/r_{zz}}$
For single bolt connections at the post leg use Eq. 3.7-8.
For multiple bolt connections at the post leg use Eq. 3.7-9.
Providing end restraint can be assumed at the post leg; otherwise use Eq.3.7-8.

*Note:* When unequal legs are used and the long leg is connected, the critical factor for out-of-plane buckling should be $(L_1 + 0.5L_2)/r_{yy}$ using the appropriate equation for *KL/r* as given previously.

2. Tension/Compression system with members connected at the crossover point. If the member in tension has a force of not less than 20% of the force in the compression member, or if both members in the same panel are in compression, the crossover point provides support resisting out-of-plane buckling.

$$L_1 \geq L_2$$

For L/r from 0 to 120.

Critical factor $\mathbf{L/r_{zz}}$ Use Eq. 3.7-6.
Critical factor $\mathbf{L/r_{xx\ or\ yy}}$ Use Eq. 3.7-7.

For L/r from 120 to 200.

(a) Critical factor $\mathbf{L/r_{zz}}$
For single bolt connections at the post leg use Eq. 3.7-8.
For multiple bolt connections at the post leg use Eq. 3.7-9.
Providing end restraint can be assumed at the post leg; otherwise use Eq.3.7-8.

(b) Critical factor $\mathbf{L/r_{xx\ or\ yy}}$
For single bolt connections at the post leg use Eq. 3.7-8.
For multiple bolt connections at the post leg use Eq. 3.7-10.
Providing end restraint can be assumed at the post leg; otherwise use Eq.3.7-8.

*Note:* When unequal legs are used and the long leg is connected, the critical factor for out-of-plane buckling should be $L_3/r_{yy}$ using the appropriate equation for KL/r as given previously.

### k) X-Brace Systems with no Intermediate Redundant Support – Case 1

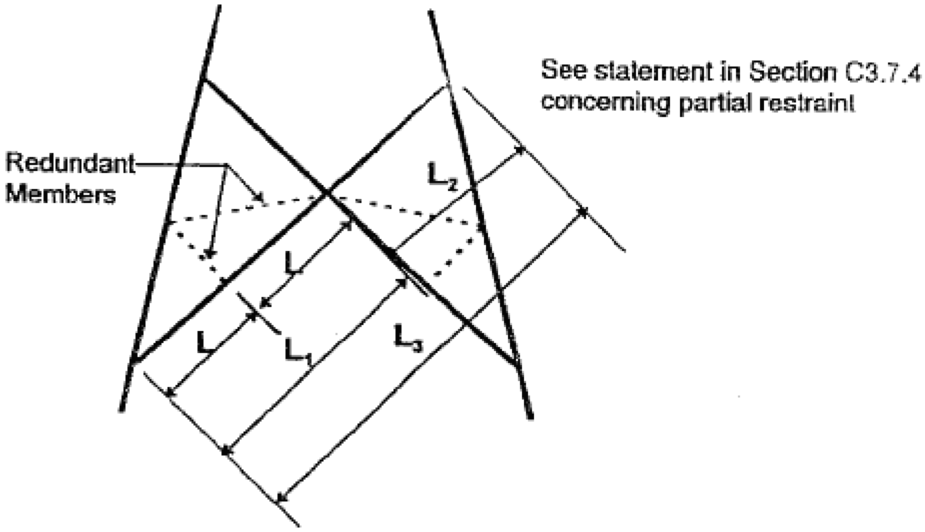



**Figure 11.11 – X-Brace Systems with no Intermediate Redundant Support**

1. Tension/Compression system with members connected at the crossover point. If the member in tension has a force of not less than 20% of the force in the compression member, the crossover point provides support resisting out-of-plane buckling.

$$L_1 > L_2$$

For L/r from 0 to 120.

Critical factor $\mathbf{L/r_{zz}}$ or $\mathbf{L_2/r_{zz}}$ whichever is maximum. Use Eq. 3.7-6.

Critical factor $\mathbf{L/r_{xx\ or\ yy}}$ Use Eq. 3.7-6.

For L/r from 120 to 200.

(a) Critical factor $\mathbf{L/r_{zz}}$ Use Eq. 3.7-8; or $\mathbf{L_2/r_{zz}}$ using Eq. 3.7-9 if end restraint can be assumed; otherwise use Eq. 3.7-8.

(b) Critical factor $\mathbf{L/r_{xx\ or\ yy}}$

For single bolt connections. Use Eq. 3.7-8.

For multiple bolt connections where end restraint can be assumed at one end, use Eq. 3.7-9; otherwise use Eq. 3.7-8.

No end restraint is assumed in segment between crossover point and intermediate redundant support point.

*Note:* When unequal legs are used and the long leg is connected, the critical factor for out-of-plane buckling should be $(L_1 + 0.5L_2)/r_{yy}$ using the appropriate equation for *KL/r* as given previously.

2. Tension/Compression system with members connected at the crossover point. If the member in tension has a force of not less than 20% of the force in the compression member, or if both members in the same panel are in compression, the crossover point provides support resisting out-of-plane buckling.

$$L_1 > L_2$$

For L/r from 0 to 120.

Critical factor $\mathbf{L/r_{zz}}$ or $\mathbf{L_2/r_{zz}}$ whichever is maximum. Use Eq. 3.7-6.

Critical factor $\mathbf{L/r_{xx\ or\ yy}}$ Use Eq. 3.7-7.

For L/r from 120 to 200.

(a) Critical factor $L/r_{zz}$ Use Eq. 3.7-8; or $L_2/r_{zz}$ using Eq. 3.7-9 if end restraint can be assumed; otherwise use Eq. 3.7-8.

(b) Critical factor $L/r_{xx \text{ or } yy}$

For single bolt connections at the post legs use Eq. 3.7-8.

For multiple bolt connections at the post legs use Eq. 3.7-10 if end restraint can be assumed at the post leg; otherwise use Eq. 3.7-8.

No end restraint is assumed in segment between crossover point and intermediate redundant support point.

**l) X-Brace Systems with no Intermediate Redundant Support – Case 2**

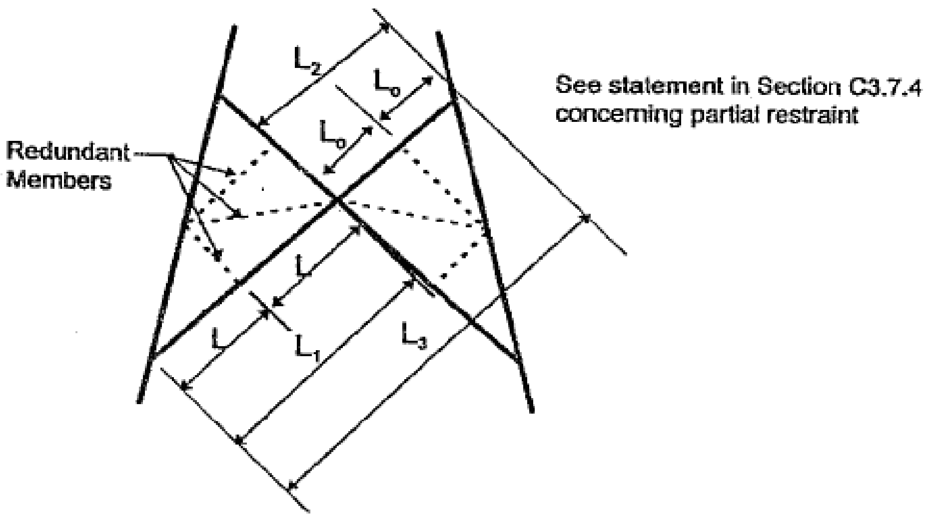



**Figure 11.12 – X-Brace Systems with no Intermediate Redundant Support**

1. Tension/Compression system with members connected at the crossover point. If the member in tension has a force of not less than 20% of the force in the compression member, the crossover point provides support resisting out-of-plane buckling.

$$L_1 \geq L_2 \text{ and } L \geq L_0$$

For L/r from 0 to 120.

Critical factor $L/r_{zz}$ Use Eq. 3.7-6.

Critical factor $L/r_{xx \text{ or } yy}$ Use Eq. 3.7-6.

For L/r from 120 to 200.

(a) Critical factor $L/r_{zz}$ Use Eq. 3.7-8.

(b) Critical factor $L/r_{xx \text{ or } yy}$

For single bolt connections. Use Eq. 3.7-8.

For multiple bolt connections where end restraint can be assumed at one end, use Eq. 3.7-9; otherwise use Eq. 3.7-8.

No end restraint is assumed in segment between crossover point and intermediate redundant support point.

*Note:* When unequal legs are used and the long leg is connected, the critical factor for out-of-plane buckling should be $(L_1 + 0.5L_2)/r_{yy}$ using the appropriate equation for *KL/r* as given previously.

2. Tension/Compression system with members connected at the crossover point. If the member in tension has a force of not less than 20% of the force in the compression member, or if both members in the same panel are in compression, the crossover point provides support resisting out-of-plane buckling.

$$L_1 > L_2 \text{ and } L > L_0$$

For L/r from 0 to 120.

Critical factor $\mathbf{L/r_{zz}}$ or $\mathbf{L_2/r_{zz}}$ whichever is maximum. Use Eq. 3.7-6.

Critical factor $\mathbf{L/r_{xx\ or\ yy}}$ Use Eq. 3.7-7.

For L/r from 120 to 200.

(a) Critical factor $\mathbf{L/r_{zz}}$ Use Eq. 3.7-8.

(b) Critical factor $\mathbf{L/r_{xx\ or\ yy}}$

For single bolt connections. Use Eq. 3.7-8.

For multiple bolt connections at the post legs use Eq. 3.7-10 if end restraint can be assumed at the post leg; otherwise use Eq. 3.7-8.

No end restraint is assumed in segment between crossover point and intermediate redundant support point.

*Note:* When unequal legs are used and the long leg is connected, the critical factor for out-of-plane buckling should be $(L_1 + 0.5L_2)/r_{yy}$ using the appropriate equation for *KL/r* as given previously.

**<u>EN 50341-1:2001 - European CENELEC Standard (J.6.2 & J.6.3)</u>**

The appropriate slenderness ratio λ for the relevant buckling mode shall be determined from the following bracing types.

In the case of design not validated by loading test the slenderness ratio λ for the different bracing types may be multiplied with a buckling length factor given in the NNAs (See Clause 10.1).

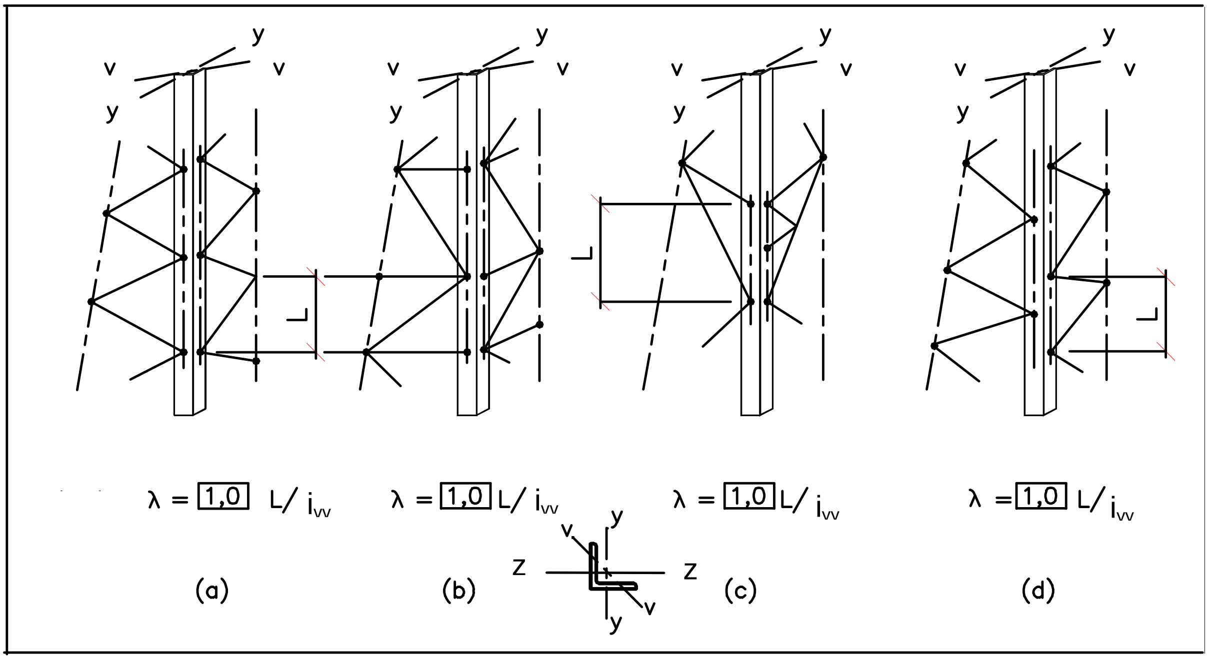



**Figure 11.13 – Symmetrical and staggered bracing to legs**

- Legs and chords (Figure 11.4) (J.6.2.2)
  - $\lambda = L/i_{vv}$ - on the axis of minimum inertia about vv for legs with symmetrical bracing (Figure 11.4.a and b)
  - $\lambda = L/i_{yy}$ - on the rectangular axis about yy for legs with intermediate transverse support (Figure 11.4.c)
  - $\lambda = L/i_{yy}$ - on the rectangular axis about yy for legs with staggered bracing (Figure 11.4.d)

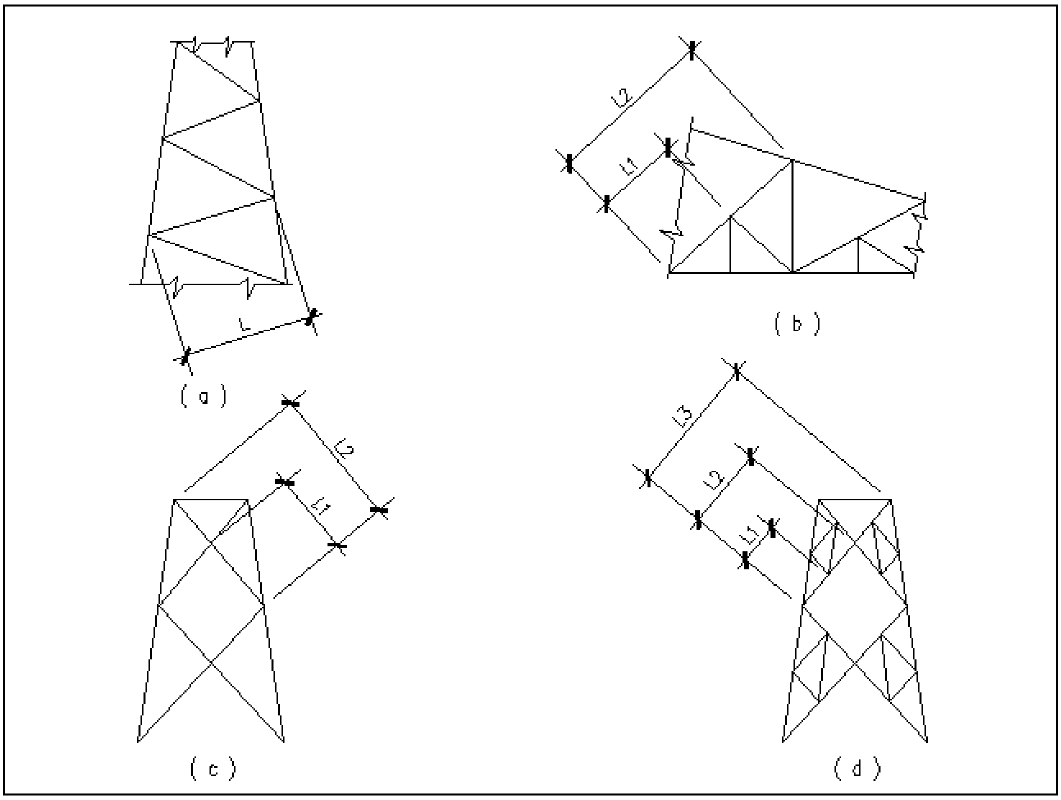



**Figure 11.14 – Typical bracing patterns**

- Single bracing (Figure 11.5.a) (J.6.3.2)
  - $\lambda = L/i_{vv}$ on the axis of the minimum inertia about vv

- Single lattice bracing with redundant members (Figure 11.5.b) (J.6.3.2)
  - $\lambda_1 = L_1/i_{vv}$ on the axis of minimum inertia about vv over buckling length $L_1$
  - $\lambda_2 = L_2/i_{yy}$ on the rectangular axis about yy over the overall buckling length $L_2$

- Cross bracing (Figure 11.5.c) (J.6.3.3)
  - $\lambda_1 = L_1/i_{vv}$ on the axis of minimum inertia about vv over the buckling length $L_1$ as the intersection of the cross may be considered as a point of full restraint in the plane of the bracing provided that both members are continuous and fixed together with at least one bolt.
  - $\lambda_2 = L_2/i_{yy}$ or less if the load $S_d$ in the supporting member (tension or compression) is lower than the load $N_d$ in the compression member (See EN 50341-1, J.6.3.3-2).

- Cross bracing with redundant members (Figure 11.5.d) (J.6.3.4)
  - $\lambda_1 = L_1/i_{vv}$ on the axis of minimum inertia about vv over buckling length $L_1$
  - $\lambda_2 = L_2/i_{yy}$ on the rectangular axis for buckling transverse to the cross bracing over buckling length $L_2$. This slenderness ratio may be reduced if $|S_d| < |N_d|$ (See Cross bracing)

$\lambda_3 = L_3/i_{yy}$ on the rectangular axis for buckling transverse to the cross bracing over the overall buckling length $L_3$.

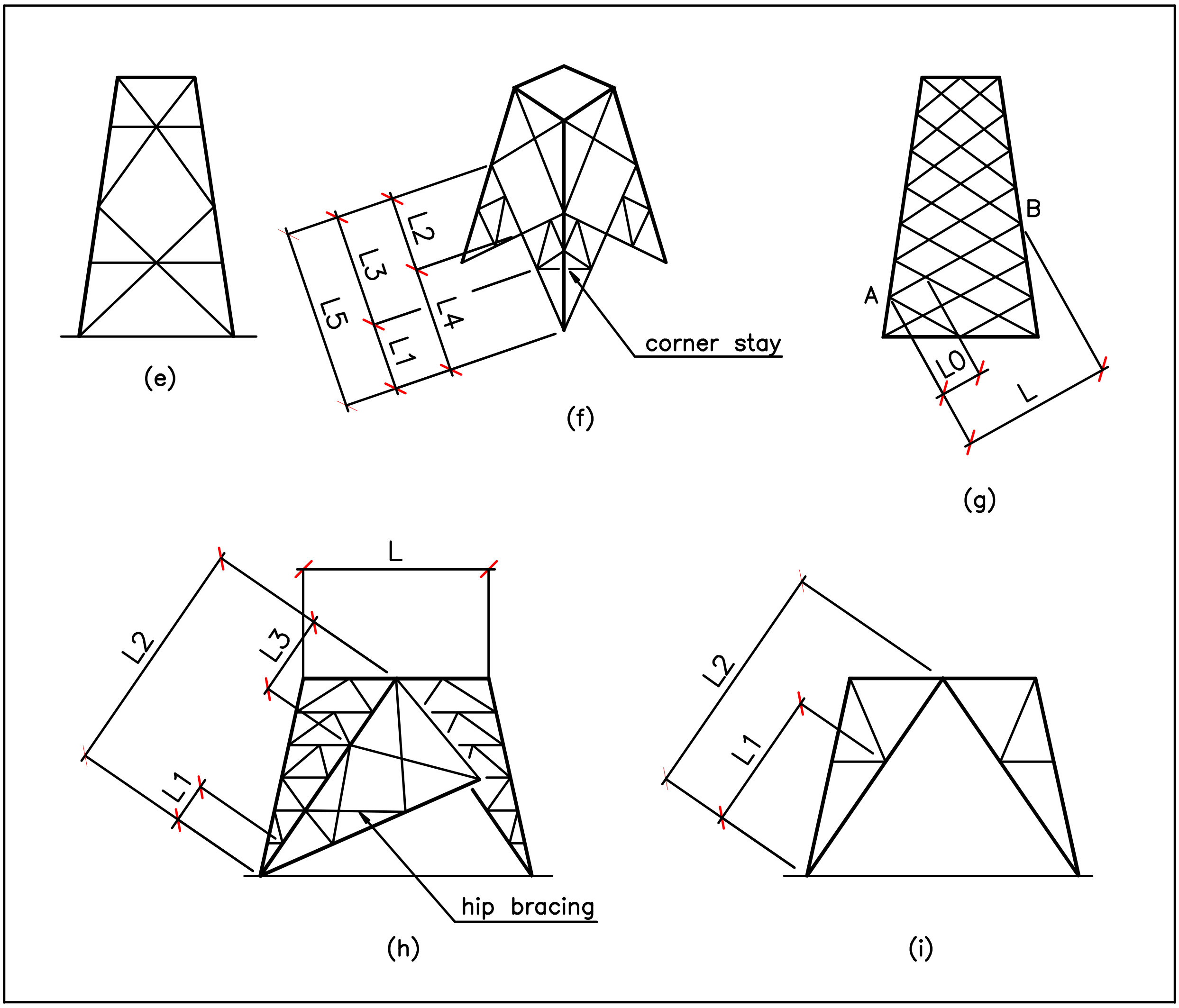



**Figure 11.15 – Typical bracing patterns**

- Cross bracing with a continuous horizontal member at centre intersection (Figure 11.6.e)
  $\lambda = L/i_{yy}$ on the rectangular axis for buckling in the transverse direction over its full length for the horizontal component of the algebraic sum of the loads in the two members of the cross bracing.

- Cross bracing with diagonal corner stays (Figure 11.6.f)
  $\lambda_1 = L_1/i_{vv}$ on the axis of minimum inertia of one member against maximum load over buckling length $L_1$
  $\lambda_2 = L_2/i_{yy}$ on the transverse rectangular axis of one member against maximum load over the buckling length $L_2$
  $\lambda_3 = L_3/i_{yy}$ on the transverse rectangular axis of 2 members in cross bracing against the algebraic sum of loads in cross bracing over the buckling length $L_3$
  $\lambda_4 = L_4/i_{yy}$ on the transverse rectangular axis of 2 members (one in each of 2 adjacent faces) against the algebraic sum of loads in the 2 members connected by the diagonal corner bracing over the buckling length $L_4$
  $\lambda_5 = L_5/i_{yy}$ on the transverse rectangular axis of 4 members (each member of cross bracing in 2 adjacent faces) against the algebraic sum of loads in all 4 members over the overall buckling length $L_5$

- Multiple lattice bracing (Figure 11.6.g)
  $\lambda_0 = L_0/i_{vv}$ on the axis of minimum inertia over buckling length $L_0$
  $\lambda = L/i_{vv}$ designed as redundant members (see Clause 12)
  For the stability of the panel $i_{xx}/i_{vv} > 1.25$ and $L/i_{xx} > 350 – 400$.

- K-bracing (Figure 11.6.h and i)
  $\lambda_1 = L_1/i_{vv}$ on the axis of minimum inertia of one member against maximum load over buckling length $L_1$
  $\lambda_2 = L_2/i_{yy}$ or $L_2/i_{zz}$ on the appropriate transverse rectangular axis of one member against maximum load over the buckling length $L_2$ if no hip bracing has been provided (Figure 11.6.i)
  $\lambda_3 = L_3/i_{yy}$ or $L_3/i_{zz}$ on the appropriate transverse rectangular axis of 2 members in cross bracing against the algebraic sum of loads in cross bracing over the buckling length $L_3$ where triangular hip bracing has been provided (Figure 11.6.h)

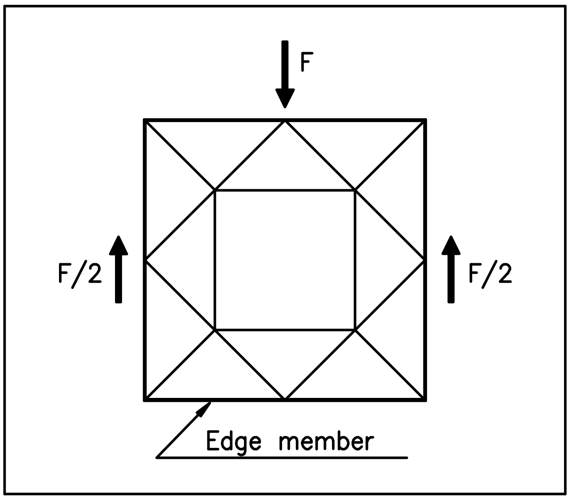



**Figure 11.16 – Typical plan bracing**

- Horizontal edge members with horizontal plan bracing (Figure 11.7)
  It is normal to provide a horizontal plan bracing to secure the tower against partial instability when the overall length L of the horizontal edge member becomes large, for instance if $L/i_{xx}$ or $L/i_{yy} > 250$.
  $\lambda_a = L_a/i$ where $L_a$ = distance between intersection points in the plan bracing for buckling transverse to the frame
  $\lambda_b = L_b/i$ where $L_b$ = distance between supports in the plan bracing for buckling in the plane of the frame
  The horizontal plan bracing needs to be stiff enough to prevent partial buckling. In case of doubt a good practice design rule is as follows:
  - The horizontal plan bracing, as indicated in Figure 11.7, has to resist a concentrated horizontal load F = 1,5 L, in kN, placed in the middle of the horizontal member, where: L = length of the horizontal edge member in m.
  - The deflection of the horizontal bracing under this load is limited to L / 1000.

**EN 50341-3: National Normative Aspects**

BE → EN 50341-3-2 – Clause J.5.1.1/BE.2 – Calculation according to ECCS 39 below.

Clause J.6.2.1/BE.2 – Legs and chords (Figure 11.4) – See ECCS 39 (4).

Clause J.6.3.2/BE.1 – Single lattice (Figure 11.5.a) - See ECCS 39 (7.1).

Clause J.6.3.2/BE.1 – Single lattice bracing with redundant members (Figure 11.5.b) - See ECCS 39 (7.2).

Clause J.6.3.3/BE.1 – Cross bracing (Figure 11.5.c) - See ECCS 39 (7.3).Clause J.6.3.4/BE.1 – Cross bracing with redundant members (Figure 11.5.d) - See ECCS 39 (7.4)

Clause J.6.3.5/BE.1 – Cross bracing with a continuous horizontal member at centre intersection (Figure 11.6.e) - See ECCS 39 (7.5).
The continuous horizontal must also been verified separately:

- with an axial force equal to 40% of the maximum compression force in one of both members crossing the horizontal at centre;
- with a force depending on the compression forces in both legs in conformity with the calculation of the redundant (See J.10/(4) BE.1).

Clause J.6.3.7.2.e/BE.1 – Cross bracing with diagonal corner stays (Figure 11.6.f) - See ECCS 39 (7.6) – The slenderness ratio of $L_5$ on transverse axis yy shall not exceed 250.

Clause J.6.3.6/BE.1 – Multiple lattice bracing (Figure 11.6.g) - See ECCS 39 (7.7)
The following check has also to be performed (in extension of the cross bracing): $\lambda_5 = L_5/i_{yy}$ on the transverse rectangular axis against the algebraic sum of the force $N_1$ in the member itself and the equivalent force obtained by taking the arithmetic mean of forces $N_2$, $N_3$, … in all members which cross the first member.

Clause J.6.3.8/BE.1 – K-bracing (Figure 11.6.h and i) - See ECCS 39 (7.8)
The stability of K-bracing must be assured by a horizontal plan bracing. The four faces have to be of equal rigidity. Staggered bracings to the legs are not allowed. When a corner stay is provided at the centre of $L_2$, the rule for $L_4$ in the cross bracing with diagonal corner stays (ECCS 39 Clause 7.6) is also applicable for $L_2$ of the K-bracing.

Clause J.7.1/BE.1 – Horizontal edge members with horizontal plan bracing
If the horizontal edge members of the K-bracing are a part of the primary members, they may not participate to the rigidity of the horizontal plan bracing. In this case the horizontal plan bracing is constituted of 3 interfering squares or of a pane with 2 diagonals. This horizontal plan bracing must be undeformable without taking into account the horizontal edge members.

DE → EN 50341-3-4
Clause J.6.2/DE.1 – Legs and chords (Figure 11.4)
$\lambda = 1.1\ L/i_{yy}$ - on the rectangular axis about yy for legs with staggered bracing (Figure 11.4.d), if:

- The member forces increase from top to bottom of the tower and
- The member length in the upper part of the tower or tower section are not longer than in the lower parts.

However if $L/i_{yy} < 80$, $\lambda = 1.0\ L/i_{yy}$.

Clause J.6.3.2/DE.1 - Single lattice (Figure 11.5.a)
The buckling length factor may be 0.9 if the requirements of Clause J.6.3.3/DE1 for cross bracing below are complied with.

Clause J.6.3.3/DE.1 – Cross bracing (Figure 11.5.c)
The buckling length factor may be 0.9 if:

- The ends of the members are fixed in each direction and
- They are sufficiently restrained in direction of buckling and
- Their cross-sectional area is smaller than that of the leg members.

Sufficient restraining is provided for example if the leg and bracing members consist of angle sections.

Clause J.6.3.4/DE.1 – Cross bracing with redundant members (Figure 11.5.c)
The buckling length factor for buckling rectangular to the plane of the bracing may be 0.9 if the requirements of Clause J.6.3.3/DE1 for cross bracing are complied with.

Clause J.6.3.8/DE.1 – K-bracing without redundant members (Figure 11.6.i)
The buckling length factor of $\lambda_1$ may be 0.9 if the requirements of Clause J.6.3.3/DE1 for cross bracing are complied with.

Clause J.6.3.8/DE.2 – K-bracing with redundant members (Figure 11.6.h)
The buckling length factor of $\lambda_2$ for buckling rectangular to the plane of the bracing may be 0.9 if:

- The requirements of Clause J.6.3.3/DE1 for the cross bracing are complied with;
- The redundant members support the member at least at its third points;
- The crossing point is restrained by a reinforcing panel not arranged in the plane of the face.

DK → EN 50341-3-5 – Clause J.5.1.b/DK.1 – For single lattice and cross bracing, where the bracing members are connected to the main members by two or more bolts, buckling lengths can be reduced by a buckling length factor 0,9, as to the lengths L and $L_1$. This rule is valid for sizes of the bracing less than the size of the main leg.

NL → EN 50341-3-15
Clause 7.3.5.3.a/NL.3 – Legs and chords (Figure 11.4) – EN 50341-1, J.6.2 above applies.

Clause 7.3.5.3.c/NL.3 – Cross bracing (Figure 11.5.c) – The buckling length shall be taken as indicated in EN 50341-1, J.6.3.3 above, provided that:

- The length of the supporting member ($L_{sup}$) should be less then 1,2 times the length of the supported member in compression ($L_{com}$);
- Equal cross-sections of members;
- Both members should be fixed together and has to restrain 20 % of the compression load in the member transverse to the plane of the bracing.

The slenderness should be taken as:

$\lambda_1 = L_1 / i_{vv}$ for angles;

$\lambda_2 = L_2 / i_{xx}$ or $L_2 / i_{yy}$ for angles,

where: $L_2 = K_b \cdot L_{com}$
$K_b$ = effective length factor for cross bracing

$L_{com}$ = the system length of the member in compression
$K_b$ = $\{0{,}75 - 0{,}38\ \alpha_1(1+1/K)\}F_{sup}/F_{com} + 0{,}25\ \alpha_1(1+1/K) + 1/2$
$K_b \leq 1{,}0$
$K_b \geq 0{,}25\ \alpha_1\ (1+1/K)$
$\alpha_1 = L_1 / L_{com}$
$K = 0{,}7 + 0{,}58\ \lambda_{com}$
$\lambda_{com} = L_{com} / i_{yy}$

Clause 7.3.5.3/NL.4 – Cross bracing with redundant members (Figure 11.5.d) – Where redundant members are inserted to stabilise the legs they also reduce the buckling length on the minimum axis of $L_1$. The slenderness ratio shall be taken as:

$\lambda_1 = L_1 / i_{vv}$ for angles;
$\lambda_3 = L_3 / i_{yy}$ or $L_3 / i_{zz}$ for angles,

where: $L_3 = K_b\, L_{com}$
and $K_b$ and $L_{com}$ are given above, see 7.3.5.3.c/NL.3.

Clause 7.3.5.3/NL.5 – Cross bracing with a continuous horizontal member at centre intersection (Figure 11.6.e)
For horizontal members which should provide restraints for the bracing members, the EN 50341-1, J.6.3.5 has to be regarded as informative requirements. Otherwise the horizontal member should be regarded only as redundant.

Clause 7.3.5.3/NL.7 – Cross bracing with diagonal corner stays (Figure 11.6.f) – The slenderness ratio shall be taken as follows:

$\lambda_1 = L_1 / i_{vv}$
$\lambda_5 = K_b\, L_5 / i_{xx}$ or $K_b\, L_5 / i_{yy}$

For $K_b$ see 7.3.5.3.c/NL.3 above.
Stability checks shall be carried out according to EN 50341, J.6.3.6 (2), which has to be regarded as informative.

Clause 7.3.5.3/NL.6 – Multiple lattice bracing (Figure 11.6.g) – The EN 50341-1, J.6.3.6 clause has to be regarded as informative requirements.

FI → EN 50341-3-17 – Clause J.6/FI.1 – The buckling length factors shall be taken from EN 1993-3-1.

SE → EN 50341-3-18 – Clause J.6.2.2/SE.1 – Legs and chords:

$\lambda = 1.10\ L/i_{yy}$ on the rectangular axis about yy for legs with staggered bracing (Figure 11.4.d) for self-supporting structures;
$\lambda = 1.25\ L/i_{yy}$ on the rectangular axis about yy for legs with staggered bracing (Figure 11.4.d) for guyed structures (See Clause 17).

## ECCS 39 Recommendations (Clauses 4 & 7)

The design slenderness ratio for the various types of bracings is given below.

- Legs and chords (Figure 11.4) (ECCS 39 - Clause 4)

  $\lambda = L/i_{vv}$ on the axis of the minimum inertia about vv for continuous legs and chords braced symmetrically in two normal planes of equal length and equal axial load (Figure 11.4.a and b) (More refined calculations can be carried out for different axial loads, buckling lengths L and moments of inertia i of two panel chords)

$\lambda = L/i_{yy}$ on the axis of the rectangular inertia about yy for continuous legs and chords braced with intermediate transverse support (Figure 11.4.c)

$\lambda = 1.2\ L/i_{yy}$ on the axis of the rectangular inertia about yy for continuous legs and chords braced with staggered bracing (Figure 11.4.d)

- Single lattice (Figure 11.5.a) (ECCS 39 - Clause 7.1)

  $\lambda = L/i_{vv}$ on the axis of the minimum inertia about vv

- Single lattice bracing with redundant members (Figure 11.5.b) (ECCS 39 - Clause 7.2)

  $\lambda_1 = L_1/i_{vv}$ on the axis of minimum inertia about vv over buckling length $L_1$

  $\lambda_2 = L_2/i_{yy}$ on the rectangular axis about yy over the overall buckling length $L_2$ with an effective buckling load approximated by taking the sum of ¾ the heavier load and ¼ the lighter load, when secondary loads are introduced to the bracing at the intermediate support.

- Cross bracing (Figure 11.5.c) (ECCS 39 - Clause 7.3)

  $\lambda_1 = L_1/i_{vv}$ on the axis of minimum inertia about vv over the length $L_1$ as the centre of the cross may be considered as a point of restraint both transverse to and in the plane of the bracing for the worst compressive load

  $\lambda_2 = L_2/i_{yy}$ with a radius of gyration about the rectangular axis parallel to the bracing plane over the overall buckling length $L_2$ when the load is not equally split into tension and compression. The member must be checked as above (single lattice with redundant members). Moreover the sum of the load carrying capacities of both members in compression must be at least equal to the algebraic sum of the loads in the two members.

  If both members are broken at the centre gusset, then the load carrying capacity is greatly reduced.

- Cross bracing with redundant members (Figure 11.5.d) (ECCS 39 - Clause 7.4)

  $\lambda_1 = L_1/i_{vv}$ on the axis of minimum inertia about vv over buckling length $L_1$

  $\lambda_2 = L_2/i_{yy}$ on the rectangular axis for buckling transverse to the cross bracing over buckling length $L_2$

  $\lambda_3 = L_3/i_{yy}$ on the rectangular axis for buckling transverse to the cross bracing over the overall buckling length $L_3$ for the algebraic sum of the loads (see cross bracing)

- Cross bracing with horizontal at centre intersection (Figure 11.6.e) (ECCS 39 - Clause 7.5)

  $\lambda = L/i_{yy}$ on the rectangular axis for buckling in the transverse direction over its full length for the horizontal component of the algebraic sum of the loads in the two members of the cross bracing, if the compression in one member exceeds the tension in the other member or if both members are in compression.

- Cross bracing with diagonal corner stays (Figure 11.6.f) (ECCS 39 - Clause 7.6)

  $\lambda_1 = L_1/i_{vv}$ on the axis of minimum inertia of one member against maximum load over buckling length $L_1$

  $\lambda_2 = L_2/i_{yy}$ on the transverse rectangular axis of one member against maximum load over the buckling length $L_2$

  $\lambda_3 = L_3/i_{yy}$ on the transverse rectangular axis of 2 members in cross bracing against the algebraic sum of loads in cross bracing over the buckling length $L_3$

  $\lambda_4 = L_4/i_{yy}$ on the transverse rectangular axis of 2 members (one in each of 2 adjacent faces) against the algebraic sum of loads in the 2 members connected by the diagonal corner bracing over the buckling length $L_4$

$\lambda_5 = L_5/i_{yy}$ on the transverse rectangular axis of 4 members (each member of cross bracing in 2 adjacent faces) against the algebraic sum of loads in all 4 members over the overall buckling length $L_5$

- Multiple lattice bracing (Figure 11.6.g) (ECCS 39 - Clause 7.7)
  $\lambda_0 = L_0/i_{vv}$ on the axis of minimum inertia over buckling length $L_0$
  $\lambda = L/i_{vv}$ designed as redundant members (see Clause 12)
  For the stability of the panel: $i_{xx}/i_{vv} > 1.25$ and $L/i_{xx} > 350 – 400$.

- K-bracing (Figure 11.6.h) (ECCS 39 - Clause 7.8)
  $\lambda = k\ L/i_{yy}$ where k varies with the ratio of the compression load $P_1$ in one half of the overall length L of the horizontal member and tension load $P_2$ in the other half, according to Table 11.2

**Table 11.2 – Effective length factor for horizontal of K-bracing**

| $P_2/P_1$ | 0 | 0.20 | 0.40 | 0.60 | 0.80 | 1.00 |
|---|---|---|---|---|---|---|
| k | 0.73 | 0.67 | 0.62 | 0.57 | 0.53 | 0.50 |

**Brazilian Industry Practice**

In accordance with ASCE 10-97

**Korean Industry Practice**

No recommendation.

**Finnish Industry Practice**

No additional recommendation

**Icelandic Industry Practice**

No additional recommendation

**Italian Industry Practice**

No additional recommendation

### 11.3 Slenderness ratio limits

See also Annex G

**ASCE 10-97 Standard**

| | |
|---|---|
| Leg members and chords | $L/i \leq 150$ |
| Primary bracing members | $L/i \leq 200$ |
| Redundant members | $L/i \leq 250$ |
| Tension-hanger members | $L/i \leq 375$ |
| Tension-only members | $300 \leq L/i \leq 500$ |
| Web-member – multiple lattices | Not specified |
| Horizontal edge members | Not specified |

**EN 50341-1:2001 - European CENELEC Standard (J.6.2 & J.6.3)**

| | |
|---|---|
| Leg members and chords | $L/i \leq 120$ (J.6.2.1) |
| Primary bracing members | $L/i \leq 200$ (J.6.3.1)) |
| Redundant members | $L/i \leq 240$ (J.6.3.1) |
| Tension hanger members | Not specified |
| Tension only members | Not specified |
| Web member – multiple lattice | $L/i \leq 350$ (on the whole cross bracing length) (J.6.3.4/6) |
| Horizontal edge members | $L/i \leq 250$ (without horizontal plan bracing) (J.7.2.5) |

**EN 50341-3: National Normative Aspects**

BE → EN 50341-3-2 – Clause J.6.2.1/BE.1 & BE.2; Clause J.6.3.1/BE.1

| | |
|---|---|
| Leg members and chords | $L/i \leq 150$ |
| Primary bracing members | $L/i \leq 200$ |
| Redundant members | $L/i \leq 200$ |
| Tension-hanger members | $L/i \leq 350$ |

If this tension member is linked to a secondary member used to reduce the slenderness of the horizontal edge members of the cross-arms, $L/i \leq 200$.

GB → EN 50341-3-9 – Clause 7.3/GB.1.b

| | |
|---|---|
| Legs and crossarms struts | $L/i \leq 120$ |
| Primary bracings members | $L/i \leq 200$ |
| Secondary bracings (redundant members) | $L/i \leq 250$ |
| Members carrying tension only | $L/i \leq 350$ |

Tension-only cross bracing systems should not be adopted on normal self-supporting lattice towers unless allowed by the Project Specifications.

SE → EN 50341-3-18 – Clause J.6.3.1/SE.1.1

| | |
|---|---|
| Redundant members | $L/i \leq 250$ |

**ECCS 39 Recommendations (4 & 7)**

| | |
|---|---|
| Leg members and chords | Not specified |
| Primary bracing members | Not specified |
| Redundant members | Not specified |
| Tension-hanger members | Not specified |
| Tension-only members | Not specified |

| | |
|---|---|
| Web-member – multiple lattice | L/i ≤ 350-400 |
| Horizontal edge members | Not specified |

**Brazilian Industry Practice**

| | |
|---|---|
| Leg members | L/i ≤ 150 |
| Other compressed members | L/i ≤ 200 |
| Redundant members | L/i ≤ 250 |
| Tension-hanger members | L/i ≤ 375 |
| Tension-only members | L/i ≤ 375 |

Cross bracing diagonals where the loads are equally or almost equally split into tension-compression system shall be dimensioned considering the centre of the cross as a point of restraint for both transverse to and in the plane of the bracing. Furthermore, the bracings shall have an effective maximum slenderness of 250 when considering the whole length.

**Figure 11.17 – Diagonals buckling**

**Korean Industry Practice**

| | |
|---|---|
| Leg members | L/i ≤ 200 |
| Other compressed members | L/i ≤ 220 |
| Redundant members | L/i ≤ 250 |
| Tension-hanger members | No recommendation |
| Tension-only members | No recommendation |
| Web-member – multiple lattice | No recommendation |
| Horizontal members | No recommendation |

**Finnish Industry Practice**

No additional recommendation.

**Icelandic Industry Practice**

No additional recommendation.

**Italian Industry Practice**

No additional recommendation

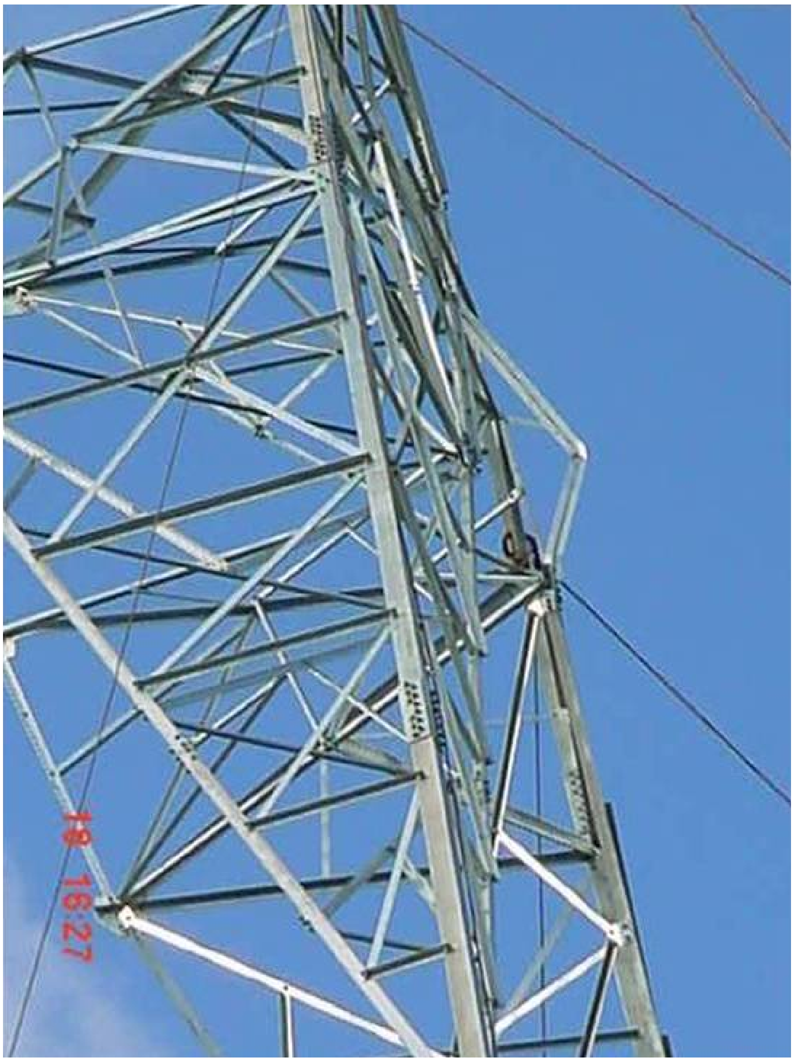


**Figure 11.18 – Double angle diagonal buckling**

## 12 Design of Redundant Members

Redundant (or secondary) members are installed on the overhead line lattice supports, basically for the function of bracing the loaded bars (leg members, chords, primary members). Even not having calculated loads, they must be rigid enough to guarantee efficient support against buckling. For this reason, it is recommended to attribute to the redundant members, hypothetical loads which magnitude are, usually, percentages of the loads of the supported members.

**ASCE 10-97 Standard**

The magnitude of the load in the redundant member can vary from 1.0 to 2.5% of the load in the supported member.

**EN 50341-1:2001 - European CENELEC Standard (J.10)**

The hypothetical load transverse to the main member being stabilised at each node of the attachment of the redundant member is equal to:

$$P = K \cdot N / 100$$

where $K = (\lambda+32)/60$ with $1 \leq K \leq 2$

N - axial force in the main member

$\lambda$ - slenderness ratio of the main member

The angle $\alpha$ between the redundant member and the supported main member has to be greater than 15º.

**EN 50341-3: National Normative Aspects**

BE → EN 50341-3-2 – Clause J.10/BE.1 – The redundant member shall also be checked for 2.5% of leg load shared equally between all the node points along the length of the leg in a panel, excluding the first and the last, all these loads acting together and in the same direction i.e. right angles to the leg and in the plane of the bracing.
In case of cranked K bracing with angle between diagonal and main leg close to 20°, secondary effects should be taken into consideration.

DE → EN 50341-3-4 – Clause J.10.1/DE.1 – The value K from EN 50341-1 for the determination of the hypothetical force shall be assumed as equal to 2.

GB → EN 50341-3-9 – Clause 7.3/GB.1.c – Redundant members supporting tower legs shall be designed to resist the hypothetical loads specified in ECCS 39 if the angle $\alpha$ between leg and main bracing is equal to or greater than 25°.
If the angle $\alpha$ is less than 25° the hypothetical loads from ECCS shall be multiplied by a factor of $0.42 / \sin \alpha$. The minimum allowable angle is 15°. As an alternative to the above procedure, a second order elastic analysis may be performed.

NL → EN 50341-3-15 – Clause 7.3.5.3/NL.8 – The design force for the redundant shall be taken as 1% of the (buckling) resistance of the supported member. This force can act as a compression force as well as a tension force.
Redundant members may be taken as supports, if the angle between the redundant members and compressed leg member or bracing member is more than 25°

measured in the plane of the bracing. If this condition is not satisfied the redundant members should be incorporated in the calculation model.

**ECCS 39 Recommendations (6)**

The magnitude of the hypothetical load transverse to the main member being stabilised at each node point of the attachment of the redundant member can vary from 1.0 to 2.5 % of the load N in the supported member and depends on the slenderness ratio $\lambda = L/i_{vv}$ of the supported member, according to Table 12.1.

The bracing shall also be checked for 2.5 % of the leg load shared equally between all the node points along the length of the leg, excluding the first and last, all these loads acting together and in the same direction.

**Table 12.1 – Variation of the hypothetical load in the redundant member – ECCS 39 Recommendations**

| $\lambda = L/i_{vv}$ | 0-40 | 50 | 60 | 70 | 80 | 90 | 100 |
|---|---|---|---|---|---|---|---|
| N (%) | 1.02 | 1.28 | 1.52 | 1.65 | 1.75 | 1.85 | 2.00 |

**Brazilian Industry Practice**

2.5% of the load in the supported member, irrespective of the number and inclination of the redundant members at the same joint.

**Korean Industry Practice**

1.0 or 2.0% of the load in the supported member

**Finnish Industry Practice**

No additional recommendation.

**Icelandic Industry Practice**

No additional recommendation

**Italian Industry Practice**

No additional recommendation

**Figure 12.1 – Main and redundant members**

## 13 Design of members acting only under tension

It is a current practice that some very long members be installed only under tension to provide better assembly and overall rigidity.

**ASCE 10-97 Standard**

Connections should be detailed with at least two bolts to make assembling easier.
Reductions in length shall be as specified per Table 13.1.

**Table 13.1 – ASCE 10-97 Tension members detailing**

| Standard ASCE 10-97 | |
|---|---|
| **Length (mm)** | **Reduction (mm)** |
| L ≤ 4600 | 3.2 |
| L > 4600 | 3.2 + 1.6 (*) |
| (*) for each additional of 3100 mm or fraction | |

**EN 50341-1:2001 - European CENELEC Standard**

Not specified

**EN 50341-3: National Normative Aspects**

GB → EN 50341-3-9 – Clause 7.3/GB.1.b – Tension-only cross bracing systems should not be adopted on normal self-supporting lattice towers unless allowed by the Project Specification.

**ECCS 39 Recommendations**

Not specified

**Brazilian Industry Practice**

Connections shall have at least 2 bolts, and members shall be detailed assuming reductions on their nominal lengths according to Table 13.2.

**Table 13.2 – Tension members detailing - Brazilian practices**

| **Length (mm)** | **Reduction (mm)** |
|---|---|
| L ≤ 4500 | 3.0 (1/8”) |
| L >4500 | 3.0(1/8”) +1.5(1/16”)(*) |
| (*) for each additional of 3000 or fraction | |

For each intermediate connection add 1.5 mm (1/16”) for each lap splice or 3 mm (1/8”) for butt splices.

**Korean Industry Practice**

No recommendation

**Finnish Industry Practice**

No recommendation

**Icelandic Industry Practice**

No recommendation

**Italian Industry Practice**

No recommendation

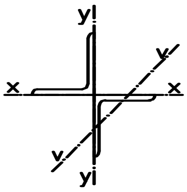
# 14 Design of double angles

**ASCE 10-97 Standard and ASCE Manual 52 – 1989**

Not specified

**EN 50341-1:2001 - European CENELEC Standard (J.6.4)**

Two, three or four angles in cruciform section or two back to back angle sections:



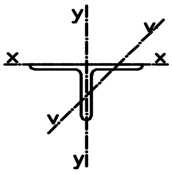



**Figure 14.1 – EN 50341-1 - Types of double angles detailing**

If welded continuously they may be taken as fully composite.
In case of a cruciform compound member, a minimum of 2 bolts per member are required at each batten plate.

The buckling capacity transverse to the principal axis zz is determined as a single compression member with a virtual slenderness of

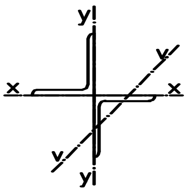
$$\lambda = \sqrt{\lambda_0^2 + \lambda_1^2 \times \frac{m}{2}}$$

$\lambda_0$ - slenderness ratio of the full members
$\lambda_1$ - slenderness ratio of one sub-member $c/i_{vv}$ $\lambda_1 \leq 50$
c - distance between batten plates
m – number of angles

**EN 50341-3: National Normative Aspects**

No additional specifications

**ECCS 39 Recommendations (4.1 & 8)**



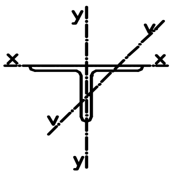



**Figure 14.2 – ECCS 1985 – Types of double angles detailing**

– Two angles in cruciform cross section welded or bolted together.

The slenderness ratio is evaluated according to the formula:

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

$\lambda_0$ - slenderness of the full member
$\lambda_1$ - slenderness of one sub-member = $c/i_{vv}$
c - distance between intermediate connection is based on a shear force of about 1-1.5% of the total buckling load. It is good practice to have $\lambda_0 > \lambda_1$ and $\lambda_1 < 40\text{-}50$.

– Two angles back to back forming a T.

The slenderness ratio is evaluated according to the formula:

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

$\lambda_0$ - slenderness of the full member about y-y

$\lambda_1$ - slenderness of one sub-member = $c/i_{vv}$

c - distance between intermediate connections. It is good practice to limit this value to a maximum $\lambda_1 \leq 90$ or $\lambda_1 \leq 0.75$ L/i whichever is the smaller.

**Figure 14.3 – Detailing using double angles**

**<u>Brazilian Industry Practice</u>**

$(kL/i)_{SP} \leq 3/4\ (kL/i)_{CP}$

where: k - effective length coefficient
L - unbraced length
i - radius of gyration
SP - single profile
CP - compound profile

For angles having leg widths greater than or equal to 100 mm (4”) connections shall be done with at least two bolts at each connection point.

**Korean Industry Practice**

No recommendation

**Finnish Industry Practice**

No additional recommendation

**Icelandic Industry Practice**

No additional recommendation

**Italian Industry Practice**

No additional recommendation

# 15 Design of members and step bolts for construction loads

This clause deals with the design of members which can be climbed, as well as step bolts at the main legs for erection and maintenance loads. Walkways, ladders, stirrup arrangements, platforms for climbing and access to working positions as well as crossarm ends are not considered hereafter.

The following items are discussed and summarised in Tables E and F:

- The Construction load (vertical or perpendicular to the member);
- Partial factor of the construction load;
- Maximum angle of inclination for the member;
- Additional (wind) loads considered (if any);
- Dimensions of step bolts (width, flat tread width, diameter of cylindrical tread, height of the lateral stop, normal spacing between two step bolts, maximum spacing, minimum spacing, maximum variation of spacing).

Loads due to erection and maintenance are sometimes summarised hereafter under the term construction loads.

**ASCE 10-97 Standard**

It is suggested that tower members that may be used for support by maintenance personnel when climbing a tower be capable of supporting a vertical load of 250 pounds (1.1 kN) applied independently of all other loads without permanent distortion of the member.

**EN 50341-1:2001 - European CENELEC Standard (4.2.6.2 or 4.3.6.2)**

The characteristic erection and maintenance load on cross-arms shall not be less than 1.0 kN acting together with the permanent loads and, as relevant, other imposed loads. These forces shall act at the individual most unfavourable node of the lower chords of one cross-arm face, and in all other cases in the axis of the cross-arms at the attachment point of the conductors.

For all members, which can be climbed and are inclined with an angle less than 30° to horizontal, a characteristic load of 1.0 kN acting vertically in the centre of the member shall be considered without any other loads. Additional requirements or precautions should be added in case pre-assembling on the ground takes place.

Steps shall be rated for a concentrated characteristic load of 1.0 kN acting vertically at a structurally unfavourable position. The partial factor for construction and maintenance loads $\gamma_P$ is 1.5 (Clause 4.2.11 of EN 50341-1). $\gamma_P$ may be amended in the National Normative Aspects or the Project Specification.

**EN 50341-3: National Normative Aspects**

AT → EN 50341-3-1 – Clause 4.3.6/AT.1 – Construction loads are to be assumed as perpendicularly acting on:

- In the middle of horizontal bracing of cross-arms (combined with normal loading conditions);
- In the middle of all horizontal bracings of the tower body (e.g. plan bracing, secondary bracing) without any additional loading.

BE → EN 50341-3-2 – Clause 4.2.6/BE.2 – For each arbitrary angle a maintenance load of 1.5 kN acting vertically in the centre of the angle is considered, combined with the own load of the angle and a reduced wind pressure distributed over the total angle length.

Clause 7.10.1/BE.1 – The partial factor $\gamma_P$ is 1.50 for new towers (and 1.25 for old towers). Additional safe access systems are stated in the Project Specifications.

DE → EN 50341-3-4 – Clause 4.3.6/DE.1 – For all members, which can be climbed and are inclined with an angle less than 30° to horizontal, a construction load of 1.0 kN acting vertically in the centre of the member shall be assumed, however, without any other loads. Step bolts shall be rated for a concentrated load of 1.0 kN acting vertically at a statically unfavourable position. For construction and maintenance loads the partial factor $\gamma_P$ = 1.5 applies.

Clause 7.10/DE.1 – The width of the step bolts shall amount to at least 150 mm in case of two line step bolt arrangements. Flat tread width shall be at least 20 mm, and the diameter of cylindrical treads at least 24 mm. To provide a protection against sliding a lateral stop at least 20 mm high measured from the top of step must be provided. Hexagon-head step bolts meet the requirement for an adequate lateral stop. Normally the step bolts shall be arranged with a constant distance of ≤ 333 mm. If due to the design of the tower the distances between alternate step bolts of an arrangement cannot be equal and/or cannot be 333 mm or less, the distance between two adjacent step bolts may vary by up to 100 mm but the spacing between step bolts or stirrups shall not exceed 403 mm. In the vicinity of the cross-arm joints structural components may be used as treads instead of step bolts.

DK → EN 50341-3-5 – Clause 4.2.11/DK.1 – The partial factor for construction and maintenance loads $\gamma_P$ = 1.5.

FI → EN 50341-3-7 – Clause 4.2.11/FI.1 – The partial factor for construction and maintenance loads $\gamma_P$ = 1.5.

FR → EN 50341-3-8 – Clause 4.3.6.2/FR.1 – A load of 1 kN acting vertically on the centre of the bar shall be taken into account for each bar that is likely to be used for climbing. The partial factor for construction and maintenance loads $\gamma_P$ = 1.0.

Clause 7.10.1/FR.1 – The devices used shall take into account peculiarities of the supports on which they are installed and shall be specified in the Project Specification.

GB → EN 50341-3-9 – Clause 4.2.6/GB.1 – Details of construction and maintenance loads shall be provided in the Project Specification.

Clause 4.2.11/GB.1 – Partial factor for construction and maintenance loads $\gamma_P$ = 1.5.

IE → EN 50341-3-11 – Clause 4.2.6.2/IE.1 – All members which can be climbed during construction and maintenance shall withstand a point load of 1.5 kN acting perpendicular to the member.

IT → EN 50341-3-13 – Clause 4.3.6/ IT.1 – Construction and maintenance loads shall not be considered

Clause 4.3.11/IT.1 – Partial factors for actions shall not be considered.

NL → EN 50341-3-15 – Clause 4.2.6/NL.1 – Values for maintenance loads on:

- Climbing facilities: 1.5 kN (vertical);

– Other parts of the structure used for climbing: 1.0 kN (vertical).

The different loads do not have to be combined.

Clause 4.2.11/NL.1 – The partial factor for construction and maintenance loads $\gamma_P$ = 1.5.

Clause A.2.1 – Step bolts must have a distance in between of at least 250 mm and 300 mm at the most. For each climbing facility the successive distances in between may not differ more than 15 mm.

Clause A.2.2 – The dimensions of step bolts must comply with the following requirements:

- the height of the raised edge must be at least 30 mm;
- the effective width of a step bolt must be at least 150 mm and of a step clip at least 300 mm;
- the diameter of a step bolt must be at least 20 mm;
- the free height above the step bolt must be at least 150 mm;
- the free space behind the step bolt must be at least 200 mm.

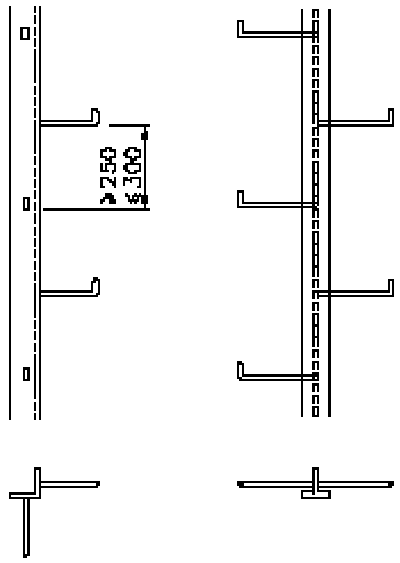



**Figure 15.1 – Step bolts distance**

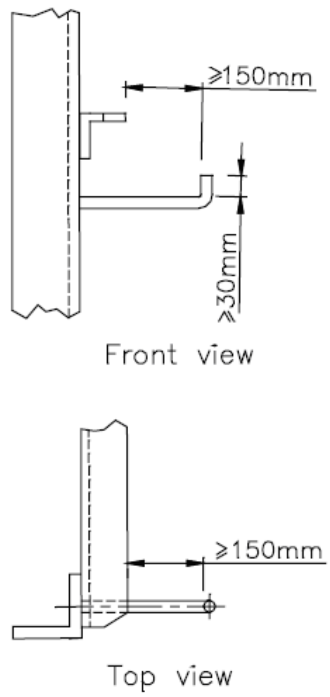



**Figure 15.2 – Minimum dimensions of step bolts**

NO → EN 50341-3-16 – Clause 4.2.6.2/NO.1 – Steps shall be rated for a concentrated ultimate load of 1.5 kN acting vertically at the most unfavourable position.

Clause 4.2.11/NO.1 – Partial factor for construction and maintenance loads $\gamma_P$ = 1.45 for good quality control of design and construction, else = 2.0.

SE→ EN 50341-3-18 – Clause 4.2.11/SE.1 – Partial factor for construction and maintenance loads $\gamma_P$ = 1.65.

CZ → EN 50341-3-19 – Clause 4.2.6.2/CZ.1 – For all members of structure which can be climbed and are inclined with an angle less than 30° to the horizontal, a minimum characteristic load of 1.0 kN, multiplied by a partial factor for an action $\gamma_P$ = 1.5, acting vertically in the centre of the member shall be assumed. The Project Specification may require a higher load.
Steps of any kind shall be rated for a concentrated characteristic load of 1.0 kN, multiplied by a partial factor for an action $\gamma_P$ = 1.5, acting vertically at a structurally unfavourable position. The Project Specification may require a higher load (e.g. for certain types of steps only).

SI → EN 50341-3-21 – Clause 4.3.6/SI.1 – For all walkable components, which are inclined at an angle less than 45° to the horizontal, a vertical force of 1 kN shall be assumed in the centre of the component. Thereby all other forces acting on the component should be neglected.
Step irons should be verified with regard to the statically most unfavourable position of the concentrated force of 1 kN.
When considering the assembling load, the partial safety factor $\gamma_P$ = 1.5 should be adopted.

Clause 7.10/SI.1 – The step width of climbing wedges shall be at least 150 mm for dual-hand paths. The thickness of stairs shall be at least 20 mm, the diameter of

round stairs minimum 24 mm. Antiskid protection shall be provided by a 20 mm high edge over the stair's upper edge. Climbing wedges have a hexagonal head to fulfil such requirements.
The standard distance between climbing wedges shall be constant within the range of ≤ 333 mm. If the tower construction does not allow constant distances between climbing wedges of ≤ 333 mm or less then two consecutive distances between climbing wedges may not differ more than 100 mm and no distance between adjacent wedges may not exceed 403 mm. Where cross-arms are fixed to tower body, certain elements of tower structure may be used for stepper posts instead of climbing wedges.

PL → EN 50341-3-22 – Clause 7.10.1/PL.1 – Lattice towers shall be equipped with access steps with maximum spacing of 400 mm.

**<u>Recommendations For Angles in Lattice Transmission Towers – ECCS 1985</u>**
Not specified

**<u>Brazilian Industry Practice</u>**
Those members should withstand a vertical load of 1 kN irrespective of any other calculated loads, and with no permanent deformation

**<u>Korean Industry Practice</u>**
No recommendation

**<u>Finnish Industry Practice</u>**
No additional recommendation

**<u>Icelandic Industry Practice</u>**
No additional recommendation

**<u>Italian Industry Practice</u>**
No additional recommendation

**Figure 15.3 – Members reinforced to withstand vertical loads**

## Part 2.3 – Particular design practices for lattice steel towers

## 16 Design of members subjected to bending and axial force

**ASCE 10-97 Standard**

Members subject to both bending and axial tension shall be to satisfy the following formula:

$$(P / P_a) + (M_x / M_{ax}) + (M_y / M_{ay}) \leq 1$$

where:

$P$ - axial tension
$P_a$ - allowable axial tension according to item 9
$M_x$ , $M_y$ - moments about x- and y-axis, respectively
$M_{ax}$ , $M_{ay}$ - allowable moments about the x- and y-axis, respectively, as defined in previous section.

$$M_{ax} = W_x f_y$$
$$M_{ay} = W_y f_y$$

where:

$W_x$ , $W_y$ - x- and y-axis section modulus, respectively
$f_y$ - yield strength

Members subjected to bending and axial compression shall be designed to satisfy the following equations:

$$(P / P_a) + C_m (M_x / M_{ax}) [1 / (1 - P / P_{ex})] + C_m (M_y / M_{ay}) [1 / (1 - P / P_{ey})] \leq 1$$

$$(P / P_y) + (M_x / M_{ax}) + (M_y / M_{ay}) \leq 1$$

where:

$P$ - Axial compression
$P_a$ - Allowable axial compression according to item 10

$$P_{ex} = \pi^2 E I_x / ( k_x L_x)^2$$
$$P_{ey} = \pi^2 E I_y / ( k_y L_y)^2$$

$I_x$ - moment of inertia about x-axis
$I_y$ - moment of inertia about y-axis
$k_x L_x$ , $k_y L_y$ - the effective lengths in the corresponding planes of bending
$M_x$, $M_y$ - the moment about the x- and y-axes respectively, see Notes below;
$M_{ax}$, $M_{ay}$ - the allowable moments about the x- and y-axis respectively, see explanation below

$C_m = 0.6 - 0.4\ M_1/M_2$ for restrained members with no lateral displacements of one end

relative to the other, and with no transverse loads in the plane of bending (linear diagram of moments).
$M_1/M_2$ is the ratio between the smaller and the larger end moments in plane of bending. $M_1/M_2$ is positive when bending is in reverse curvature (Fig. 16.1a) and negative when it is in single curvature (Fig. 16.1b).

$C_m$ = 1.0 for members with unrestrained ends, and with no transverse loads between supports

$C_m$ = 0.85 if the ends are restrained and there are transverse loads between supports

– Laterally Supported Beams:

$M_{ax} = W_x f_y$
$M_{ay} = W_y f_y$

where:

$W_x$ , $W_y$ - x- and y-axis section modulus respectively based on the gross section or on the reduced section (when applicable)
$f_y$ - yield strength

– Laterally Unsupported Beams:
Verify lateral buckling according to paragraphs 4.14.4 and 4.14.8 of the ASCE Manual 52 – 1989 or 3.14.4 and 3.14.8 of Standard ASCE 10-97

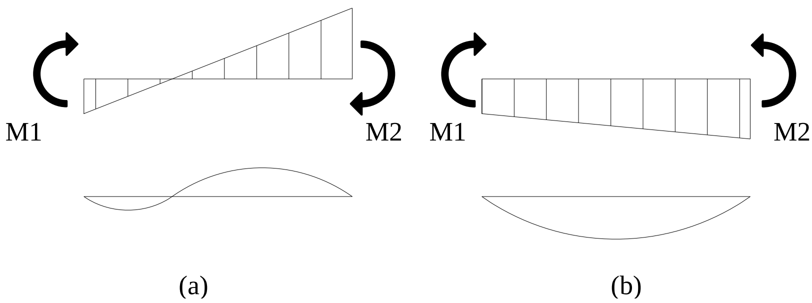



**Figure 16.1 – Lateral buckling moments**

Notes:

* $M_x$ and $M_y$ are determined as below:

a) If there are transverse loads between points of support (in the plane of bending):
$M_x$ and $M_y$ in the first equation above are the maximum moments between these points, which in the second are the larger of moments at these points;

b) If there are no transverse loads between points of support (in the plane of bending):
$M_x$ and $M_y$ in both equations above are the larger of the values of $M_x$ and $M_y$ at these points.

**EN 50341-1:2001 - European CENELEC Standard (J.4.4)**

$$(N_{sd} / (A_{eff} \cdot f_{yd})) + (M_{ysd} /(W_{effy} \cdot f_{yd})) + ( M_{zsd} / (W_{effz} \cdot f_{yd})) \leq 1$$

where:

$A_{eff}$ - effective area of the cross-section when subject to uniform tension (See Item 10.2)

$W_{eff}$ - effective section modulus consistent with $A_{eff}$ (see Item 10.2)

$f_{yd} = f_y / \gamma_{M1}$

$f_{yd}$ - design yield strength

$\gamma_{M1}$ - partial factor for resistance of members to buckling

$\gamma_{M1} = 1.10$ (Clause 7.3.5.1.1 of EN 50341-1)

**EN 50341-3: National Normative Aspects**

NL →EN 50341-3-15 – Clause 7.3.5.3/NL.2 – The resistance of cross sections of members for bending and axial force shall be determined in accordance with the requirements of Clause 6.2.9 of ENV 1993-1-1.

**ECCS 39 Recommendations**

Not specified

**Brazilian Industry Practice**

In accordance with ASCE 10-97

**Korean Industry Practice**

In accordance with ASCE 10-97

**Finnish Industry Practice**

No additional recommendation

**Icelandic Industry Practice**

No additional recommendation

**Italian Industry Practice**

No additional recommendation

# 17 Design of guyed lattice steel structures

A guyed support can be a lattice steel structure or a pole of tubular steel, concrete or timber with guys of galvanized extra high strength steel wire strands.

In this Clause only guyed lattice steel structures are considered.

## 17.1 General

**ASCE 10-97 Standard**

The requirements for design guyed steel structures are complemented by ASCE Manuals and Reports on Engineering Practice nº 91 – Design of Guyed Electrical Transmission Structures. Guyed structures are commonly used to support electric transmission lines. They generally have the advantage of lightweight, erection ease, pre-assembly, and simple foundation design. There is a considerable range of applications, from simple guyed wood poles to the very large guyed steel latticed structures. This publication describes the various types of guyed structures that have been used; presents typical guys and fittings; illustrates guy anchors and foundations; explores analysis and design techniques specific to guyed structures; discusses unique construction and maintenance problems; and displays both hand and computer calculations to illustrate some of the concepts discussed in the document.

**EN 50341-1:2001 - European CENELEC Standard (7.2.7 & 7.7.1)**

The guy material properties, including the characteristic strength, shall be taken from the relevant standards. The characteristic strength of the guy fittings and insulators shall be at least that of the guy itself.

The requirements of parent Eurocodes shall be complied with, except where otherwise specified in the EN 50341-1 Standard.

**EN 50341-3 - National Normative Aspects**

BE → EN 50341-3-2 – Clause 7.7/BE.1 & BE.2 – Guyed structures are not authorised except for provisional works. Provisional guys must be earthed or equipped with an inaccessible insulator.

SI → EN 50341-3-21 – Clause 7.7/SI.1 – The use of guyed structures is allowed only for temporary solutions.

PL → EN 50341-3-22 – Clause 7.7/PL.1 – Guyed structures shall be designed in accordance with PN-EN 1993-1-1:2006 and PN-EN 1993-3-1:2006 and PN-EN 1993-1-11:2006.

**ECCS 39 Recommendations**

Not specified

**Brazilian Industry Practice**

Guyed structures shall be designed in accordance with Brazilian Standard NBR 8850.

**Korean Industry Practice**

No recommendation

**Finnish Industry Practice**

No additional recommendation

**Icelandic Industry Practice**
No additional recommendation

**Italian Industry Practice**
No additional recommendation

## 17.2 Material

**ASCE 10-97 Standard**
Guy materials, guy fittings and tension devices shall comply with requirements given in Chapter 3 of ASCE practice nº 91.

**EN 50341-1:2001 - European CENELEC Standard (7.7.3)**
Materials shall comply with requirements given in Clause 6.2 of EN 50341-1 and documents relative to parent single supports.

**EN 50341-3 - National Normative Aspects**

DE → EN 50341-3-4 – Clause 7.7/DE.2 – Galvanized steel ropes according to DIN 3051, Part 4, (round-shaped flexible stranded ropes with steel core only) and to DIN 48201, Part 3, shall be used as guy wires. Ropes with thickly galvanized strands should be used preferably. Steel ropes with any other type of corrosion protection may be used if that protection is at least as effective as the specified galvanizing.

SE → EN 50341-3-18 – Clause 7.7.3/SE.1 – The guy steel wire strands shall be in accordance with the Swedish Standards SS 424-08-06. The minimum breaking strength of the guy shall be 30 kN.

PL → EN 50341-3-22 – Clause 7.7/PL.1 – Guyed structures shall be designed in accordance with PN-EN 1993-1-1:2006 and PN-EN 1993-3-1:2006 and PN-EN 1993-1-11:2006.

**ECCS 39 Recommendations**
Not specified

**Brazilian Industry Practice**
Materials shall comply with requirements given in Chapter 5 of Brazilian Standard NBR 8850.

**Korean Industry Practice**
No recommendation

**Finnish Industry Practice**
No additional recommendation

**Icelandic Industry Practice**
No additional recommendation

**Italian Industry Practice**
No additional recommendation

## 17.3 Basic design requirements

**ASCE 10-97 Standard**

Flexible self-supporting structures and guyed structures normally required a second-order analysis. Guyed structures and latticed H-frames may include masts built-up with angles at the corners and lacing in the faces. The overall cross-section of the mast is either square, rectangular, or triangular. Latticed masts typically include a very large number of members and are relatively slender, that is, may be susceptible to second-order stresses. One alternative to modeling a mast as a three-dimensional truss system is to represent it by a model made up of one or several equivalent beams. The properties of equivalent beam that deflects under shear and moment can be worked out from structure analysis principles. The beams are connected to form a three-dimensional model of the mast or entire structure. That model may be analyzed with any three-dimensional finite element computer program. If large deflections are expected, a second order (geometrically nonlinear) analysis should be used. Once the axial loads, shears, and moments are determined in each equivalent beam, they can be converted into axial loads in the members that make up the masts.

**EN 50341-1:2001 - European CENELEC Standard (7.7.5)**

The guyed structure shall generally be analysed using the second order theory. The analysis shall be based on the assumption that the stress-strain behaviour of the material is linear.

A latticed column (leg or cross-arm) shall be analysed for bending and buckling using a 3-dimensional beam model where:

- the axial and bending stiffness shall be calculated from the main member properties;
- while the torsional stiffness shall be derived from the bracing member properties.

Local buckling of main legs and bracing members shall be taken into account.

Shear force distribution shall be taken into consideration when calculating member forces at both ends of a hinged latticed column. To consider imperfections in the column an additional force acting transverse to the column may be added.

An initial out of straightness shall be assumed for sections hinged at both ends (tower legs). A normal design value is L/600 for steel sections, where L is the length of the leg.

The partial factor $\gamma_{M2}$ shall be taken as specified in the parent support, and in addition a partial factor for the resistance of guys to ultimate strength: $\gamma_{M2} = 1.60$.

**EN 50341-3 - National Normative Aspects**

DE → EN 50341-3-4 – Clause 7.7/DE.2 – The partial factor for the resistance of guys to ultimate strength: $\gamma_{M2} = 1.65$.

FI → EN 50341-3-7 – Clause 7.7.5.3/FI.1 – In single-guyed portal supports (one guy level with four guys only) the linear elastic analysis can be applied when calculating the forces in the main components (legs, cross-arms and guys) of the support.

Clause 7.7.5.3/FI.2 – In multi-guyed steel supports the analysis model shall take into account also the large displacements and changing locations of the load points. Due account shall be allowed for checking the slacking guys in each load case concerned.

Clause 7.7.5.3/FI.3 – A separate verification for the global stability is not needed, if the analysis is made by using an incremental large displacement FEM-analysis, where the effects of the 2$^{nd}$ order geometric nonlinearity (PD-effect) of compressed beam elements are taken into account in the stiffness of the elements.

Clause 7.7.6/FI.2 – The maximum slenderness of structural elements in guyed supports is listed below.

| | |
|---|---|
| Lattice steel leg | 120 |
| Tubular steel leg | 200 |
| Wooden leg | 250 |
| Horizontal beam between legs | 250 |
| Tension members in cross-arm | 350 |

Clause 7.7.6/FI.3 – The characteristic resistance of the guy material shall not exceed 1.6 kN/mm$^2$. In the attachment of the guys wedge clamps or other relevant fittings based on reliable and documented type tests shall be used. However, rope clamps are not accepted.

Clause 7.7.5.1/FI.1 – The partial factor $\gamma_{M2}$ for guys and their fittings to the ultimate strength shall be taken as follows:

| | |
|---|---|
| Guys and their fittings | $\gamma_{M2} = 1.40$ |
| Strain insulators | $\gamma_{M2} = 2.00$ |

The strength of the guying set (guy with fittings) shall not be less than 90 % of the strength of the guy.

IS → EN 50341-3-12 – Clause 7.7.5.1/IS.1 – The partial material factor for guys shall be 1.50 or as defined in Project Specification

NL → EN 50341-3-15 – Clause 7.7/NL.1 – The partial factor for the resistance of guys to ultimate strength: $\gamma_{M2} = 1.60$.

NO →EN 50341-3-16 – Clause 7.7/NO.1 – The partial factor $\gamma_{M2}$ for the resistance of guys to ultimate strength shall be 1.50 unless otherwise stated in the Project Specification.

SE → EN 50341-3-18 – Clause 7.7.5.2/SE.1 – For a lattice column in both ends hinged the additional shear force is 1.7 % of the axial force in the column.

Clause 7.7.5.1/SE.1 – The partial factor $\gamma_{M2}$ for the resistance of guys to ultimate strength is:

$\gamma_{M2} = 1.40$ for tangent supports;

$\gamma_{M2} = 1.55$ for permanent loaded guys, e.g. for angle and terminal supports.

**ECCS 39 Recommendations**

Not specified

**Brazilian Industry Practice**

According Brazilian Standard NBR 8850 and ASCE 10-97.

**Korean Industry Practice**

No recommendation

**Finnish Industry Practice**

No additional recommendation

**Icelandic Industry Practice**

No additional recommendation

**Italian Industry Practice**

No additional recommendation

### 17.4 Design practices for guys

**ASCE 10-97 Standard**

The design tension in guys shall not exceed 0.65 times the specified minimum breaking strength of the cable.

The elastic limit of wire rope is approximately 65% of its strength; therefore, the specified 0.65 times the minimum breaking strength is consistent with the standards ultimate stress Fy for tension members. The factor of safety of a guy with respect to its elastic limit is equal to the load factor, as with other structural components.

The determination of the tension of guys must be based on the movement of the guy anchor under load, the length and size of the guy, the allowable deflection of the structure, and the modulus of the guy. On tangent structures, pretensioning of guys to 10% of their rated breaking strength is normally sufficient to avoid a slack guy.

**EN 50341-1:2001 - European CENELEC Standard (7.7.6)**

The guys used in structures are generally pre-tensioned to a specified value in order to reduce the deformation at extreme loads. The possible pre-tensioning of the guys shall be checked and maintained during periodical inspections. For a multi-level guyed support, instructions for the erection work are needed because the structure is sensitive to the pre-tensioning of the guys. In order to minimize the possibility of guy vibrations the pre-tension should be less than 10 % of the breaking load of the guy.

The guys shall be equipped with devices for retightening. The connection between the guy rope and the anchor device shall be accessible. The connections and tightening devices shall be secured against loosening in service. Also a slackened or loose condition caused by the wind, maintenance or other event shall be considered. Due care shall be taken for protection of the guy in populated areas for possible galvanic corrosion and possible flashover. In some cases, insulation of the guy can be recommendable.

## EN 50341-3 - National Normative Aspects

DE → EN 50341-3-4 – Clause 7.7/DE.2 – Guys shall be equipped with devices for retightening. The connection of the stay ropes with the anchor device shall be accessible. The jointing elements shall be secured against unintentional loosening. For all steel towers the stays shall be bonded to the earthing system of the support.

FI → EN 50341-3-7 – Clause 7.7.5.3/FI.3 – The characteristic resistance of the guy material shall not exceed 1.6 kN/mm$^2$. In the attachment of the guys wedge clamps or other relevant fittings based on reliable and documented type tests shall be used. However, rope clamps are not accepted.

SI → EN 50341-3-21 – Clause 7.7/SI.1 – The use of guyed structures is allowed only for temporary solutions.

PL → EN 50341-3-22 – Clause 7.7/PL.1 – Guyed structures shall be designed in accordance with PN-EN 1993-1-1:2006 and PN-EN 1993-3-1:2006 and PN-EN 1993-1-11:2006.

## ECCS 39 Recommendations

Not specified

## Brazilian Industry Practice

According to the Brazilian Standard NBR 8850, the design tension in guys shall not exceed 0.75 times the specified minimum breaking strength of the cable multiplied by r "strength factor". r = 0.93.

On the suspension structures is recommended a pretensioning of guys to 10% of their rated breaking strength.

## Korean Industry Practice

No recommendation

## Finnish Industry Practice

No additional recommendation

## Icelandic Industry Practice

No additional recommendation

## Italian Industry Practice

No additional recommendation

# 18 Design of stub angles and anchor bolts

Stubs angles are the structural elements currently used to connect towers legs to concrete foundations. Depending on the solutions adopted for the foundations, anchor bolts are used as well.

## 18.1 Design of stub angles

**ASCE 10-97 Standard**

The stub angle area shall be checked for a combination of tension and shear or compression and shear, as follows:

$$A_a = (P / f_y) + (V / 0.75\ f_y)$$

where

$A_a$ - gross area of stub angle, or net area, if there is a hole at the intersecting plane;
P - tensile or compressive load on the angle
V - shear load perpendicular to the angle
$f_y$ - yield strength of stub angle steel

The compression stress ($f_{ca}$) in shear connectors (clip angles) is:

$f_{ca} = 1.19\ f_c$ where
$f_c$ - compressive strength of the concrete

The capacity of angles shear connectors shall be determined by the following:

$$P = f_{ca}\ b\ (t + r + x/2)$$

$$x = t\ (f_y/f_{ca})^{1/2} \leq w - r - t$$

where

b - length of angle shear connector
t - thickness of angle shear connector
r - radius of fillet
w - width of angle shear connector

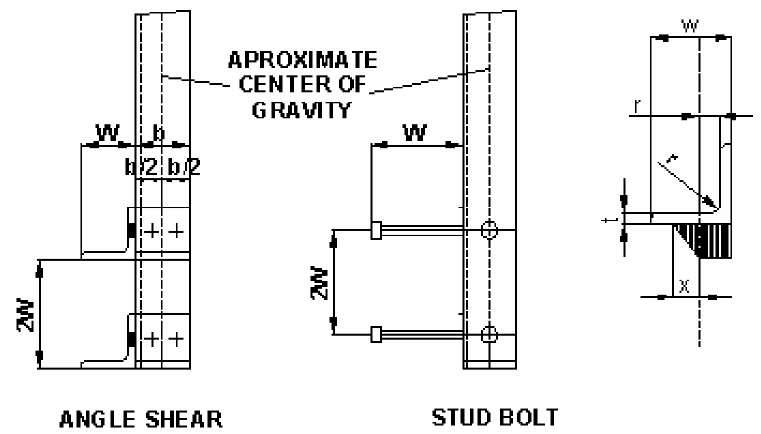



**Figure 18.1 – Stub angles design**

The minimum centre-to-centre spacing of the shear connectors shall be two times the width of the angle leg or two times the length of stud bolt as shown in Figure 18.1.

**EN 50341-1:2001 - European CENELEC Standard (7.3.6.4)**

Steel angle stub members with anchoring attachments such as angle cleats or studs shall be checked for the required shear capacity under the compression loads between the attachment and the concrete.

No bending moment in cleats or studs shall be considered.

**EN 50341-3 - National Normative Aspects**

DE → EN 50341-3-4 – Clause 8.7/DE.2.2 – If the total tensile and compression load of steel members anchored in concrete is transferred to the concrete by anchor cleats, anchor plates, lugs or the like then it shall be proved that the compression stresses between the anchoring elements and the concrete do not exceed the values given in Table 18.1, and the shearing stress in the contour surface of the anchoring elements does not exceed the values in Table 18.1. If these values are exceeded the resistance against splitting tensile forces shall be proved.
The shortest envelope of the anchoring elements shall be assumed for the contour surface. An embedment of structural steel elements without anchoring elements is not permitted.

**Table 18.1 – Ultimate stresses for anchoring stubs in concrete – German NNA**

| Strength quality class of concrete | Ultimate shearing stress $MN/m^2$ | Ultimate compression stress $MN/m^2$ |
|---|---|---|
| C 20/25 | 2.6 | 14.0 |
| C 25/30 | 2.7 | 17.5 |
| C 30/37 | 3.4 | 21.0 |

GB → EN 50341-3-9 – Clause 8.7/GB.1 – Where stub angles, which are cast in concrete to form part of the foundation, shall be fitted with angle cleats sufficient to transfer the whole of the uplift load into the concrete. In the case of compression loads, the cleats should be designed to take at least 50% of the loads, with the balance taken in bond between the stub and the concrete. The concrete cover to stubs and cleats shall not be less than 100 mm.

**ECCS 39 Recommendations**

Not specified

**Brazilian Industry Practice**

The cross sectional area is determined in accordance with ASCE10-97. The stub member shall be as the same size of the leg member or larger.

The bearing stress $f_{ca}$ on the shear connectors angles are determined in accordance with ASCE 10-97 (the use of stud bolts is not an usual practice). The shear connectors are considered as transferring both compression and tension loads to the concrete. Alternatively, the leg load may be considered transferred by the shearing surface between the concrete and the steel angle. In this case, the length of stub should be calculated as prescribed by the Brazilian concrete standard NBR 6118. For additional security a minimum of at least four (4) shear connections should be installed. Design shearing ($f_{bd}$) is given by:

$$f_{bd} = \eta_3 \; 0{,}21.(f_{ck})^{2/3} \; /1{,}4$$

where:

$f_{ck}$ - characteristic compression strength of the concrete

$\eta_3 = 1.0$ if $d < 32$ mm
$\eta_3 = (132\text{-}d) / 100$ if $d > 32$ mm

d - nominal diameter of the bolt

For the minimum centre-to-centre spacing of the shear connectors, ASCE 19-97 is adopted (see Figure 18.1). As an alternative, Figure 18.2 shows a widely used shear angles connectors detailing in Brazil.

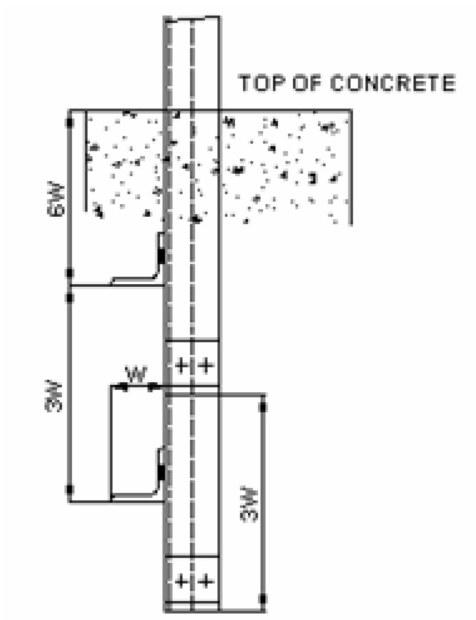



**Figure 18.2 – Stub Angle detailing – Brazilian practice**

It is recommended a maximum distance of 6W between the upper shear angle and the top of concrete surface, with a minimum value of about 500 mm.
The loading capacity of the shear angle is determined according to ASCE 10-97.

## Korean Industry Practice

The stub angle shall be of the same size of the leg member. Use of anchoring angles.

## Finnish Industry Practice

No additional recommendation

## Icelandic Industry Practice

No additional recommendation

## Italian Industry Practice

No additional recommendation.

## 18.2 Design of anchor bolts with base plates

**ASCE 10-97 Standard**

When the anchor bolt bases are subjected to tension and shear loads, the area of the bolt ($A_s$) is to be determined by

$$A_s = T / f_y + V / (\mu\ 0.85\ f_y)$$

where

- $A_s$ - thread section in accordance with the formulas of item 8.2.2
- T - tension load on the anchor bolt
- V - shear load perpendicular to the anchor bolt
- $f_y$ - yield strength of the anchor bolt
- $\mu$ - coefficient of friction

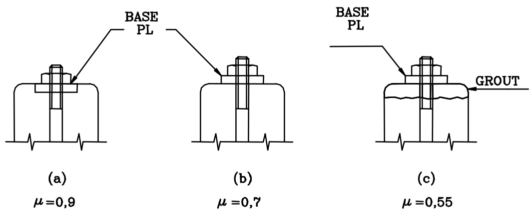



**Figure 18.3 – Anchor bolts settlement**

Values for $\mu$ are:

- a - 0.9 for concrete or grout against as-rolled steel with the contact plane a full plate thickness below the concrete surface
- b - 0.7 for concrete or grout placed against as-rolled steel with contact plane coincidental with the concrete surface
- c - 0.55 for grouted conditions with the contact plane between grout and as-rolled steel above the concrete surface.

When the anchor bolt bases are subjected to shear load or a combination of compression and shear loads, the area of steel ($A_s$) is determined by:

$$A_s = (V - 0.3\ D) / (\ \mu\ 0.85\ f_y)$$

where: D - compression load.

When shear lugs are attached to the base assembly to transfer the shear to the concrete, the area of the anchor bolt does not need to be checked by previous methods.

**EN 50341-1:2001 – European CENELEC Standard (K.6)**

Holding-down bolts in the base plates should be checked for shear, axial load as well as potential bending moments due to lateral displacement of the bolts.

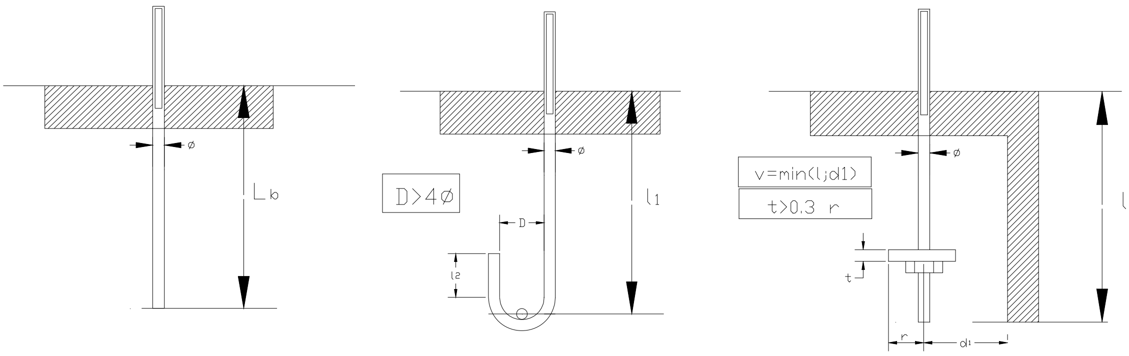



**Figure 18.4 – Anchor bolts detailing**

| Straight anchor | Anchor with bend | Anchor with plate |
|---|---|---|
| $F_{a,Rd} = \pi \Phi L_b f_{bd}$ | $F_{a,Rd} = \pi \Phi L_b f_{bd}$<br>$L_b = ( l_1 + 3.2\ D + 3.5\ l_2)$ | $F_{a,Rd} = \pi \Phi L_b f_{bd}$<br>$L_b = 2.45\Phi \frac{f_{cd}}{f_{bd}} (\frac{r^2}{\Phi^2} - 0.25)(1 - \frac{r}{v}) + l_0$ |

where: $f_{bd}$ - bonding stress of steel into concrete

with: $f_{bd} = \dfrac{0.36\sqrt{f_{ck}}}{\gamma_c}$ for plain bars

and $f_{bd} = \dfrac{2.25 f_{ctk0.05}}{\gamma_c}$ for bent bars

with: $f_{ctk0.05} = 0.7 f_{ctm}$ and $f_{ctm} = 0.3 f_{ck}^{2/3}$

where $f_{ck}$ - characteristic strength of concrete in compression
$f_{ctm}$ - average strength of the concrete in tension
$f_{ctk0.05}$ - characteristic strength of concrete in tension
$\gamma_c$ - partial factor on bonding = 1.50

The anchoring length shall be such that:

$$F_{a,Rd} = \pi \Phi L_b f_{bd} \geq F_{t,Sd}$$

where $F_{t,Sd}$ - design tension force per bolt for the ultimate limit state

The size of the bolt shall be such that:

$$F_{t,Sd} \leq F_{t,Rd} = 0.9\ f_{ub}\ A_s / \gamma_{Mb}$$

where: $f_{ub}$ - ultimate tensile strength of holding-down bolt
$A_s$ - tensile stress area of holding-down bolt
$\gamma_{Mb}$ - partial factor on resistance of holding-down bolt (see 8.2.1)

**EN 50341-3 - National Normative Aspects**

DK → EN 50341-3-5 – Clause 7.2.4/DK.1 – Straight undeformed anchors are not allowed for round bars without bends or plates.

Clause 7.3.6.4/DK.1 – For design of anchor lengths, reference is made to EN 1992-1-1. Holding-down bolts are checked for shear, axial load as well as possible bending moments due to lateral forces at the bolts.

NL →EN 50341-3-15 – Clause 7.3.6.4/NL.1 – The tensile or compression resistance of anchors embedded in concrete should be checked according to ENV 1992-1-1, 5.2.3. For the resistance of the holding down bolt it is referred to ENV 1993-1-1, 6.5.5.

SE → EN 50341-3-18 – Clause K.6/SE.1 – Alteration of $\gamma$ in Table K.2

$\gamma_c = \gamma_{MC} \cdot \gamma_n$

$\gamma_{MC}$ for concrete shall be taken from EN 1992-1-1.

$\gamma_{Mb} = 1.32$ in accordance with 7.3.6.1/SE.1.

The tension and compression forces shall be combined with shear and bending forces in the bolt, depending on detail design.

**ECCS 39 Recommendations**

Not specified

**Brazilian Industry Practice**

The anchor bolt is checked for tension load according to Brazilian standard NBR 8800:

– On yielding of gross area:

$$A_g = ( 0.90 \cdot S_{dt} )/ f_y$$

– On rupture of net area:

$$A_s = (0.75 \cdot S_{dt} )/ f_u$$

Additionally, anchor bolts should be checked for tension and shear load combinations:

$$S_{dt}/f_y + S_{dv}/ (\mu \cdot 0.85 \cdot f_y) \leq f_r A_r$$

$A_g$ - gross section as shown in the Table 18.2 here below;
$A_s$ - thread section as shown in the same Table 18.2;
$A_r$ - root section at the thread as shown in the same Table 18.2;
$S_{dt}$ - design tensile load on the anchor bolt;
$S_{dv}$ - design shear load perpendicular to the anchor bolt;
$f_y$ - yield strength of anchor bolt;
$f_u$ - tensile strength of anchor bolt;
$f_r$ - strength factor (< 1.0);
$\mu$ - coefficient of friction as in accordance with ASCE 10-97.

**Table 18.2 – Anchor bolt stress areas – Brazilian practices**

| Stress Areas ($A_s$) and Stress Root Areas ($A_r$) | | | | | | | |
|---|---|---|---|---|---|---|---|
| **d** | n | $A_s$ ($cm^2$) | $A_r$ ($cm^2$) | **d** | P (mm) | $A_s$ ($cm^2$) | $A_r$ ($cm^2$) |
| **1/2”** | 13 | 0,915 | 0,811 | **M12** | 1,75 | 0,843 | 0,743 |
| **5/8”** | 11 | 1,458 | 1,302 | **M14** | 2,00 | 1,154 | 1,021 |
| **3/4”** | 10 | 2,158 | 1,948 | **M16** | 2,00 | 1,567 | 1,410 |
| **7/8”** | 9 | 2,979 | 2,704 | **M20** | 2,50 | 2,448 | 2,204 |
| **1”** | 8 | 3,908 | 3,554 | **M24** | 3,00 | 3,525 | 3,173 |
| **1 1/8”** | 7 | 4,924 | 4,470 | **M27** | 3,00 | 4,594 | 4,191 |
| **1 1/4”** | 7 | 6,252 | 5,739 | **M30** | 3,50 | 5,606 | 5,087 |
| **1 3/8”** | 6 | 7,451 | 6,799 | **M33** | 3,50 | 6,936 | 6,357 |
| **1 1/2”** | 6 | 9,066 | 8,345 | **M36** | 4,00 | 8,167 | 7,451 |
| **1 3/4”** | 5 | 12,255 | 11,249 | **M42** | 4,50 | 11,209 | 10,264 |
| **2”** | 4.5 | 16,118 | 14,836 | **M48** | 5,00 | 14,731 | 13,526 |
| **2 1/2”** | 4 | 25,799 | 23,970 | **M52** | 5,00 | 17,578 | 16,260 |
| **3”** | 4 | 38,499 | 36,258 | **M56** | 5,50 | 20,300 | 18,742 |
| | | | | **M64** | 6,00 | 26,760 | 24,807 |

where $A_s$ and $A_r$ are calculated as follows:

– For metric bolts:

$$A_s = (\pi/4) \cdot (d - 0.9382 \cdot p)^2 \cdot 0.01$$

$$A_r = (\pi /4) \cdot (d - 0.9743 / n)^2 \cdot 6.4516$$

– For imperial bolts:

$$A_s = (\pi /4) \cdot (d - 1.3 \cdot p)^2 \cdot 0.01$$

$$A_r = (\pi /4) \cdot (d - 1.3 / n)^2 \cdot 6.4516$$

where:

d - bolt diameter in inch or mm as appropriate;
n - number of threads per inch;
p - pitch of thread in mm as appropriate.

The anchor bolt length should be calculated as prescribed in the Brazilian concrete standard NBR 6118. When using plain bars, hooks should be provide to reduce the required friction length according to NBR 6118.

**Korean Industry Practice**

No recommendation

**Finnish Industry Practice**

No additional recommendation for the anchoring length.

The tension forces shall be combined with shear, and the capacity shall be checked with the following formula:

$$\left(\frac{T}{f_y A_s}\right)^{\frac{4}{3}} + \left(\frac{V}{f_y A_s}\right)^{\frac{4}{3}} \leq 1$$

Where (As) is the area of the bolt.

**Icelandic Industry Practice**

No additional recommendation

**Italian Industry Practice**

No additional recommendation

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## Annexes

## Annex A – List of National Normative Aspects (NNAs) to EN 50341-1

| | Code | Country | Standard |
|---|---|---|---|
| | ASCE | - | ASCE 10-97 |
| | EN | - | EN 50341-1 |
| | ECCS | - | ECCS 39 |
| 1 | AT | Austria | EN 50341-3-1 |
| 2 | BE | Belgium | EN 50341-3-2 |
| 3 | CH | Switzerland | EN 50341-3-3 |
| 4 | DE | Germany | EN 50341-3-4 |
| 5 | DK | Denmark | EN 50341-3-5 |
| 6 | ES | Spain | EN 50341-3-6 |
| 7 | FI | Finland | EN 50341-3-7 |
| 8 | FR | France | EN 50341-3-8 |
| 9 | GB | Great Britain and North Ireland | EN 50341-3-9 |
| 10 | GR | Greece | EN 50341-3-10 |
| 11 | IE | Ireland | EN 50341-3-11 |
| 12 | IS | Iceland | EN 50341-3-12 |
| 13 | IT | Italy | EN 50341-3-13 |
| 14 | LU | Luxembourg (not available) | EN 50341-3-14 |
| 15 | NL | Netherlands | EN 50341-3-15 |
| 16 | NO | Norway | EN 50341-3-16 |
| 17 | PT | Portugal | EN 50341-3-17 |
| 18 | SE | Sweden | EN 50341-3-18 |
| 19 | CZ | Czech Republic | EN 50341-3-19 |
| 20 | EE | Estonia | EN 50341-3-20 |
| 21 | SI | Slovenia | EN 50341-3-21 |
| 22 | PL | Poland | EN 50341-3-22 |

## Annex B – Standards considering detailing and design practices

| | | Standards | | | | | Practices per country |
|---|---|---|---|---|---|---|---|
| Lattice tower | | ASCE | ASTM | EN 50341-1 + NNAs | Euro-code | ECCS | BR, KO, FI, IS, IT |
| Detailing | General | - | x | - | - | - | x |
| | Material | (x) | x | x | x | - | x |
| | Design of bolts | x | - (1) | x | x | x | x (4) |
| Design | Steel structures | x | - | x | x | - | |
| | Lattice towers | x | - | x | - | x | |
| | Particular design (2) | x | - | x | - | x (3) | |

(1) except shear and tension bolt
(2) combined axial load and bending, stubs, anchor bolts
(3) except combined axial load and bending
(4) except if NNA is available; for those items FI, IS and IT refer to EN 50341-1

## Annex C – Correspondence between TB and Standard clauses

| Part TB | Clause TB | ASCE | ASTM | EN 1993-1-1 | EN 50341-1 | EN 50341-3 Number of NNAs | ECCS 39 |
|---|---|---|---|---|---|---|---|
| 1 | 1 | - | - | - | - | - | - |
| | 2.1 | 3.3 | - | 1.1.2 | - | 5 | - |
| | 2.2 | 3.3 | - | - | - | - | 5.2.1 |
| | 2.3 | - | - | - | - | - | - |
| | 2.4.1 | 7.3 | - | - | - | - | - |
| | 2.4.2 | 7.3 | - | - | - | - | - |
| | 2.5 | - | - | - | 7.2.3 & 7.9 | 8 | - |
| | 3.1 | - | A394 | - | - | 4 | - |
| | 3.2 | - | A394 | - | 7.2.1 | 1 | - |
| | 3.3 | - | - | - | - | 4 | - |
| | 3.4 | - | A394 | - | - | - | - |
| | 3.5 | 4.1 | - | - | - | 2 | - |
| | 3.6 | - | A394 | - | - | 1 | - |
| | 3.7 | - | - | - | - | - | - |
| | 3.8.1 | - | A394 | - | - | 9 | - |
| | 3.8.2 | - | A394 | - | - | - | - |
| | 3.8.3 | 5.1.2 | - | - | - | 3 | - |
| | 3.8.4 | 5.1.2 | - | - | - | 4 | - |
| | 3.8.5 | 5.1.2 | - | - | - | 3 | - |
| | 3.8.6 | 5.1.2 | - | - | - | - | - |
| | 3.9 | - | - | - | - | 5 | - |
| | 3.10 | - | - | - | - | - | - |
| | 4.1 | - | A563 | - | 7.2.1 | - | - |
| | 4.2 | - | A563 | - | 7.2.1 | - | - |
| | 5.1 | - | - | - | 7.2.1 | - | - |
| | 5.2 | - | - | - | - | - | - |
| | 6.1 | 3.2 | - | | 7.2.3 | - | 3 |
| | 6.2 | - | A36 | 3.2.1 | 7.2.1 | 7 | 2 |
| | 6.3 | - | A36 | 3.2.5 | - | - | - |
| | 7.1 | 5.2.4 | - | - | - | - | - |
| | 7.2 | - | - | - | - | - | - |
| 2.1 | 8.1.1 | 4.5 | - | - | J.11 | - | 10 |
| | 8.1.2 | 4.5.1 | - | - | J.11.2 | 11 | 10.4 |
| | 8.1.3 | 4.5.3 | - | - | J.11.2 | 4 | 10.3 |
| | 8.1.4 | 4.5.3 | - | - | J.11.2 | 4 | 10.5 |
| | 8.2.1 | - | A394 | - | J.11.1 | 10 | 10.1 |
| | 8.2.2 | - | A394 | - | J.11.3 | 2 | - |
| | 8.2.3 | 4.3.4 | - | - | - | - | - |
| | 8.2.4 | 4.4 | - | - | J.11.2 | 4 | 10.2 |
| | 9 | 3.10 | - | 6.2.2.2 | J.4.1 | 3 | 9 |
| | 10.1 | 3.6 | - | 6.3.1.2 | J.5.1.1 | 12 | 3 |
| | 10.2 | 3.7 | - | - | J.2.3 & J.5.1.2 | 5 | 3 |
| 2.2 | 11 | - | - | - | 7.3 | 9 | - |
| | 11.1 | 3.7.3 | - | - | J.8 & J.9 | 8 | 5.4 |
| | 11.2 | 3.7.4 | - | - | J.6.2 & J.6.3 | 6 | 4 & 7 |
| | 11.3 | 3.7.4 | - | - | J.6.2 & J.6.3 | 3 | 4 & 7 |

| | | | | | | | |
|---|---|---|---|---|---|---|---|
| | 12 | 3.7.4 | - | - | J.10 | 4 | 6 |
| | 13 | 3.10 | - | - | - | 1 | - |
| | 14 | - | - | - | J.6.4 | - | 4.1 & 8 |
| | 15 | - | - | - | 4.2.6.2 or 4.3.6.2 | 14 | - |
| 2.3 | 16 | 3.12 | - | 6.2.9 | J.4.4 | 1 | - |
| | 17.1 | - | - | - | 7.2.7 & 7.7 | 3 | - |
| | 17.2 | - | - | - | 7.7.3 | 3 | - |
| | 17.3 | - | - | - | 7.7.5 | 6 | - |
| | 17.4 | - | - | - | 7.7.6 | 4 | - |
| | 18.1 | 7.2.3 | - | - | 7.3.6.4 | 2 | - |
| | 18.2 | 7.2.4 | - | - | K.6 | 3 | - |
| Total | 57 | 42 | | 6 | 31 | 178 | 17 |

# Annex D – Values of thickness, partial factor, distance to hole

| | Code | Thickness of angle leg | | | | Partial factor mat. | | | Distances holes | | |
|---|---|---|---|---|---|---|---|---|---|---|---|
| | | $t_p$ | $t_s$ | $t_h$ | $t_{pun}$ | $\gamma_{M1}$ | $\gamma_{M2}$ | $\gamma_{Mb}$ | s/d | e/d | f/d |
| | | open prim brac | opens ec brac | hol sect | pun-ching | tens bend buck | at bolt hole | Bolt conn | hole-hole | long dist end | perp dist edge |
| | | mm | mm | mm | mm | - | - | - | - | - | - |
| | ASCE | - | - | - | - | - | - | - | - | - | - |
| | EN | - | - | - | - | 1.10 | 1.25 | 1.25 | - | - | - |
| | EC3 | - | - | - | - | 1.10 | 1.25 | 1.25 | - | - | - |
| 1 | AT | 4 | - | 3.5 | - | 1.10 | - | 1.25 | 2.5 | 1.5 | 1.25 |
| 2 | BE | - | - | - | - | 1.00 | 1.25 | 1.25 | - | - | - |
| 3 | CH | - | - | - | - | - | - | - | - | - | - |
| 4 | DE | 4 | - | 3 | 12 | 1.10 | 1.25 | 1.25 | 2.5 | ~1.5 | 1.20 |
| 5 | DK | - | - | - | - | - | - | - | 2.5 | 1.75 | 1.30 |
| 6 | ES | - | - | - | - | - | - | - | - | - | - |
| 7 | FI | - | - | - | - | - | - | - | - | - | - |
| 8 | FR | - | - | - | - | - | - | - | - | - | - |
| 9 | GB | - | - | - | - | - | - | - | - | - | - |
| 10 | GR | - | - | - | - | - | - | - | - | - | - |
| 11 | IE | - | - | - | - | - | - | - | - | - | - |
| 12 | IS | - | - | - | - | - | - | - | - | - | - |
| 13 | IT | - | - | - | - | - | - | - | - | - | - |
| 14 | LU | - | - | - | - | - | - | - | - | - | - |
| 15 | NL | - | - | - | - | 1.00 | 1.25 | 1.25 | - | - | - |
| 16 | NO | 5 | 4 | 4 | 12 | 1.10 | 1.25 | 1.25 | - | - | - |
| 17 | PT | - | - | - | - | - | - | - | - | - | - |
| 18 | SE | 4 | 3 | 3 | 13 | 1.15 | 1.32 | 1.32 | - | - | - |
| 19 | CZ | - | - | - | - | 1.15 | 1.30 | 1.45 | - | - | - |
| 20 | EE | - | - | - | - | - | - | - | - | - | - |
| 21 | SI | 4 | - | 3 | 12 | 1.00 | 1.25 | 1.25 | 2.5 | ~1.5 | 1.20 |
| 22 | PL | - | - | - | - | - | - | - | - | - | - |

t - Minimum thickness of members ($t_p$ - primary members; $t_s$ – secondary or redundant members; $t_h$ - hollow sections; $t_{pun}$ – max. thickness for punching) (Clauses 2.1 & 3.9);
$\gamma_M$ – Partial factor for resistance of material ($\gamma_{M1}$ – tension, bending or buckling; $\gamma_{M2}$ – net cross section at bolt holes; $\gamma_{Mb}$ – bolted connections) (Clauses 10.1, 8.1.2 & 8.2.1);
s – Spacing between two holes (Clause 8.1);
e – Longitudinal distance of the hole to the end of the member;
f – Perpendicular distance of the hole to the end of the member;
d – Bolt diameter

# Annex E – Construction loads for members

| | Code | $\gamma_P$ | P | β | Wind | V/P | Comments |
|---|---|---|---|---|---|---|---|
| | | - | kN | ° | | | |
| | ASCE | - | 1.1 | - | no | vert | |
| | EN | 1.5 | 1.0 | 30° | no | vert | |
| | EC 3 | - | - | - | - | - | |
| 1 | AT | - | - | - | norm | perp | Horizontal bracing of crossarms |
| | | | | | no | perp | Hor. brac of tower body& bolt steps |
| 2 | BE | 1.5 | 1.5 | - | red | vert | |
| 3 | CH | - | - | - | - | - | |
| 4 | DE | 1.5 | 1.0 | 30° | no | vert | |
| 5 | DK | 1.5 | - | - | - | - | |
| 6 | ES | - | - | - | - | - | |
| 7 | FI | 1.5 | - | - | - | - | |
| 8 | FR | 1.0 | 1.0 | - | - | - | |
| 9 | GB | 1.5 | PS | - | - | - | |
| 10 | GR | - | - | - | - | - | |
| 11 | IE | - | 1.5 | - | - | perp | |
| 12 | IS | - | - | - | - | - | |
| 13 | IT | - | No | - | - | - | |
| 14 | LU | - | - | - | - | - | |
| 15 | NL | 1.5 | 1.0 | - | no | vert | |
| 16 | NO | - | - | - | - | - | |
| 17 | PT | - | - | - | - | - | |
| 18 | SE | 1.65 | - | - | - | - | |
| 19 | CZ | 1.5 | 1.0 | 30° | - | vert | |
| 20 | EE | 1.5 | - | - | - | - | |
| 21 | SI | 1.5 | 1.0 | 45° | no | vert | |
| 22 | PL | - | - | - | - | - | |

$\gamma_P$ – Partial factor for construction and maintenance load (Clause 15)
P – Construction load in the center of members (kN)
β – Maximum angle of inclined member to consider with a construction load (°)
Wind – If construction load is combined with reduced/normal wind load on structure
V/P – Direction of the construction load: vertical or perpendicular to the member
Comments – if any

## Annex F – Construction loads for step bolts

| | Code | $\gamma_P$ | P | Width step bolt | Flat tread width | Diam cyl tread | Height lateral stop | Norm spa-cing | Max spa-cing | Min spa cing | Max vari-ation |
|---|---|---|---|---|---|---|---|---|---|---|---|
| | | | kN | mm | mm | mm | mm | mm | mm | mm | mm |
| | ASCE | | 1.1 | | | | | | | | |
| | EN | 1.5 | 1.0 | | | | | | | | |
| | EC 3 | | | | | | | | | | |
| 1 | AT | | PS | | | | | | | | |
| 2 | BE | | PS | | | | | | | | |
| 3 | CH | | | | | | | | | | |
| 4 | DE | 1.5 | 1.0 | 150 | 20 | 24 | 20 | 333 | 403 | | 100 |
| 5 | DK | | | | | | | | | | |
| 6 | ES | | | | | | | | | | |
| 7 | FI | | | | | | | | | | |
| 8 | FR | | | | | | | | | | |
| 9 | GB | | | | | | | | | | |
| 10 | GR | | | | | | | | | | |
| 11 | IE | | | | | | | | | | |
| 12 | IS | | | | | | | | | | |
| 13 | IT | | | | | | | | | | |
| 14 | LU | | | | | | | | | | |
| 15 | NL | 1.5 | 1.5 | 150 | 20 | 20 | 30 | | 300 | 250 | 15 |
| 16 | NO | 1.45 | 1.5 | | | | | | | | |
| 17 | PT | | | | | | | | | | |
| 18 | SE | 1.65 | | | | | | | | | |
| 19 | CZ | 1.5 | 1.0 | | | | | | | | |
| 20 | EE | 1.5 | | | | | | | | | |
| 21 | SI | 1.5 | 1.0 | 150 | 20 | 24 | 20 | 333 | 403 | | 100 |
| 22 | PL | | | | | | | | 400 | | |

$\gamma_P$ – Partial factor for construction and maintenance load (Clause 15)
P – Vertical construction load for bolt steps (kN)

## Annex G – Limits of slenderness ratios for legs and bracings

| | Code | Legs & Chord | Prim bracing mem-bers | Second bracing Redun dants | Tension hangers | Tension only | Web multiple bracing | Horiz edge |
|---|---|---|---|---|---|---|---|---|
| | ASCE | 150 | 200 | 250 | 375 | 500 | - | - |
| | EN | 120 | 200 | 240 | - | - | 350 | 250 |
| | EC3 | - | - | - | - | - | - | - |
| | ECCS | - | - | - | - | - | 400 | - |
| 1 | AT | - | - | - | - | - | - | - |
| 2 | BE | 150 | 200 | 200 | 350 | - | - | - |
| 3 | CH | - | - | - | - | - | - | - |
| 4 | DE | - | - | - | - | - | - | - |
| 5 | DK | - | - | - | - | - | - | - |
| 6 | ES | - | - | - | - | - | - | - |
| 7 | FI | - | - | - | - | - | - | - |
| 8 | FR | - | - | - | - | - | - | - |
| 9 | GB | 120 | 200 | 250 | - | 350 | - | - |
| 10 | GR | - | - | - | - | - | - | - |
| 11 | IE | - | - | - | - | - | - | - |
| 12 | IS | - | - | - | - | - | - | - |
| 13 | IT | - | - | - | - | - | - | - |
| 14 | LU | - | - | - | - | - | - | - |
| 15 | NL | - | - | - | - | - | - | - |
| 16 | NO | - | - | - | - | - | - | - |
| 17 | PT | - | - | - | - | - | - | - |
| 18 | SE | - | - | 250 | - | - | - | - |
| 19 | CZ | - | - | - | - | - | - | - |
| 20 | EE | - | - | - | - | - | - | - |
| 21 | SI | - | - | - | - | - | - | - |
| 22 | PL | - | - | - | - | - | - | - |
| See Clause 11.3 | | | | | | | | |

## Annex H – List of symbols

### List of symbols for ASCE, ASTM, EN and ECCS for Clauses 1 to 15

| Sym-bol | Signification | Item | ASCE | EN | ECCS |
|---|---|---|---|---|---|
| $A$ | Cross section area of bolt; gross cross section area of bolt in shear plane | 8.2.1 | - | x | x |
| $A_e$ | Effective cross section area for eccentric loads | 9 | x | - | x |
| $A_{eff}$ | Effective cross section area for uniform compression of members (with $b = b_{eff}$) | 10.1 | - | x | - |
| $A_g$ | Gross cross section area | 9 | x | x | x |
| $A_n$ | Net cross section area at holes | 9 | x | x | x |
| $A_s$ | Tensile stress area of bolt; net tensile area of bolt | 8.2.1 | x | x | - |
| $b$ | Nominal width of the leg | 10.2 | x | x | x |
| $b_{eff}$ | Effective width of the leg for uniform compression | 10.2 | x | - | - |
| $b_1$ | Width of connected leg | 9 | - | x | - |
| $b_2$ | Width of free leg | 9 | - | x | - |
| $c$ | Distance between batten plates | 14.2 | - | x | x |
| $d$ | Bolt diameter | 8.1.2 | x | x | x |
| $d_0$ | Hole diameter | 8.1.2 | x | x | x |
| $E$ | Modulus of elasticity | 10.1 | x | x | x |
| $e$ | Distance of the hole to the end or the cut edge | 8.1.3 | x | x | x |
| $e_1$ | End distance from centre of hole to adjacent end in angle | 8.2.4 | - | x | - |
| $e_2$ | Edge distance from centre of hole to adjacent edge in angle | 8.2.4 | - | x | - |
| $F_{b,Rd}$ | Bearing resistance per bolt | 8.2.4 | - | x | - |
| $F_{t,Rd}$ | Tension resistance per bolt | 8.2.2 | - | x | - |
| $F_{v,Rd}$ | Shear resistance per shear plane | 8.2.1 | - | x | - |
| $f$ | Distance between hole and edge (axial and transversal) | 8.1.4 | x | x | x |
| $f_a$ | Allowable compression stress | 10.1 | x | - | x |
| $f_{cr}$ | Yield strength modified for local and torsional buckling | 10.2 | x | - | - |
| $f_{cv}$ | Computed shear stress on effective area (thread or body) | 8.2.3 | x | - | - |
| $f_p$ | Ultimate bearing stress | 8.1.3 | x | - | x |
| $f_t$ | Tensile stress on net area under tension only | 8.2.3 | x | x | x |
| $f_t(v)$ | Tensile stress when bolts are subject to combined shear and tension | 8.2.3 | x | - | - |
| $f_u$ | Ultimate tensile strength of plate or bolt | 3.2 | x | x | x |
| $f_v$ | Allowable shear stress | 3.2 | x | - | - |
| $f_v$ | Design shear stress under tension only | 8.2.3 | x | - | - |
| $f_y$ | Yield strength | 8.1.4 | x | - | x |
| $f_{yd}$ | Design yield strength | 16 | - | x | - |
| $g$ | Distance between adjacent holes in the transversal direction | 9 | x | x | - |
| $i$ | Radius of gyration about the relevant buckling axis | 10.1 | x | - | x |
| $i_{vv}$ | Minimum radius of gyration | 10.2 | x | x | x |
| $k$ | Effective buckling length coefficient; buckling length factor | 10.1 | x | x | x |
| $L$ | System length; unbraced length between nodes | 10.1 | x | - | x |
| $m$ | Number of angles | 14.2 | - | x | - |
| $N$ | Axial force | 11.3 | - | x | - |
| $N_d$ | Compression force, force in compression member | 11.1 | - | x | - |

| | | | | | |
|---|---|---|---|---|---|
| $N_{R,d}$ | Design buckling resistance of compression member | 10.1 | - | x | - |
| n | Number of holes | 9 | x | x | x |
| $n_t$ | Number of threads per length | 8.2.2 | x | - | - |
| P | Axial force | 8.1.2 | x | x | x |
| $P_1$ | Spacing of two holes in the direction of load transfer | 8.2.4 | - | x | - |
| p | Pitch of thread | 8.2.2 | x | - | - |
| $S_d$ | Tension force; force in the supporting member (tension or compression) | 11.1 | - | x | - |
| s | Spacing of two staggered holes in the longitudinal direction of the member axis | 8.1.2 | x | x | x |
| t | Plate thickness | 8.1.2 | x | x | x |
| vv | Minor principal axis | 11.2 | x | x | x |
| $W_{eff}$ | Effective cross section modulus for uniform compression (with $b = b_{eff}$) | 16 | x | x | - |
| w | Flat width of the member | 10.2 | x | - | - |
| yy | Axis of a cross-section | 11.2 | x | x | x |
| zz | Axis of a cross-section | 11.2 | x | x | x |
| $\alpha$ | Value for the maximum bearing stress | 8.2.4 | - | x | - |
| $\alpha$ | Imperfection factor for buckling curve | 10.1 | - | x | - |
| $\alpha$ | Angle between the redundant member and the supported member | 12 | - | x | - |
| $\gamma_{M1}$ | Partial factor for resistance of member in tension, bending or buckling | 10.1 | - | x | - |
| $\gamma_{M2}$ | Partial factor for resistance of net cross section at bolt holes | 8.1.2 | - | x | - |
| $\gamma_{Mb}$ | Partial factor for resistance of bolted connections | 8.2.1 | - | x | - |
| $\Lambda$ | Non dimensional slenderness ratio | 10.1 | - | - | x |
| $\overline{\Lambda}$ | Modified non dimensional slenderness ratio | 10.1 | - | - | x |
| $\lambda$ | Slenderness ratio for the relevant buckling load | 10.1 | - | x | x |
| $\lambda_p$ | Ratio of width to thickness (b/t) | 10.2 | - | x | - |
| $\lambda_0$ | Slenderness ratio of the full member (double angles) | 14.2 | - | x | x |
| $\lambda_1$ | Slenderness ratio of the sub member (double angles) | 14.2 | - | x | x |
| $\overline{\lambda}$ | Non-dimensional slenderness for the relevant buckling load | 10.1 | - | x | - |
| $\overline{\lambda}_{eff}$ | Effective non-dimensional slenderness for the relevant buckling load | 10.1 | - | x | - |
| $\overline{\lambda}_p$ | Reduced ratio of width to thickness | 10.2 | - | x | - |
| $\rho$ | Reduction factor for uniform compression of members | 10.2 | - | x | - |
| $\overline{\sigma}$ | Conventional yield strength taking into account local and torsional buckling | 10.2 | - | - | x |
| $\chi$ | Reduction factor for the buckling resistance | 10.1 | - | x | - |

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