SSAB DOMEX TUBE
STRUCTURAL HOLLOW SECTIONS

EN 1993 –HANDBOOK 2016

SSAB

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EN 1993 – handbook

2016

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FOREWORD - SSAB handbook (2016 edition)

This publication is the SSAB edition of the former 2012 handbook for Ruukki structural hollow sections. The motivation to publish this edition comes from the merge of SSAB and Rautaruukki in 2014. In consequence of this merge, the former Ruukki's EN 10219 structural hollow sections are now SSAB's EN 10219 structural hollow sections. At the same time also the structural hollow sections' product families and their features have been renewed. The product families for structural hollow sections are now SSAB Domex Tube and Strenx Tube.

The above-mentioned need for this edition has also enabled to correct the misprints which have been discovered in the 2012 publication, as presented in the separately published Errata dated 20.4.2016 for the Ruukki handbook 2012 edition. In addition, also some further improvements of editorial nature have been carried out.

Like the principle with the Ruukki handbook, also this SSAB handbook is based on the up-to-date standards and design codes, being valid at the time of publishing. Though it has already passed four years since publishing the 2012 edition, the most important reference standards related to this handbook (i.e. product standard EN 10219, applicable Parts of EN 1991 and EN 1993 for structural design, applicable Parts of EN 1090 for execution of steel structures) have still stayed unchanged. This enables that the standard-based content of the handbook has also been kept unchanged (except the misprints and certain changes related to the technical features of the hollow section products, as stated above). **The applied reference standards and their related revisions of that time, have been presented more detailed on the next page, see *Foreword - Ruukki handbook 2012 edition*.** The applied reference standards and their revisions have been presented in full details at *References*, located in this handbook at the end of each Chapter.

The above-stated holds true with one exception: In 2014, Part EN 1993-1-1 of Eurocode was amended with a fully new annex (Annex C), published as Amendment A1:2014. It gives provisions for selection of execution class (EXC). These provisions differ from former ones presented in standard EN 1090-2. In order to be applied, Annex C of EN 1993-1-1 requires national choices, which are to be determined in a National Annex (NA). In Finland, Annex C is not yet applicable, since Finland has not published a new revision for the related National Annex, that would be required. This is also why Annex C is not presented in this handbook, but the selection of execution class has been presented according to provisions given in the current EN 1090-2.

The current reference standards are the reason why CE marking, presented in Chapter 1 of this handbook, is still based on the Construction Products Directive (CPD) of the European Union, though it has been superseded already in 2013 by the Construction Products Regulation (CPR): the current product standard EN 10219 for structural hollow sections as well as EN 1090 for the execution of steel structures, are still based on the CPD, though it is superseded.

The handbook for Ruukki structural hollow sections has a long history in Finland. Consequently, even though the Swedish SSAB and the Finnish Rautaruukki have now merged, the Finnish background is still present in the book: the national choices determined in Eurocode context are presented herein as given in the Finnish National Annex.

The handbook has been revised into SSAB context by Petri Ongelin (M.Sc.Tech), Jussi Minkkinen (M.Sc.Tech) and Petteri Steen (Lic.Sc.Tech).

Hämeenlinna 24.5.2016

SSAB Europe Oy
Tubes and Sections

FOREWORD - Ruukki handbook (2012 edition)

This volume is a fully revised edition of the handbook for Rautaruukki structural hollow sections (publication MEF 27/99), that replaces all the former editions.

The aim of this book is to provide design guidance for structures manufactured of Ruukki structural hollow sections. It is also intended as a textbook. The book is primarily based on the European design code for steel structures, Eurocode 3 (EN 1993). In the book also the Amendment Corrigendas (AC) to the different Parts of Eurocode, published by the end of year 2011, have been taken into account.

The rules of Eurocode 3 have been complemented, when necessary, by design guidance published by CIDECT (Comité International pour le Développement et l'Étude de la Construction Tubulaire) for structures made of structural hollow sections. It should be noted, however, that the renewed rules by CIDECT published in the years 2008 - 2009 have not been used as source material because they differ somewhat from the current rules in Eurocode 3.

The primary scope of this handbook is building construction, but it can also be used in machine construction, where applicable.

The main focus of the book is in the structural design, but also the workshop manufacturing, transport and erection of structures made of structural hollow sections are briefly discussed. In respect to the steel structure designer, the essential matters related to workshop processing of structural hollow sections are presented based on EN 1090. This enables that the requirements related to workshop manufacturing can be taken into account already in the designing phase. In the book also Amendments (A1) to the different Parts of EN 1090, published by the end of year 2011, have been taken into acount. The design provisions of Eurocode 3 apply to steel structures which shall be executed according to EN 1090.

The handbook supersedes the former volume which was based on the ENV version of Eurocode, that was in design praxis at that time. Although the rules of EN-Eurocode have basically stayed the same as in the ENV version, a lot of changes have also taken place. Both design systems themselves are their own ensemble. Different parts of them are not allowed to be combined with each other.

The handbook is written by Petri Ongelin (M.Sc.Tech): Chapters 1-7 and Ilkka Valkonen (M.Sc.Tech): Chapters 8-10. In making the book, mechanical technician Mirkka Salonen and engineer Markus Tupala as well as engineering student Kaisa Saari and engineering student Jesse Saari have all assisted. In checking the book, (M.Sc.Tech) Jouko Kouhi, (M.Sc.Tech) Jouko Kansa, (M.Sc.Tech) Jan Jensén, (M.Sc.Tech) Juha Ikkala, (M.Sc.Tech) Juha Rajala (Chapter 1), M.Sc.(Tech) Jussi Minkkinen (Chapter 1), (M.Sc.Econ) Mikko Alinikula, (D.Sc.Tech) Jyri Outinen (fire design) and (M.Sc.Tech) Unto Kalamies (EN 1090: execution of steel structures, CE marking) have all participated. The layout of the book is done by Graafinen Palvelu Martti Lepistö. The book is printed and bound by Otavan Kirjapaino Oy.

Any comments and suggestions for improving the contents of the handbook are welcome.

Hämeenlinna 28.5.2012

RAUTARUUKKI OYJ

CONTENTS

INTRODUCTION

The structural hollow section is a modern and versatile element for steel structures. It is also an environmentally friendly choice, since it is easy to recycle and re-use. The simplistic form of hollow sections and their excellent strength properties make hollow section structures a light-weight and cost-efficient solution.

SSAB structural hollow sections provide the engineer with a wide range of dimensions which, when combined with the appropriate choice of steel grade for the structure, makes it possible to optimize the functionality and cost-efficiency of the structure. Additionally, structural hollow sections make it possible to create visually imposing structures, and especially the laser technology having become a more common cutting method, has improved the usability of circular hollow sections.

In a lattice structure, the high buckling resistance of hollow sections enables the use of long spans and a large spacing between diagonals. Due to the superior torsional stiffness of the closed cross-sectional shape, lattice structures made of hollow sections, as well as individual hollow sections, have good resistance to lateral-torsional buckling. The simple joint details are cost-efficient to fabricate. The rounded corners and easily accessible joints facilitate the surface treatment.

By using structural hollow sections, it is possible to design light-weight and rigid frame structures, since their torsional and bending stiffness in all directions is high. The torsional stiffness of the hollow sections can be utilised also in various console and cantilever structures. In bracing members, the high stiffness serves to produce a low amount of deflection.

Another well-suited application for hollow sections is in composite structures. When using a concrete-filled composite column, the properties of steel and concrete can be optimally utilised under on-site execution, normal loading and fire situation.

The design of a hollow section structure is easy and quick. The simple geometry can be expressed with few parameters, which makes software-based design a feasible option. The weight, resistance and stiffness of the structure can be optimized by changing the wall thickness, without needing to change the external dimensions of the hollow section or the geometry of the structure.

This handbook includes data on materials and dimensions of the structural hollow sections manufactured by SSAB. It also provides instructions for the structural design of cross-sections, joints and structures. In addition to statically loaded structural hollow sections, the book covers also fatigue design and fire design of structural hollow sections.

The handbook is complemented by the WinRami design software, developed by SSAB (Ruukki) especially for structural design of hollow section structures (see Annex 11.6). Additional information on WinRami software is available on Ruukki web site [www.ruukki.com].

Although the main focus of the book is in the structural design, also workshop manufacturing, transport and erection of structures made of structural hollow sections are briefly discussed. In respect to the steel structure designer, the essential matters related to workshop processing of structural hollow sections are presented based on EN 1090. This enables that the requirements related to workshop manufacturing can be taken into account already in the designing phase. In the book also Amendments (A1) to the different Parts of EN 1090, published by the end of year 2011, have been taken into acount.

The design rules in the manual are based on Eurocode 3 (EN 1993), which is the European design code for steel structures. The design provisions of Eurocode 3 apply to steel structures which shall be executed according to EN 1090. In regard to Eurocode, the different Parts and Annexes of Eurocode are used as applicable. In the book also the Amendment Corrigendas (AC) to the different Parts of Eurocode, published by the end of year 2011, have been taken into account. **Worked examples in the handbook are based on the general assumptions and recommended values of Eurocode, unless otherwise mentioned.**

The Eurocode shall always be used together with the National Annex (NA) of the relevant country to the applied Part of Eurocode. In the handbook, along with each presented subject, the rules given in the Finnish National Annex are separately presented when the item is subjected to national choices.

The rules of Eurocode 3 have been complemented, when necessary, by design guidance published by CIDECT (Comité International pour le Développement et l'Étude de la Construction Tubulaire) for structures made of structural hollow sections. It should be noted, however, that the renewed rules by CIDECT published in the years 2008 - 2009 <u>have not</u> been used as source material because they differ somewhat from the current rules in Eurocode 3.

1. SSAB COLD-FORMED STRUCTURAL HOLLOW SECTIONS

SSAB manufactures high-quality cold-formed structural hollow sections in product families of Strenx Tube and SSAB Domex Tube. By cold-forming, high dimensional accuracy and surface quality are obtained.

SSAB Domex Tube and Strenx Tube families comprise the steel grades presented in Table 1.1.

SSAB Domex Tube product family comprises structural hollow sections in steel grades S235-S550. Compared to the requirements in the product standard, the entire SSAB Domex Tube product family has in many respects better properties guaranteed. The standard steel grade for SSAB structural hollow sections is SSAB Domex Tube Double Grade, which meets or exceeds the requirements of product standard EN 10219:2006 for both steel grades S355J2H and S420MH. It is provided with material certificates in regard to the both steel grades separately.

Strenx Tube product family comprises structural hollow sections in steel grades S700-S960. Their compliance for structural design by Eurocode 3 (EN 1993) must be considered in each case separately.

All SSAB Domex Tube and Strenx Tube structural hollow sections in steel grades S235-S700 are CE marked.

The design provisions given in this handbook cover cold-formed SSAB Domex Tube structural hollow sections in steel grades S235-S460 complying with product standard EN 10219:2006 [1,2]. Structural hollow sections in steel grades S235-S460 of EN 10219:2006 comply with design provisions given in Eurocode 3 (EN 1993), where they are deemed to satisfy the mechanical properties as required for the materials to be used [3,4,5].

The worked-out Examples in this handbook are based on using SSAB Domex Tube Double Grade. The Examples include also a comparison showing the potential weight-saving of steel material, enabled by utilising the higher strength of S420MH instead of usual S355J2H.

Table 1.1 Cold-formed SSAB Domex Tube and Strenx Tube structural hollow sections. Grades

	SSAB structural hollow section	EN 10219:2006 compatibility	prEN 10219:2016 compatibility
1)	SSAB Domex Tube 235JRH [a)]	S235JRH	S235JRH
	SSAB Domex Tube 355J2H [a)]	S355J2H	S355J2H
	SSAB Domex Tube Double Grade [a)]	**S420MH / S355J2H**	**S420MH / S355J2H**
	SSAB Domex Tube 460MH [a)]	S460MH	S460MH
2)	SSAB Domex Tube 500MH [b)]	–	S500MH
	SSAB Domex Tube 550MH [b)]	–	S550MH
	Strenx Tube 700MH,MLH [b)]	–	S700MH,MLH
	Strenx Tube 900MH [b)]	–	S900MH
	Strenx Tube 960MH [b)]	–	S960MH

1) This handbook covers design provisions by Eurocode 3 (EN 1993) for structural hollow sections in steel grades S235-S460.
2) This handbook does not cover structural design of steel grades over S460. Part EN 1993-1-12 of Eurocode gives supplementary design provisions for steel grades S500-S700 also in respect to hollow sections, but grades S500-S700 are not covered in current product standard EN 10219:2006, and their compliance for structural design by Eurocode 3 (EN 1993) must be considered in each case separately.

a) SSAB Domex Tube structural hollow sections meet or exceed the requirements of EN 10219:2006.
b) Steel grades S500-S960 are not covered in the current product standard EN 10219:2006. The forthcoming revision of the standard (draft prEN 10219:2016) is anticipated to include also steel grades S500-S960.

1.1 SSAB Domex Tube structural hollow sections

SSAB Domex Tube product family comprises structural hollow sections in steel grades S235-S550 as presented in Table 1.1. From those steel grades, the grades S235-S460 are covered in the current product standard EN 10219:2006, and grades S500-S550 are anticipated in the forthcoming revision of the standard (draft prEN 10219:2016).

The design provisions given in this handbook cover cold-formed SSAB Domex Tube structural hollow sections in steel grades S235-S460 complying with product standard EN 10219:2006 [1,2]. Structural hollow sections in steel grades S235-S460 of EN 10219:2006 comply with design provisions given in Eurocode 3 (EN 1993), where they are deemed to satisfy the mechanical properties as required for the materials to be used [3,4,5].

Compared to the requirements in the product standard, the entire SSAB Domex Tube product family has in many respects better properties guaranteed. The standard steel grade for SSAB structural hollow sections is SSAB Domex Tube Double Grade, which meets or exceeds the requirements of product standard EN 10219:2006 for both steel grades S355J2H and S420MH. It is provided with material certificates in regard to the both steel grades separately.

SSAB Domex Tube structural hollow sections apply in welded structures even at operating temperatures below -50 ºC. The subject is covered more detailed in Chapter 5.

SSAB Domex Tube structural hollow sections have good weldability also within the corner region. For welding at the corner of the cold-formed structural hollow sections, there are provisions and requirements in Part EN 1993-1-8 of Eurocode. SSAB Domex Tube structural hollow sections fulfill the given requirements, why they can be welded also at the corner without special actions (see more details in clause 1.12).

In addition to the basic requirements of EN 10219, all longitudinally welded *) SSAB Domex Tube structural hollow sections fulfill also the following additional requirements:

- all products are manufactured using fully aluminium-killed non-aging fine grain steels suitable for cold-forming ($Al_{total} \geq 0{,}02\,\%$)
- impact toughness testing at -40 ºC
- chemical composition: better values than in EN 10219
- carbon equivalent: maximum value better than in EN 10219
- welding at the corner permitted without restrictions (fulfills requirements of EN 1993-1-8)
- guaranteed non-existence of cracks in corners
- tolerance of wall thickness: better than in EN 10219:
  the tolerance of wall thickness is guaranteed at -5 % / +10 %,
  however, the minimum of ±0,2 mm and maximum of ±0,5 mm
- the products meet the following EN 10219:2006 options:
  - option 1.2: contents of alloying elements will be reported in the inspection certificate
  - option 1.3: impact properties are verified also for quality grades JR and JO
  - option 1.4: suitability for hot-dip galvanizing
  - option 1.5: no weld repairs are made to the base material of structural hollow sections
  - option 1.6: inspection and testing per delivery lot also for quality grades JR and JO

More detailed information on product properties is presented in clause 1.6, and on web site [www.ssab.com].

* Spirally welded structural hollow sections are not included in SSAB Domex Tube product family, and their technical delivery conditions are to be agreed separately for each order.

1.2 Manufacture of structural hollow sections

SSAB manufactures the structural hollow sections from hot-rolled steel strip by cold-forming and welding. Hollow sections in square and rectangular cross-sections, and small circular ($d \leq$ 323,9 mm) hollow sections, are seam-welded longitudinally using high-frequency welding (HFW), Figure 1.1. Large circular sizes ($d \geq$ 406,4 mm) are seam-welded spirally using submerged arc welding (SAW), Figure 1.2.

Quality control of the structural hollow sections is performed at all tube mills according to certified EN ISO 9001 quality system and EN ISO 14001 environmental managing system.

Longitudinally welded structural hollow sections

The material used in longitudinally welded hollow sections is steel strip, cut accurately to correspond the width of the external dimensions of the section. At the beginning of the production line, the steel strip is decoiled and the strip ends are welded together. The strip is then fed into a strip-accumulator to enable continuous manufacturing process.

The steel strip is shaped with forming rolls at room temperature step-by-step into a circular hollow section preform. The edges of the preform are heated to the welding temperature with high-frequency current using an induction coil, and then pressed together. External weld flash is removed from the hollow section. Seam quality is ensured using a continuous eddy current or ultrasonic inspection. Peeling the inside weld flash off is possible if separately agreed.

The diameter of the circular hollow section is calibrated to its final dimension. After this, the cross-section can be further formed to square or rectangular shape with profiling rolls. A continuous marking is made to the hollow section, and it is cut to the length according to customer orders. Samples are taken from the hollow section for testing the mechanical properties, and the destructive testing of welding will be performed according to EN 10219 and the quality system of the fabricating tube mill.

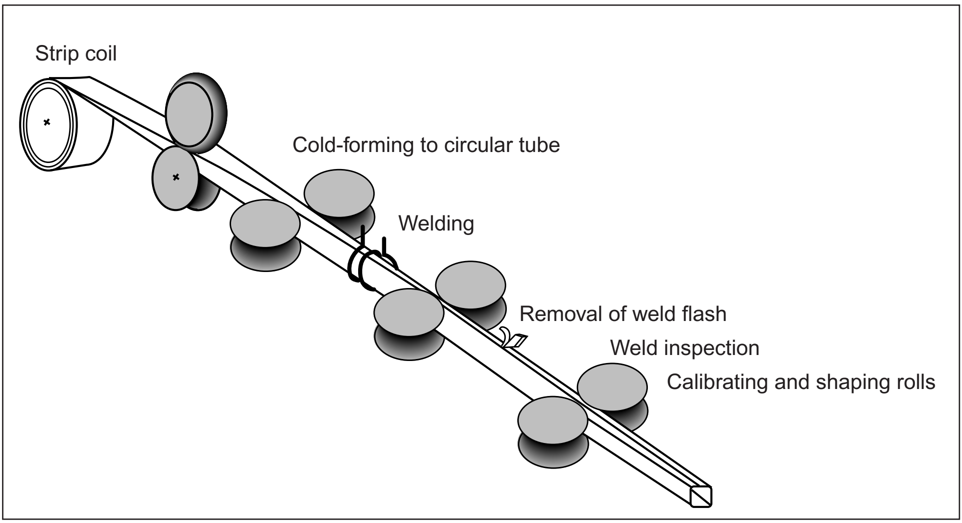

Figure 1.1 Longitudinally welded structural hollow sections. Manufacturing principle by cold-forming

After cutting, the dimensions of the hollow sections are checked and the products are packed in bundles. The specific hollow section which contains the welded end-to-end joint is scrapped. Each bundle is marked with an identification label specifying the properties of the products and their identification code. Based on the identification label, the properties of the hollow section can be traced back to the manufacture of the steel material.

Spirally welded structural hollow sections

SSAB also manufactures circular structural hollow sections in diameters Ø 406,4 ...1219 mm from hot-rolled steel strip by spirally welding. At the beginning of the production line, the single steel strips are welded to form a continuous strip which is then straightened and formed into a spirally welded tube using three-roll bending at room temperature. The spiral seam is welded both inside and outside using submerged arc welding.

The mechanical properties are tested using test coupons which are cut from the hollow section. The hollow sections are cut to the length according to customer orders, inspected and delivered to customers.

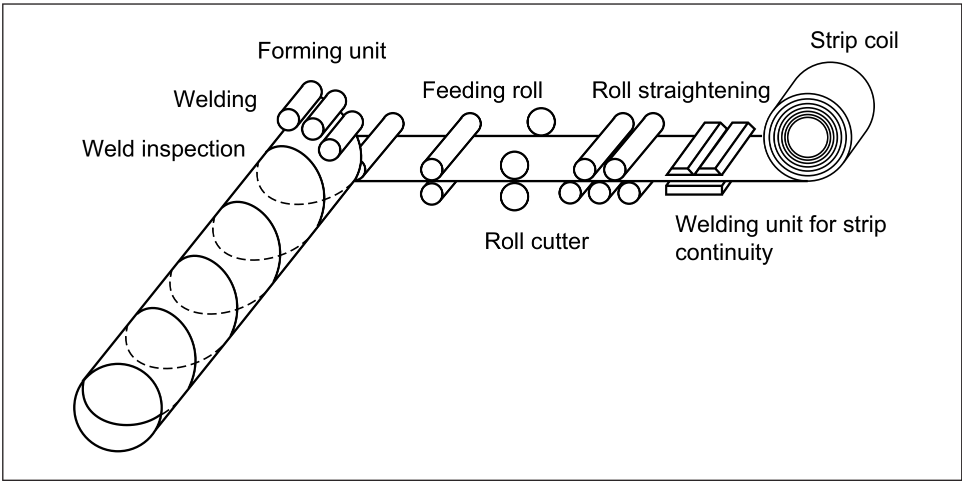

Figure 1.2 Spirally welded structural hollow sections. Manufacturing principle by cold-forming

1.3 Approvals

The whole fabrication chain from the steel mill to the finished tube is covered by the certified EN ISO 9001 quality system and EN ISO 14001 environmental managing system.

SSAB is entitled to CE marking of structural hollow sections made according to EN 10219, issued by Inspecta Sertifiointi Oy. CE marking is discussed in more details in clause 1.13.

1.4 Cross-sections, Dimensions, Cross-sectional properties and Marking on drawing

SSAB's production programme covers circular, square and rectangular cold-formed structural hollow sections according to EN 10219. The dimensions and cross-sectional properties of the hollow sections are presented in the tables of Annex 11.1. The dimensions apply to SSAB's standard steel grade for structural hollow sections, SSAB Domex Tube Double Grade. For other steel grades, the dimensions and their availability, as well as the possiblity for customer-designed dimensions, should be checked from SSAB sales office or on the web site [www.ssab.com].

The cross-sectional properties have been calculated using the nominal cross-section dimensions h, b, d and t and the nominal external radius r according to EN 10219, stated at the top of the tables.

By separate agreement, structural hollow sections can also be supplied in other steel grades, with other surface protection or surface treatment, cut off to specific lengths, and manufactured with special tolerances.

Spirally welded structural hollow sections are manufactured only by separate agreement.

Marking on drawing

Marking of SSAB structural hollow sections on drawing and order is presented in Figure 1.3.

SSAB
structural hollow section 18000 - CFRHS - EN 10219 - S460MH - 300 x 300 x 12,5 - Options 1.2 1.4 1.5 - 3.1

Name
Length
Cross-section form
Number of EN-standard
Steel grade
External dimensions and wall thickness (mm)
Options specified in accordance with EN 10219-1:2006
Inspection certificate EN 10204:2004

Keys:

- CFRHS		= cold-formed square or rectangular structural hollow section
- CFCHS		= cold-formed circular structural hollow section
- S460MH		
	S	= structural steel
	460	= yield strength (N/mm^2)
	M	= required impact energy 40J using a 10x10 mm^2 V-notch specimen acc. to EN ISO148-1. Required impact test temperature -20 ºC (EN 10219:2006) (on SSAB Domex Tube 460MH, required impact toughness is 27 J / -40 ºC, which exceeds EN 10219 requirement 40 J / -20 ºC, see EN 10219-1 / EN 1993-1-10)
	H	= hollow section

Figure 1.3 Marking of SSAB structural hollow sections

1.5 Tolerances on dimensions

SSAB guarantees for its longitudinally welded SSAB Domex Tube structural hollow sections a better thickness tolerance than required in EN 10219. All other dimensional tolerances are established according to EN 10219 as presented in Table 1.2 [2]. By request also different tolerances may be agreed.

For spirally welded structural hollow sections, technical delivery conditions are to be agreed separately for each order.

Table 1.2 Longitudinally welded SSAB Domex Tube structural hollow sections. Tolerances on dimensions

CHARACTERISTIC	Tolerance (equal or better than in EN 10219-2:2006)	
	Square and rectangular structural hollow section	Circular structural hollow section
Outside dimensions (b, h) [a)]	For $b, h < 100$ mm: ±1 %, at least ±0,5 mm For 100 mm $\leq b, h \leq 200$ mm: ±0,8 % For $b, h > 200$ mm: ±0,6 %	–
Outside diameter (d) [a)]	–	±1 %, with a minimum of ±0,5 mm and a maximum of ±10 mm
Out-of-roundness [b)]	–	2 %, for $d/t \leq 100$ [b)]
Thickness (t)	-5 % / +10 % [x)], with a minimum of ±0,2 mm and a maximum of ±0,5 mm x) EN 10219 requirement: -10 % / +10 %	For $d \leq 323{,}9$ mm: -5 % / +10 % [x)], with a minimum of ±0,2 mm and maximum of ±0,5 mm x) EN 10219 requirement: -10 % / +10 %
External corner radius (r)	For $t \leq 6$ mm: $1{,}6 \times t \ldots 2{,}4 \times t$ For 6 mm $< t \leq 10$ mm: $2{,}0 \times t \ldots 3{,}0 \times t$ For $t > 10$ mm: $2{,}4 \times t \ldots 3{,}6 \times t$	–
Squareness of side	90° ± 1°	–
Concavity/convexity of side [c)]	0,8 % with a minimum of 0,5 mm	–
Twist	2 mm + 0,5 mm/m	–
Straightness	0,15 % of total length [d)]	0,20 % of total length [d)]
Mass	± 6 % on individual delivered lengths	± 6 % on individual delivered lengths
Mill length	≥6000 mm: 0/+50 mm	≥6000 mm: 0/+50 mm
Exact length	To be agreed when ordering	To be agreed when ordering

a) All external dimensions on square sections shall be measured at a distance of at least b, and on rectangular sections at a distance of at least h, or on circular cross-sections at a distance of at least d from the end of the hollow section. The distance shall, however, be at least 100 mm.
b) For $d/t > 100$, tolerance shall be agreed when placing the order.
c) Tolerance of concavity/convexity is independent of tolerance on outside dimensions.
d) Local out-of-straightness maximum 3 mm over any 1 m length.

1.6 Materials to be used

1.6.1 Steel grades

SSAB manufactures its structural hollow sections using steel grades which are all fully aluminium-killed non-aging fine grain steels suitable for cold-forming ($Al_{total} \geq 0,02$ %).

SSAB Domex Tube product family comprises structural hollow sections in steel grades S235-S550 as presented in Table 1.1. From those steel grades, the grades S235-S460 are covered in the current product standard EN 10219:2006, and grades S500-S550 are anticipated in the forthcoming revision of the standard (draft prEN 10219:2016).

The design provisions given in this handbook cover cold-formed SSAB Domex Tube structural hollow sections in steel grades S235-S460 complying with the current product standard EN 10219:2006 [1,2]. Structural hollow sections in steel grades S235-S460 of EN 10219:2006 comply with design provisions given in Eurocode 3 (EN 1993), where they are deemed to satisfy the mechanical properties as required for the materials to be used [3,4,5].

The material properties of SSAB Domex Tube structuctural hollow sections for aforementioned steel grades S235-S460 are presented on next pages on Tables 1.3 - 1.6. For structural hollow sections in steel grades over S460, the compliance for structural design by Eurocode 3 (EN 1993) must be considered in each case separately.

All SSAB Domex Tube structural hollow sections are CE marked.

The standard steel grade for SSAB structural hollow sections is SSAB Domex Tube Double Grade. The chemical composition according to Table 1.3 and mechanical properties according to Table 1.4 are guaranteed for SSAB Domex Tube Double Grade.

The product complies with EN 10219 and fulfills the requirements set therein for the both steel grades S420MH and S355J2H. Thereby the design calculations for the hollow section structure may be performed at designer's own choice either according to grade S420 or grade S355. The product is provided with material certificates in regard to the both steel grades separately. The product is CE marked.

More detailed information on product properties is presented on web site [www.ssab.com].

Table 1.3 Comparison of the chemical composition of SSAB structural hollow section standard steel grade SSAB Domex Tube Double Grade, and the EN 10219 steel grades S355J2H and S420MH

Steel grade		Chemical composition (%)						
		C	Si	Mn	P	S	Al_{total}	CEV [c)]
SSAB Domex Tube Double Grade								
S355J2H	Maximum	0,16	0,25 [a)]	1,60	0,020	0,012	0,020 [b)]	0,39
S420MH	Minimum	0,16	0,25 [a)]	1,60	0,020	0,012	0,020 [b)]	0,39
EN 10219:2006								
S355J2H	Maximum	0,22	0,55	1,60	0,035	0,035	0,020 [b)]	0,45
S420MH	Minimum	0,16	0,50	1,70	0,035	0,030	0,020 [b)]	0,43

SSAB Domex Tube structural hollow sections meet or exceed the requirements of product standard EN 10219.

a) Si content guaranteed at 0,15...0,25 %
(corresponds to class 3 for hot-dip galvanizing in flat steel standard EN 10025-2, and to class B in hot-dip galvanizing standard EN ISO 14713-2)
b) Minimum %
c) Weldability is good when the value of the carbon equivalent is CEV < 0,41.
Formula for carbon equivalent, see clause 1.12.

Table 1.4 Comparison of the mechanical properties of SSAB structural hollow section standard steel grade SSAB Domex Tube Double Grade, and the EN 10219 steel grades S355J2H and S420MH

Steel grade	Wall thickness t (mm)	R_{eH} minimum (N/mm^2)	R_m (N/mm^2) wall thickness t (mm)		A_5 minimum (%)	Impact toughness test temperature (°C)	Impact energy minimum [b)] (J)
			$t < 3$	$3 \leq t \leq 16$			
SSAB Domex Tube Double Grade							
S355J2H	2,0 - 12,5	355	510 - 680	470 - 630	20 [a)]	-40	40
S420MH	2,0 - 12,5	420	500 - 660	500 - 660	19 [a)]	-40	40
EN 10219:2006							
S355J2H		355	510 - 680	470 - 630	20 [a)]	-20	27
S420MH		420	500 - 660	500 - 660	19 [a)]	-20	40

SSAB Domex Tube structural hollow sections meet or exceed the requirements of product standard EN 10219. SSAB tests the mechanical properties of the structural hollow sections using longitudinal test specimens. On rectangular hollow sections, the mechanical properties are tested on the longer side of the cross-section.

a) For structural hollow sections with $d/t < 15$ (circular) or $(b+h)/2t < 12,5$ (square and rectangular), the minimum value for elongation is 2 %-units smaller.
b) Impact energy using a 10 x 10 mm^2 V-notch test specimen in accordance with EN ISO 148-1.
The impact testing according to EN ISO 148-1 is performed on thicknesses ≥ 6 mm.

Table 1.5 Non-alloy SSAB Domex Tube structural hollow sections. Mechanical properties

Steel grade EN 10219	Wall thickness t (mm)	R_{eH} minimum (N/mm^2)	R_m (N/mm^2) wall thickness t (mm) $t < 3$	$3 \le t \le 16$	A_5 minimum (%)	Impact toughness test temperature (ºC)	Impact energy minimum [d)] (J)
SSAB Domex Tube							
S235JRH	2,0 - 12,5	235	360 - 510	360 - 510	24 [a)]	- 40 [b)]	27
S355J2H	2,0 - 12,5	355	510 - 680	470 - 630	20 [a)]	- 40 [c)]	27

SSAB Domex Tube structural hollow sections meet or exceed the requirements of product standard EN 10219. SSAB tests the mechanical properties of the structural hollow sections using longitudinal test specimens. On rectangular hollow sections, the mechanical properties are tested on the longer side of the cross-section.

a) For structural hollow sections with d/t < 15 (circular) or $(b+h)/2t$ < 12,5 (square and rectangular), the minimum value for elongation is 2 % -units smaller.
b) SSAB guaranteed value (EN 19219 requirement for test temperature is +20 ºC).
c) SSAB guaranteed value (EN 10219 requirement for test temperature is -20 ºC).
d) Impact energy using a 10 x 10 mm^2 V-notch test specimen in accordance with EN ISO 148-1. The impact testing according to EN ISO 148-1 is performed on thicknesses ≥ 6 mm.

Table 1.6 Fine grain SSAB Domex Tube structural hollow sections. Mechanical properties

Steel grade EN 10219	Wall thickness t (mm)	R_{eH} minimum (N/mm^2)	R_m (N/mm^2)	A_5 minimum (%)	Impact toughness test temperature (ºC)	Impact energy minimum [c)] (J)
SSAB Domex Tube						
S460MH	2,0 - 12,5	460	530 - 720	17 [a)]	- 40 [b)]	27 [b)]

SSAB Domex Tube structural hollow sections meet or exceed the requirements of product standard EN 10219. SSAB tests the mechanical properties of the structural hollow sections using longitudinal test specimens. On rectangular hollow sections, the mechanical properties are tested on the longer side of the cross-section.

a) For structural hollow sections with d/t < 15 (circular) or $(b+h)/2t$ < 12,5 (square and rectangular), the minimum value for elongation is 2 % -units smaller.
b) SSAB guaranteed impact toughness value 27 J / -40 ºC corresponds to 40 J / -30 ºC (see EN 1993-1-10), which exceeds EN 10219 requirement 40 J / -20 ºC.
c) Impact energy using a 10 x 10 mm^2 V-notch test specimen in accordance with EN ISO 148-1. The impact testing according to EN ISO 148-1 is performed on thicknesses ≥ 6 mm.

In addition to the aforementioned steel grades, SSAB manufactures cold-formed structural hollow sections also in other steel grades, as in Strenx Tube 700MH-960MH and SSAB Weathering Tube 355WH-500WH. Their compliance for structural design by Eurocode 3 (EN 1993) must be considered in each case separately. For these steel grades the mechanical properties, chemical composition, design guidance and dimensional ranges are presented on web site [www.ssab.com].

1.6.2 Material requirements in Eurocode for steel grades S235-S460 and nominal strength values of the materials

Eurocode provides that the steel material to be used shall have sufficient ductility. The ductility is expressed by setting limits for the following characteristics:

- the ratio between the minimum values of ultimate tensile strength f_u and yield strength f_y
- minimum value of elongation at failure on gauge length $L_o = 5{,}65\sqrt{A_o}$
- minimum value of ultimate strain ε_u (total uniform elongation) corresponding to ultimate tensile strength f_u

The minimum values for the above stated characteristics may be defined in the National Annex. For steel grades S235-S460, the recommended values in Eurocode are [3,4,5]:

- $f_u / f_y \geq 1{,}10$
- elongation at failure on gauge length $L_o = 5{,}65\sqrt{A_o}$ at least 15 %
  (or elongation A_5 at least 15 %)
- $\varepsilon_u \geq 15\varepsilon_y$, where ε_y is the yield strain ($\varepsilon_y = f_y/E$)

Finnish National Annex to standard EN 1993-1-1 [6]:

Recommended values of Eurocode are used if not otherwise stated in some Part of EN 1993 or in a National Annex to some Part of EN 1993.

In structural design, as it comes to the material properties, steels of grades S235-S460 have thereby sufficient toughness and deformability to apply, in addition to the theory of elasticity, also the theory of plasticity:

- resistances can be calculated according to plastic theory, if the cross-section of the member fulfills the requirements of cross-section Class 1 or 2

- forces and moments can be calculated using plastic theory in the global analysis of the structure (or part of the structure), if the cross-section of the member fulfills the requirements of cross-section Class 1.

In Part EN 1993-1-1 of Eurocode, it is stated that the EN 10219 cold-formed structural hollow sections are deemed to satisfy the requirements set for the materials to be used [3,4,5].

Finnish National Annex to standard EN 1993-1-1 [6]:

Additionally those steel grades can be used that have a valid product approval.

Comment by the author:
In Finland, one form of product approvals is Certified Product Declaration. The Finnish Constructional Steelwork Association keeps up-to-date record covering the valid Certified Product Declarations, issued for steel construction applications.
The list of the certificates, and the certificates in pdf-format, are placed on the web site of FCSA: [www.terasrakenneyhdistys.fi]

It has been proved also in praxis by research that the material ductility of EN 10219 cold-formed structural hollow sections is sufficient also for plastic design. Thereby, in praxis, the cross-section Class dependent geometric deformation capacity of the structural member becomes decisive in structural design, instead of the ductility of the material [7]. Furthemore, it has also been proved that there is no essential difference between cold-formed and hot-formed hollow sections in regard to the form of moment-rotation characteristic, or in the rate of the available plastic rotation capacity [7].

In calculations, the nominal values of material properties are used as characterictic values. The nominal values of yield strength f_y and ultimate tensile strength f_u are obtained for structural hollow sections as follows [3,4,5]:

a) either by adopting the values $f_y = R_{eH}$ and $f_u = R_m$ direct from the product standard (EN 10219)

b) or by using the simplified nominal values presented in Eurocode (EN 1993-1-1) and in Table 1.7.

The method to be used may be defined in the National Annex.

Finnish National Annex to standard EN 1993-1-1 [6]:
Both alternatives may be used.

In the calculated Examples of this handbook, for structural hollow sections of steel grades S235-S460 the values presented in Table 1.7 according to Eurocode have been used. In the table, the designations of different steel grades have been combined and shortened when applicable.

The material to be used shall also have sufficient fracture toughness to avoid brittle fracture of tensile elements at the lowest operating temperature expected to occur within the intended design life of the structure. Furthermore, the structural details can cause a risk to lamellar tearing, that can be avoided by requiring improved through-thickness properties for the material. Material selection in respect to brittle fracture and lamellar tearing is covered in Chapter 5.

Table 1.7 Nominal values of yield strength and ultimate tensile strength stated in Part EN 1993-1-1 of Eurocode for EN 10219 cold-formed structural hollow sections in steel grades S235-S460 [3,4,5]

Standard and steel grade	Wall thickness t ≤ 40 mm	
EN 10219-1	f_y (N/mm^2)	f_u (N/mm^2)
S235 H	235	360
S275 H	275	430
S355 H	355	510
S275 NH/NLH	275	370
S355 NH/NLH	355	470
S460 NH/NLH	460	550
S275 MH/MLH	275	360
S355 MH/MLH	355	470
S420 MH/MLH	420	500
S460 MH/MLH	460	530

The values in this table are the simplified values presented in Part EN 1993-1-1 of Eurocode. These values have been used in the calculated Examples of this handbook. **The national provisions shall be checked from the National Annex of the relevant country.**

1.6.3 Design values of the material coefficients

The material coefficients to be adopted in design calculations for structural hollow sections at ambient temperature, shall be taken as follows [3,4,5]:

- Young's modulus of elasticity $E = 210\,000\ N/mm^2$
- shear modulus $G = \dfrac{E}{2(1+\nu)} \approx 81\,000\ N/mm^2$
- Poisson' s ratio in elastic stage $\nu = 0{,}3$
- coefficient of linear thermal expansion $\alpha = 12 \cdot 10^{-6}/°C$ (for $T \leq 100\ °C$)
- unit mass $\rho = 7850\ kg/m^3$

1.7 Inspection document

Structural hollow sections are supplied with an EN 10204 inspection document as agreed upon in the order contract. In the inspection document, the cast analysis is reported in compliance with EN 10219 option 1.2. Usually the inspection certificate EN 10204-3.1 is used, if not otherwise agreed. Inspection certificate EN 10204-3.1 meets also the requirements specified in EN 1090-2 for the execution of steel structures (Table 1.8).

Table 1.8 Determination of the type of inspection document according to EN 1090-2 and EN 10025-1 [9,10]

Standard	Inspection document
EN 1090-2:	
Structural steels (incl. structural hollow sections)	EN 10025-1: according to Table B.1 a)
Welding consumables	2.2
Structural bolting assemblies	2.1 b)
a) For structural steel grades S355 JR or J0, inspection document 3.1 is however required in execution classes EXC2, EXC3 and EXC4. b) If certificate type 3.1 is required, this may be substituted by a manufacturing lot identification mark.	
EN 10025-1:	
Specified minimum yield strength for the thinnest thickness range ≤ 355 N/mm^2 and a specified impact energy tested at a temperature of 0 ºC or 20 ºC	2.2
Specified minimum yield strength for the thinnest thickness range ≤ 355 N/mm^2 and a specified impact energy tested at a temperature less than 0 ºC	3.1 or 3.2
Specified minimum yield strength for the thinnest thickness range > 355 N/mm^2	3.1 or 3.2

1.8 Length

The standard lengths of structural hollow sections are 6 and 12 metres, and for large square and rectangular hollow sections (up from sizes 100 × 100 / 120 × 80 mm) also 18 metres. By agreement, square and rectangular hollow sections are available in specified lengths up to 12 or 24 metres depending on the size, and circular hollow sections in specified lengths up to 12 or 16 metres.

SSAB provides also various cut-off services. The hollow sections can be band-sawed to the lengths required by the customer. Also laser cutting is available. Laser cutting is recommendable when, for example, the end of the hollow section requires mitre cutting or ‘profiled’ cutting to a specific prescribed form. Cutting to a prescribed form, enabled by 3D-laser, is useful especially considering the joints of circular hollow sectons. The subject is discussed in more details in Chapter 8.

1.9 Surface protection

Structural hollow sections are delivered by agreement either without surface protection (delivery state ‘dry’) or lightly oiled. By agreement, structural hollow sections can also be delivered shot blasted and primer coated.

1.10 Identification

SSAB’s longitudinally welded structural hollow sections are marked with continuous ink-jet label indicating the following information:

- the SSAB emblem
- dimensions of the hollow section
- identification number to ensure the traceability of the hollow sections back to the proper productions and material data.

1.11 Surface treatment of structural hollow sections

1.11.1 Painting

The metal surface to be painted should be as smooth and round-cornered as possible. Hollow sections apply in this respect excellently also for spray painting, since they are by nature round-cornered. The outside surface protection of the hollow sections is cheaper than that of the corresponding open sections due to the closed form of the hollow sections. The painting surface of the hollow sections (i.e. external surface area A_u) is presented in the tables of Annex 11.1.

Paint coating shall be done to a dry and clean surface according to the specifications by the paint manufacturer. Recommended surface preparation methods and paint combinations are presented in EN ISO 12944. Paint coating is covered more detailed in Chapter 9.

1.11.2 Hot-dip galvanizing

To ensure the suitability for hot-dip galvanizing, the silicon content of SSAB Domex Tube Double Grade structural hollow sections is guaranteed at 0,15...0,25 % Si. This corresponds to class 3 for hot-dip galvanizing in flat steel standard EN 10025-2, and to class B in hot-dip galvanizing standard EN ISO 14713-2. With an appropriate galvanizing procedure this silicon content enables a zinc coating thickness of over 100 μm being sufficient for most applications (Figure 1.4).

The Si content of other steel grades depends on the steel grade. On some steel grades, silicon and phosphorus content is Si+P $\leq$ 0,04 %. With an appropriate galvanizing procedure this silicon content enables a thin zinc coating (below 100 μm). In this case, the surface quality of zinc coating will be smoother and shinier than with 0,15...0,25 % Si. Should there be specific requirements regarding silicon and phoshorus content, these shall be agreed when placing the inquiry and order. Hot-dip galvanizing is discussed in more details in Chapter 9.

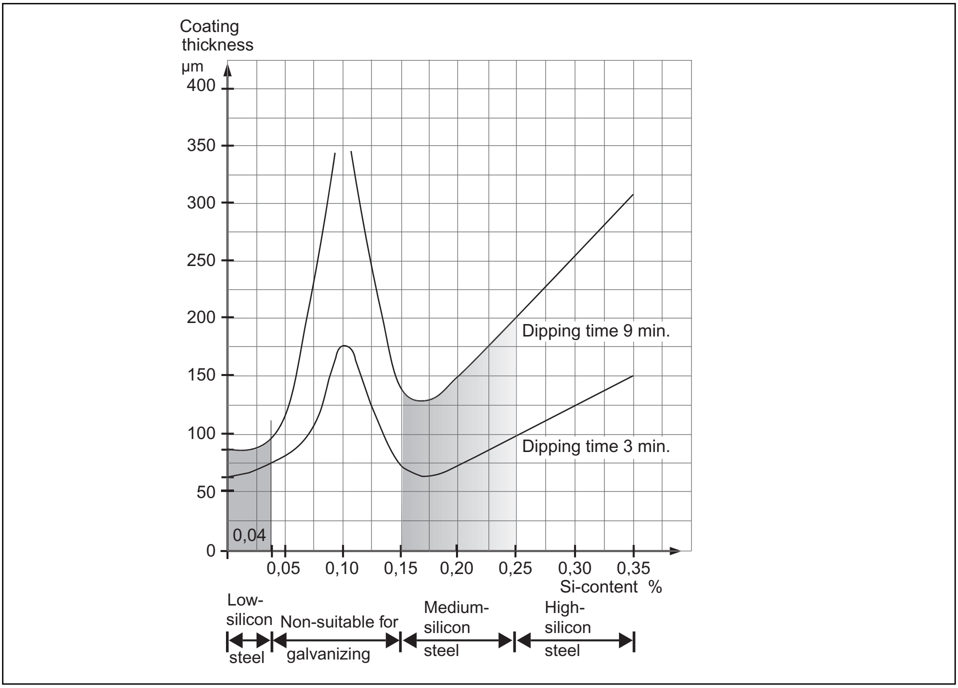

Figure 1.4 Hot-dip galvanizing. The thickness of zinc coating in relation to the Si-content. When the Si-content is on the shaded area in the graph, the thickness and adhesion of the zinc coating can be controlled well, provided an appropriate galvanizing procedure is applied.

1.12 Welding

SSAB Domex Tube structural hollow sections manufactured in steel grades presented in Tables 1.3 - 1.6 have good weldability with all common welding methods. In normal workshop conditions, elevated working temperature is not required when welding material thicknesses applied in hollow sections.

The weldability of steel is usually evaluated by the value of carbon equivalent (CEV) calculated on the basis of chemical composition of steel. The smaller the carbon equivalent, the better the steel is to weld. The most commonly used formula to calculate the carbon equivalent is the formula by IIW (International Institute of Welding):

$$CEV = C + \frac{Mn}{6} + \frac{Cr + Mo + V}{5} + \frac{Cu + Ni}{15} \tag{1.1}$$

As regard to cold cracking (hydrogen cracking), the steel is easily weldable by any common process, when the CEV value is less than 0,41. On SSAB structural hollow section standard steel grade SSAB Domex Tube Double Grade, the maximum value of carbon equivalent is 0,39, as stated in Table 1.3.

The cold-formed SSAB Domex Tube structural hollow sections have good weldability also within the corner region. For welding at the corner of the cold-formed structural hollow sections, there are provisions and requirements in Part EN 1993-1-8 of Eurocode. The SSAB Domex Tube structural hollow sections fulfill the given requirements, why they can be welded also at the corner without special actions (see more details in Chapter 3, Table 3.7).

1.13 CE marking and attestation of conformity

1.13.1 Construction Products Directive and new Construction Products Regulation

Directives are legal acts of European Union, addressed to the Member States in order to be implemented in their national legislation accordingly. Normally they are not directly effective laws in Member States, but after a Directive has been set into force by European Union, its content shall be implemented in the national legislation of each Member State within a specified time schedule [12].

Unlike a Directive, a European Union Regulation is, after it has been set into force by European Union, immediately an effective regulation which has to be complied in each Member State. Thus, it does not require national implementation into a part of legislation in each Member State. However, when a Regulation comes into force, national decrees shall be changed if needed so that they are not in contradiction with the Regulation of EU [12].

Construction Products Directive

Construction Products Directive 89/106/EEC is a so-called New Approach directive that enables CE marking of products.

CE marked construction products can be marketed and used, without technical barriers to trade, on markets of all countries within the European Economic Area. The objective of the Directive is to create a single internal market area covering the whole European Economic Area without any national technical barriers to trade.

Construction products cannot be CE marked due to the Directive only, but it requires also a harmonised product standard covering the specific product group, or a manufacturer and product specific European Technical Approval. To be entitled to affix CE marking to a product, the manufacturer has to comply to a harmonised product standard (hEN) or a European Technical Approval (ETA).

In most Member States of EU, CE marking of construction products has been regulated obligatory for those products having a harmonised product standard whose co-existence period has expired. In some Member States (Finland, Sweden, Great Britain and Ireland, and additionally also Norway as from the countries of the European Economic Area) CE marking is not obligatory, unless it has been particularly regulated obligatory for a specific product group [12].

New Construction Products Regulation

The first CE marked construction products on the basis of European Technical Approval came to the markets in the year 2000, and the first CE marked products on the basis of a harmonised product standard in the year 2001.

Now that the system has already extended to cover hundreds of product groups, it has been discovered that the implementation of the CE marking system, based on the Construction Products Directive, has proven to be non-uniform in extent of the European Economic Area. Non-uniformity has appeared, for example, in obligatoriness of CE marking, in withdrawal of national overlapping product approval systems, in accreditation and policy of Notified Bodies, and also in market surveillance of construction products. The above mentioned inconsistent practices have caused the problem that the CE marking system, based on Construction Products Directive, has not fully attained the goals the European Union has set to it.

To remove the above mentioned problems, the European Commission has drawn up a proposal for a Construction Products Regulation that, when coming into force, will supersede the former Construction Products Directive.

Construction Products Regulation (CPR) has been completed and has been published on 9.3.2011 (Regulation (EU) No 305/2011). The regulation comes into force in two steps:

- 24.4.2011: Construction Products Regulation comes into force in specific parts
- 1.7.2013: Construction Products Regulation comes into force in all parts.

In order to simplify, and due to a still ongoing transition period, and due to some still open national consequences and interpretations, the general information and references given on the following pages of this handbook in regard to CE marking, are still based on Construction Products Directive. However, it should be mentioned already now, that when the Construction Products Regulation supersedes the former CPD, the manufacturer will be obliged to provide a Declaration of Performance (DoP) instead of the former Declaration of Conformity (DoC). The DoP shall comprise all the same information and product characteristics, which are presented on the next pages according to former CPD for Declaration of Conformity and the CE marking. In regard to structural hollow sections as well as hollow section structures, the product group specific requirements are based on the effective harmonised product standards EN 10219-1 and EN 1090-1 respectively.

1.13.2 General rules regarding CE marking of construction products

The product standard EN 10219 [1,2] covers structural hollow sections. Furthermore, load-bearing hollow section components that are to be used in load-bearing structures, are covered by product standard EN 1090 for load-bearing structures [8,9]. The Part 1 of both aforementioned standards is a so-called harmonised standard that comprises a specific annex, Annex ZA, assigning the provisions for the product-specific CE marking, and provisions for the product characteristics to be presented in the CE marking.

In EN-Eurocode, the primary provision is that concerning steel structures, the design rules and reliability levels hold true for structures which are executed exclusively according to EN 1090.

Under Construction Products Directive (CPD, 89/106/EEC), CE marking is obligatory on internal market of EU for those construction products that have been published on Official Journal (OJ) of European Union, and for which the therein stated co-existence period of CE marking of the harmonised product standard has expired.

Apart from other Member States, based on the legislation in Finland, Sweden, England and Ireland, CE marking is for the time being basically a voluntary product approval system. However, CE marking will come obligatory also in these countries latest along with the Construction Products Regulation (CPR).

The main goal of the Construction Products Directive is to remove the technical barriers to trade, caused by the different national requirements set to construction products in Member States. In harmonised product standards and in European Technical Approvals, the characteristics to be required for the product are harmonised as well as the methods how to present these properties. This way the technical barriers to trade due to different product requirements set by national authorities, should automatically vanish. Under national regulations, the user of a CE marked product cannot be required to carry out a new testing procedure or to assess the product under a national approval system. Thus, the attestation of conformity carried out, for example, in Finland is sufficient for placing the product on market on the whole European Economic Area [11].

The regulations concerning CE marking shall be applied equally for construction products which are produced only to domestic market, as well as products to be exported to the European Economic Area, as also to products imported to Finland.

CE marking is not a mark of origin, and it does not prove that the product has been manufactured within the European Economic Area. CE marking must be affixed to a construction product by the manufacturer or its authorized representative. By fastening the CE mark, the manufacturer declares that the product conforms to the harmonised product standard, or to the European Technical Approval (ETA), under which it is manufactured. This means in praxis that:

- the manufacturer has manufactured the product in compliance with the referred document
- the manufacturer has taken care of Factory Production Control as well as relevant testing procedures as appropriate
- a Notified Body (such as a testing laboratory, inspection body or assessment body), being independent from the manufacturer, has carried out the tasks assigned to it, if such tasks are necessary regarding the relevant product.

Conformity of the CE marked construction product is the responsibility of the natural or legal person who places the product on the market. Primarily this means the manufacturer of the product. In case of a non-domestic product, the importer is responsible for the conformity of the product on the Finnish market, no matter is the product manufactured in some country inside the European Economic Area, or outside it [11].

It is the responsibility of market surveillance of construction products to ensure that on the market of European Economic Area there are only such CE marked construction products, which are in compliance with the requirements. In Finland it is TUKES (Turvatekniikan keskus) who acts as the market surveillance authority for the CE marked construction products, authorized by the Ministry of the Environment. The market surveillance is directed to the products placed on the market in Finland, no matter are they manufactured in Finland, or in some other country of the European Economic Area, or outside it [11].

In the CE marking, it is not always necessary to declare all the harmonised characteristics. If there are Member States not having regulated requirements for a specific characteristic of the product in a specific application, the manufacturer is not obliged to declare conformity of those characteristics in those Member States. In the CE marking, such a characteristic may be marked thereby as NPD, No Performance Determined. When given in the harmonised standard, the marking NPD may also be used to tell that a specific characteristic is not relevant for that product. (This marking should not be confused with marking NDP, Nationally Determined Parameter, used commonly in Eurocode.)

Each Member State decides, based on its own national premises, whether all the characteristics established for the product in the harmonised standard or Technical Approval shall be declared, or only some of them. However, any other or differing characteristics for the product cannot be required by the authorities [11].

Requirement levels to be set to the structures and their components are still regulated on national level, and the fit-for-purpose of the product is always judged on the basis of these requirements. In national regulations, there are special requirements and classifications that depend on climatic and geographic conditions, why it is always necessary to check the fulfillment of those requirements when considering a specific product. In Finland and other Nordic countries, a typical requirement of this kind is the freeze-thaw durability. This is why a product, even if it conforms to a European harmonised standard or European Technical Approval and thereby carries CE marking, cannot necessarily be used for the same application in all countries of European Economic Area [11].

Attestation of Conformity (AoC or AC) means those procedures, with which the conformity of (i) the manufacture of the product, (ii) the characteristics of the product, and (iii) the control of the product characteristics, is specified in accordance to the requirements in the harmonised standard or European Technical Approval.

Additionally, in many cases it is required (in the extent as required by the AoC system of the specific product), that also a third-party performs assessment, control and testing. Only an independent Notified Body (NB) having designated competence in respect to the specific product group, can act as the third-party. Such bodies may exist several for each product group, and the manufacturer is free to choose anyone of them for the surveillance contract. The manufacturer may choose any such Notified Body in any country within the European Economic Area [11]. Up-to-date lists of the designated Notified Bodies under Construction Products Directive, are published for each standard on web site of European Commission (entry: NANDO).

Table 1.9 Systems of AoC, and the tasks of different parties [11]

	Systems of Attestation of Conformity							
CONTROL MEASURES	**1+**	**1**	**2+**		**2**		**3**	**4**
Initial type-testing of the product	❑	❑	❍	❍	❍	❍	❑	❍
Testing of samples taken from factory	❍	❍	❍		❍			
Audit testing of samples taken from factory, market or site	❑							
Factory production control	❍	❍	❍	❍	❍	❍	❍	❍
Initial inspection of factory and its production control	❑	❑	❑	❑	❑	❑		
Continuous surveillance, assessment and approval of production control	❑	❑	❑	❑				
❍ = manufacturer				❑ = assessing body (so-called notified body)				

Various systems of AoC are presented in Table 1.9. As for the system of AoC of the load bearing structures, EN 1090-1 designates system 2+. This means, among other things, that a third-party is required to carry out the initial inspection of the Factory Production Control, as well as continuous surveillance [8].

Conformity assessment covers tasks of the **manufacturer** for continuous factory production control (sampling, testing frequency, analysis of results) and also corresponding tasks of the **Notified Body** (for example initial type-testing, initial inspection of the quality control of the factory, as well as continuous surveillance).

Provisions for conformity assessment are usually presented in the harmonised product standard itself, or in the therein stated reference standard. It is to be noted that in harmonised product standards or European Technical Approvals, all characteristics may not be required to be determined by testing. In addition to (or instead of) a testing method, the standard may for example comprise a table from which the quantitative value for a product characteristic may be directly taken for CE marking. Or the standard may define a calculating method, or reference to a calculating method, by which the characteristic can be determined. In case of load bearing structures and components, the standards refer to EN-Eurocode for the calculation method, or the standards present a calculating method that is based on Eurocode.

The execution standard of steel structures EN 1090-1 provides [8], that when the manufacturer declares in the CE marking also product performance values regarding structural resistance, the conformity assessment shall also cover the related design matters (design personnel, applied design tools / software).

When the provisions concerned are fulfilled and the Notified Body has drawn up the certificate underneath, the manufacturer or its representative on European Economic Area shall draw up and keep up-to-date a **Declaration of Conformity** that entitles the manufacturer to affix the CE marking to the product. The Declaration shall have the following information [11]:

- name and address of the manufacturer or its authorized representative on European Economic Area, and the place of production
- description of the component (for example type, identification data, intended use etc.) and copy of the information accompanying the CE marking
- requirements which the product fulfills (for example, Annex ZA of the EN standard concerned)
- special provisions regarding the use of the product (for example, requirements regarding its use under special terms etc.)
- the number of the accompanying factory production control certificate
- name of, and the position held by, the person who is empowered to sign the Declaration of Conformity on behalf of the manufacturer or its authorized representative.

Along with the Declaration there shall be a Certificate drawn up by the Notified Body about the internal factory production control system of the manufacturer. In addition to the information stated above, the Certificate shall have the following information:

- name and address of the Notified Body
- the number of internal factory production control certificate
- conditions and period of validity, where applicable
- name of, and position held by, the person who is empowered to sign the Certificate

By request, the manufacturer or importer of the CE marked construction product is responsible to present for a public authority the above defined Declaration and Certificate in the official languages of the Member State where the product is intended to be used. In Finland this means Finnish and Swedish [11].

1.13.3 CE marking of a structural hollow section (EN 10219-1)

Part EN 10219-1 of the standard EN 10219 is a harmonised product standard. For these products, the co-existence period has expired on 1.2.2008. That means, under Construction Products Directive, that CE marking is obligatory for these steel products (however, as presented on previous pages, note the different interpretation in certain countries like Finland). As for the system of AoC for structural hollow sections complying with EN 10219, system 2+ shall be applied.

In the CE marking of structural hollow sections complying with EN 10219, the manufacturer shall present the following information [1]:

- identification number of the Notified Body
- name or identifying mark and the registered address of the manufacturer
- the last two digits of the year in which the marking is affixed
- the number of the EC certificate of conformity, and the name or number of the Notified Body that has approved the internal factory production control system
- reference to this EN standard (SFS-EN 10219-1)
- product designation
- “No performance determined” (NPD) for characteristics where this is relevant.

CE	*CE conformity marking, consisting of the “CE”-symbol given in Directive 93/68/EEC.*
01234	*Identification number of the Notified Body*
AnyCo Ltd, PO Box 21, B-1050	*Name or identifying mark and registered address of the manufacturer*
06	*Last two digits of the year in which the marking was affixed*
01234-CPD-00234	*Certificate number*
EN 10219-1:2006	*No. of European standard*
Cold-formed steel hollow sections for building and civil engineering **Product designation: S355J2H** **Dangerous substance: NPD** **Durability: (by coating) NPD**	*Description of product and information on regulated characteristics*

Figure 1.5 Example of CE marking of a structural hollow section complying with EN 10219 [1]

1.13.4 CE marking of a structural component (EN 1090-1) made of structural hollow sections

Part EN 1090-1 of the standard EN 1090 is a harmonised product standard, for which an exceptionally long co-existence period of 1.1.2011 - 1.7.2014 has been announced (originally the co-existence period was supposed to be 1.1.2011 - 1.7.2012, that however proved to be inadequate due to certain problems in praxis).

As for the system of AoC for the load bearing structures complying with EN 1090, system 2+ shall be applied.

According to EN 1090-1, CE marking of a structural component or a sub-structure (for example a column or a roof truss), shall always include the following information [8]:

Basic data:

- identification number of the certification body (Notified Body)
- name or identifying mark and the registered address of the manufacturer
- the last two digits of the year in which the marking is affixed
- the number of the EC factory production control certificate
- reference to this EN standard (EN 1090-1)
- description of the components:
  generic name, materials, dimensions and intended use
- information of those product charasteristics related to the chosen CE marking method (Methods 1-3; these are described on next pages)
- NPD (No Performance Determined) for characteristics where this is relevant
- the execution class (EXC) of the component referring to EN 1090-2
- reference to the component specification.

EN 1090-1 defines different alternative methods to provide a CE marking. The methods differ from each other in regard to the content of the CE marking, i.e. in regard to the product characteristics that shall be declared in addition to the above stated basic data. The manufacturer shall for each case adopt a relevant method [8].

Method 1:
Declaration of product characteristics by material properties and geometric data:

The manufacturer shall specify the applied materials and the geometrical data of the product, that are needed for determining the structural resistance of the product by calculations. In addition to the information stated on previous pages under the title *Basic data*, also the following properties shall be declared [8]:

- geometrical data (tolerances in dimensions and shape)
- weldability, if needed, otherwise NPD is declared
- requirement of the minimum value of impact toughness of the used steel grade
- reaction to fire:
  to be declared that the materials are classified as Class A1,
  or;
  if a coating with organic content larger than 1 %,
  the relevant class on the basis of the organic content in accordance with EN 13501-1
  (in this context, anodizing or galvanizing is not considered as coating)
- release of cadmium and its compounds:
  NPD to be declared
- emission of radioactivity:
  NPD to be declared
- durability:
  to be declared according to component specification
- structural resistance:
  NPD to be declared
- reference to component specification
- execution class (EXC)

An example of CE marking according to Method 1 for a column, made of structural hollow section, is presented in Figure 1.6.

A unique mark shall be used to identify the component and trace it back to its component specification and manufacturing information. In the example in Figure 1.6, ‘M 101’ is used as the identification mark.

CE	*CE conformity marking, consisting of the “CE”-symbol given in Directive 93/68/EEC.*
01234	*Identification number of the Notified Body*
AnyCo Ltd, PO Box 21, B-1050	*Name or identifying mark and registered address of the manufacturer*
11	*Last two digits of the year in which the marking was affixed*
01234-CPD-00234	*Certificate number*
EN 1090-1:2006+A1:2011	*No. of European standard*
Steel column - M 101 CFRHS - EN 10219 - S355J2H - 300x300x12,5	*Description of product and information on regulated characteristics*
Tolerances on geometrical data: EN 1090-2. **Weldability: Steel S355J2H according to EN 10219-1.** **Impact toughness: 27 J at -20 ºC.** **Reaction to fire: Material classified: Class A1.** **Release of cadmium: NPD.** **Emission of radioactivity: NPD.** **Durability: Surface preparation according to EN 1090-2, preparation grade P3.** **Surface painted according to EN ISO 12944-5, paint system S.1.09.** **<u>Structural characteristics</u>:** **<u>Design</u>: NPD.** **<u>Manufacturing</u>: According to component specification CS-034/2006, and EN 1090-2, execution class EXC2.**	

Figure 1.6 Example of CE marking according to Method 1 [8]

Method 2:
Declaration of the strength value(s) of the component:

This Method is used when the manufacturer declares and is responsible also for structural properties and resistance of the component, determined according to EN-Eurocode [8].

The properties of the component can be declared in two ways [8]:
- option 2a:
  a structure designed in accordance to Eurocode for a known location, which thereby shall be declared in the CE marking
- option 2b:
  a structure designed in accordance to Eurocode, but without the knowledge of the location (the product is manufactured and sold 'from stock')

In addition to the information stated on previous pages under the titles *Basic data* and *Method 1*, also the following resistance related information, determined according to EN-Eurocode, shall be declared in the CE marking, under the title *Structural characteristics*:

- load bearing capacity
- deformation (deflection) at serviceability limit state
- fatigue resistance
- fire resistance
- reference to the design calculations

The resistances may be determined either as characteristic values (all) or as design values (all). When designing according to Eurocode, the Nationally Determined Parameters (NDP) may be chosen either as the recommended values given in Eurocode, or as the values according to the National Annex of the country where the structure is to be located.

In practice, the aforementioned data is declared in the CE marking by making reference to EN-Eurocode and to the specific design document (design brief) of the component, and by declaring about the NDP-values whether the recommended values of Eurocode or the NDP-values of the National Annex of a specific (which) Member State have been adopted.

An example of CE marking according to Method 2a for a roof truss is presented in Figure 1.7. This also means that the product has been manufactured to an identified construction project.

A unique mark shall be used to identify the component and trace it back to its component specification and manufacturing information. In the example in Figure 1.7, 'M 102' is used as the identification mark.

CE	*CE conformity marking, consisting of the "CE"-symbol given in Directive 93/68/EEC.*
01234	*Identification number of the Notified Body*
AnyCo Ltd, PO Box 21, B-1050	*Name or identifying mark and registered address of the manufacturer*
11	*Last two digits of the year in which the marking was affixed*
01234-CPD-00234	*Certificate number*
EN 1090-1:2006+A1:2011	*No. of European standard*
Roof trusses in steel, to be used in Berlin library - M 102	*Description of product and information on regulated characteristics*
Tolerances on geometrical data: EN 1090-2. **Weldability: Steel S355J2H according to EN 10219-1.** **Impact toughness: 27 J at -20 ºC.** **Reaction to fire: Material classified: Class A1.** **Release of cadmium: NPD.** **Emission of radioactivity: NPD.** **Durability: Surface preparation according to EN 1090-2, preparation grade P3.** **Surface painted according to EN ISO 12944, paint system S.1.09.** **Structural characteristics:** **Load bearing capacity: Design according to EN 1993-1, see accompanying design brief and design calculations. NDPs for Germany apply. Reference: DC 102/3.** **Deformation at serviceability limit state: NPD** **Fatigue strength: NPD.** **Resistance to fire: Calculated value: R30, see DC 102/3.** **Manufacturing: According to component specification CS-016/2006, and EN 1090-2, execution class EXC3.**	

Figure 1.7 Example of CE marking according to Method 2a [8]

Method 3a:
Declaration of conformity based on component specification given to the manufacturer:

This Method is used when the component is been designed by others than the manufacturer. This also thereby means, that the design process and structural calculations are not covered by the CE marking.

Requirements to the manufacturing of the component are identified by the component specification which is based on information from the design of the component. The component specification is been prepared by the purchaser, or by the purchaser in cooperation with the manufacturer [8].

In addition to the information stated on previous pages under the titles *Basic data* and *Method 1*, also the following information shall be declared in the CE marking under the title *Structural characteristics*:

- reference to the design made by other parties (purchaser)

An example of CE marking according to Method 3a is presented in Figure 1.8.

A unique mark shall be used to identify the component and trace it back to its component specification and manufacturing information. In the example in Figure 1.8, 'M 103' is used as the identification mark.

CE	*CE conformity marking, consisting of the "CE"-symbol given in Directive 93/68/EEC.*
01234	*Identification number of the Notified Body*
AnyCo Ltd, PO Box 21, B-1050	*Name or identifying mark and registered address of the manufacturer*
11	*Last two digits of the year in which the marking was affixed*
01234-CPD-00234	*Certificate number*
EN 1090-1:2006+A1:2011	*No. of European standard*
Steel column, to be used in the National Theatre of Luxembourg City - M 103 CFRHS - EN 10219 - S355J2H - 300x300x12,5 **Tolerances on geometrical data: EN 1090-2.** **Weldability: Steel S355J2H according to EN 10219-1.** **Impact toughness: 27 J at -20 ºC.** **Reaction to fire: Material classified: Class A1.** **Release of cadmium: NPD.** **Emission of radioactivity: NPD.** **Durability: Surface preparation according to EN 1090-2, preparation grade P3.** **Surface painted according to EN ISO 12944, paint system S.1.09.** **Structural characteristics:** **Design: Provided by purchaser, doc. Ref. no 123.** **Manufacturing: According to component specification CS-017, and EN 1090-2, execution class EXC3.**	*Description of product and information on regulated characteristics*

Figure 1.8 Example of CE marking according to Method 3a [8]

Method 3b:
Declaration of the strength value(s) of the component based on purchaser's order

This Method is used when the manufacturer is responsible also for the structural design of the component (based on the initial data / requirements given by the purchaser), complying with the regulations (other than EN-Eurocode) valid in the country where the structure is to be located [8].

<u>In addition</u> to the information stated on previous pages under the titles *Basic data* and *Method 1*, also the following information shall be declared in the CE marking under the title *Structural characteristics*:

- design brief, standards and any other design specifications
- load bearing capacity
- fatigue resistance
- fire resistance
- reference to the design calculations

The resistances may be determined either as characteristic values (all) or as design values (all) with the definitions for those terms given in the relevant design provisions.

In practice, the aforementioned data is declared in the CE marking by making reference to the relevant design provisions (design codes) and to the specific design document (design brief) of the component.

An example of CE marking according to Method 3b is presented in Figure 1.9.

A unique mark shall be used to identify the component and trace it back to its component specification and manufacturing information. In the example in Figure 1.9, 'M 104' is used as the identification mark.

CE	*CE conformity marking, consisting of the “CE”-symbol given in Directive 93/68/EEC.*
01234	*Identification number of the Notified Body*
AnyCo Ltd, PO Box 21, B-1050	*Name or identifying mark and registered address of the manufacturer*
11	*Last two digits of the year in which the marking was affixed*
01234-CPD-00234	*Certificate number*
EN 1090-1:2006+A1:2011	*No. of European standard*
4 steel lattices for bridge Bergen - M 104 **Tolerances on geometrical data: EN 1090-2.** **Weldability: Steel S355J2H according to EN 10219-1.** **Impact toughness: 27 J at -20 ºC.** **Reaction to fire: Material classified: Class A1.** **Release of cadmium: NPD.** **Emission of radioactivity: NPD.** **Durability: Surface preparation according to EN 1090-2, preparation grade P3. Surface painted according to EN ISO 12944, see component specification for details.** **<u>Structural characteristics</u>:** **<u>Load bearing capacity</u>: Design according to NS 3472 and specification RW 302 from the Railway administration, see accompanying design brief and design calculations, DC 501/06.** **<u>Deformation at serviceability limit state</u>: See accompanying design brief and design calculations, DC 501/06.** **<u>Fatigue strength</u>: RW 302.** **<u>Resistance to fire</u>: NPD.** **<u>Manufacturing</u>: According to component specification CS-506/2006, and EN 1090-2, execution class EXC3.**	*Description of product and information on regulated characteristics*

Figure 1.9 Example of CE marking according to Method 3b [8]

1.14 References

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2. RESISTANCE OF HOLLOW SECTION STRUCTURES

2.1 Limit state design and partial safety factors

The resistance of a structure means its ability to bear the loads it is subjected to without failure or excessive deformation. Resistance and load vary according to time and location. Thus, they do not have a single absolute value, but their values are distributed according to statistic probability. In design, the dispersion of resistance and load must be taken into account by using partial safety factors.

The Examples in this handbook have been calculated using recommended values as given in EN-Eurocode, if not otherwise mentioned. The national values must be checked from the National Annex of the relevant country.

The design rules given in Eurocode may be regarded as complex when compared with the conventional national rules. It is good to remember though, that the designer is always allowed to make such kind of simplifications, which lead to the safe side.

The general format of the design condition for the Ultimate Limit State (ULS) is:

$$E_d \leq R_d \quad \Leftrightarrow \quad \gamma_F \cdot E_k \leq \frac{R_k}{\gamma_M} \tag{2.1}$$

where γ_F is the partial safety factor for <u>load</u>
γ_M is the partial safety factor for <u>resistance</u>
E_d is the design value of the force or moment caused by the load
E_k is the characteristic value of the force or moment caused by the load
R_d is the design value of resistance
R_k is the characteristic value of resistance

The design condition in expression (2.1) can be written also in the following form which shows directly the utilisation ratio of the resistance considered:

$$\frac{E_d}{R_d} \leq 1,0 \tag{2.2}$$

The design value R_d of resistance is usually presented in the form R_k/γ_M where R_k is the characteristic value of resistance and γ_M is the partial safety factor for resistance. The partial safety factor γ_M can obtain variable values as presented later on.

The design values F_d of the single loads are obtained by multiplying the characteristic values F_k of the loads by partial safety factors γ_F for the loads. The characteristic values of the loads are defined in various Parts of Eurocode 1 (EN 1991).

In practice, the design values of the loads have to be determined as a load combination of the simultaneously acting loads.

At ultimate limit state, in usual situations without accidental loads, the load combination can be determined by following expression [1,1a]:

$$\sum_{j \geq 1} \gamma_{G.j} G_{k.j} \text{ "+" } \gamma_{Q.1} Q_{k.1} \text{ "+" } \sum_{i > 1} \gamma_{Q.i} \psi_{0.i} Q_{k.i} \qquad (2.3)$$

or alternatively by following expressions, from which always the more unfavourable is selected:

$$\sum_{j \geq 1} \gamma_{G.j} G_{k.j} \text{ "+" } \gamma_{Q.1} \psi_{0.1} Q_{k.1} \text{ "+" } \sum_{i > 1} \gamma_{Q.i} \psi_{0.i} Q_{k.i} \qquad (2.4a)$$

$$\sum_{j \geq 1} \xi_j \gamma_{G.j} G_{k.j} \text{ "+" } \gamma_{Q.1} Q_{k.1} \text{ "+" } \sum_{i > 1} \gamma_{Q.i} \psi_{0.i} Q_{k.i} \qquad (2.4b)$$

where "+" means "to be combined with"
(i.e. simultaneous action of the loads)
j is the index for permanent load
i is the index for variable load
$G_{k.j}$ is the characteristic value of permanent load
$Q_{k.1}$ is the characteristic value of the leading variable load
$Q_{k.i}$ is the characteristic value of other variable load
$\gamma_{G.j}$ is the partial safety factor for permanent load (Table 2.1)
$\gamma_{Q.1}$ is the partial safety factor for the leading variable load (Table 2.1)
$\gamma_{Q.i}$ is the partial safety factor for other variable load (Table 2.1)
$\psi_{0.1}$ is the combination factor for the leading variable load (Table 2.4)
$\psi_{0.i}$ is the combination factor for other variable load (Table 2.4)
ξ_j is the reduction factor for unfavourable permanent load (Table 2.1).

Expressions (2.4a) and (2.4b) have been taken into use because expression (2.3) is overly conservative for heavy structures. Expressions (2.4a) and (2.4b) lead in steel structures usually to smaller loads than expression (2.3).

The National Annex will define whether expression (2.3) or expressions (2.4a) and (2.4b) shall be used.

Finnish National Annex to standard EN 1990 [2]:

Expressions (2.4a) and (2.4b) are used as well as the partial safety factors and combination factors presented in Tables 2.1 and 2.4. In expression (2.4a) only the permanent loads are taken into account.

For fire design, the load combinations are presented in Chapter 6.

For Serviceability Limit State (SLS), the load combinations and deflection limits are presented in Chapter 7.

For fatigue design, the load combinations are defined in those Parts of Eurocode that cover fatigue loaded structures, such as EN 1993-2 (Bridges) and EN 1993-6 (Crane supporting structures). Fatigue design in respect to the resistance of the structure is presented in Chapter 4 of this handbook.

Load factor K_{FI} which appears together with partial safety factors in Table 2.1, is applied only at ultimate limit state for load combinations in persistent or transient design situations, and therein only for unfavourable loads as presented in Table 2.1. The factor is not applied in fire design, fatigue design or in serviceability limit state. The value of the factor depends on the reliability class (RC1-RC3) of the structure or the member, as presented in Table 2.2. The reliability class on its behalf depends on the consequences class (CC1-CC3) of the structure, presented in the same table. The consequences class depends on the severity of the possible collapse of the structure. The consequences class and reliability class of a single member may be higher or lower than the classification of the other part of the structure [1,1a,2].

Table 2.1 Buildings, partial safety factors for loads [1,1a,2,22...25]

Design case	Permanent loads (γ_G)	Variable loads (γ_Q) a)	
		Leading variable load	Other variable loads
The effect of the load is unfavourable	EN 1990 c) : 1,35xK_{FI} Finland c) : 1,35xK_{FI}	EN 1990 : 1,50xK_{FI} Finland b) : 1,50xK_{FI}	EN 1990 : 1,50xK_{FI} Finland b) : 1,50xK_{FI}
The effect of the load is favourable	EN 1990 : 1,0 Finland : 0,9	EN 1990 : 0 Finland : 0 d)	EN 1990 : 0 Finland : 0 d)
Fatigue design	EN 1993-1-9 : 1,0 Finland : 1,0	EN 1993-1-9 : 1,0 Finland : 1,0	EN 1993-1-9 : 1,0 Finland : 1,0
Fire design	EN 1990 : 1,0 Finland : 1,0	EN 1990 : 1,0 Finland : 1,0	EN 1990 : 1,0 Finland : 1,0
Serviceability limit state	EN 1990 : 1,0 Finland : 1,0	EN 1990 : 1,0 Finland : 1,0	EN 1990 : 1,0 Finland : 1,0

a) The loads presented in Table 2.4 are variable loads
b) In expression (2.4a) only permanent loads are taken into account in Finland
c) In expression (2.4b) a reduction factor ξ = 0,85 is used (so that $\xi \gamma_G K_{FI} = 0{,}85 \times 1{,}35 \times K_{FI} \approx 1{,}15 \times K_{FI}$)
d) This value is not presented in the Finnish National Annex, but this is the intended value

In this table the partial factors are presented according to Eurocode (EN 1990 and EN 1993) and Finnish National Annex. **The values valid in other countries must be checked from the National Annex of the relevant country.**

Table 2.2 Values of load factor K_{FI} in different reliability classes [1,1a,2]

Consequences class	Reliability class	Load factor K_{FI}
CC3	RC3	EN 1990 : 1,1 Finland : 1,1
CC2	RC2	EN 1990 : 1,0 Finland : 1,0
CC1	RC1	EN 1990 : 0,9 Finland : 0,9

In this table the values of load factor K_{FI} are presented according to Eurocode (EN 1990) and Finnish National Annex. **The values valid in other countries must be checked from the** National Annex **of the relevant country.**

Table 2.3 Definition of consequences classes and reliability classes [1,1a,2]

Consequences class	Reliability class	Description	Examples of buildings and civil engineering works
CC3	RC3	EN 1990 and Finland: **High** consequence for loss of human life *or* economic, social or environmental consequence **very great**	EN 1990: Grandstands, public buildings where consequences of failure are high (e.g. a concert hall) Finland: The load bearing system [a)] with its bracing parts in building which are often occupied by a large number of people for example: - residential, office and business buildings with more than 8 storeys [b)] - concert halls, theatres, sports and exhibition halls, spectator stands - heavily loaded buildings with long spans Special structures such as high masts and towers. [a)] Roofs and floors are however in class CC2 if they do not form a part of the stiffening system of the whole structure. [b)] Underground floors included.
CC2	RC2	EN 1990 and Finland: **Medium** consequence for loss of human life *or* economic, social or environmental consequences **considerable**	EN 1990: Residential and office buildings, public buildings where consequences of failure are medium (e.g. an office building) Finland: Buildings and structures not belonging to classes CC3 or CC1.
CC1	RC1	EN 1990 and Finland: **Low** consequence for loss of human life**,** *and* economic, social or environmental consequences **small or negligible**	EN 1990: Agricultural buildings where people do not normally enter, greenhouses Finland: Structures, which when damaged, don't pose major risk.
Examples of structures belonging to different consequences classes (reliability classes) are presented in more details in Eurocode Parts EN 1990 and EN 1991-1-7 as well as in their National Annexes.			
In this table the definitions of consequences classes and reliability classes are presented according to Eurocode (EN 1990) and Finnish National Annex. **The definitions valid in other countries must be checked from the National Annex of the relevant country.**			

Table 2.4 Buildings, combination factors for loads [1,1a,2]

Load	EN 1990			Finland		
	ψ_0	ψ_1	ψ_2	ψ_0	ψ_1	ψ_2
Imposed loads in buildings (see EN 1991-1-1)						
- Category A: residential areas	0,7	0,5	0,3	0,7	0,5	0,3
- Category B: office areas	0,7	0,5	0,3	0,7	0,5	0,3
- Category C: congregation areas	0,7	0,7	0,6	0,7	0,7	0,3
- Category D: shopping areas	0,7	0,7	0,6	0,7	0,7	0,6
- Category E: storage areas	1,0	0,9	0,8	1,0	0,9	0,8
- Category F: traffic areas, vehicle weight ≤ 30 kN	0,7	0,7	0,6	0,7	0,7	0,6
- Category G: traffic areas, 30 kN < vehicle weight ≤ 160 kN	0,7	0,5	0,3	0,7	0,5	0,3
- Category H: roofs	0	0	0	0	0	0
Snow loads on buildings (see EN 1991-1-3) a)				b)		
- Finland, Iceland, Norway, Sweden	0,7	0,5	0,2	0,7 c) 0,7 d)	0,4 c) 0,5 d)	0,2 c) 0,2 d)
- Other CEN Member States, when the altitude is H > 1000 m above the sea level	0,7	0,5	0,2			
- Other CEN Member States, when the altitude is H ≤ 1000 m above the sea level	0,5	0,2	0			
Ice load e)				0,7	0,3	0
Wind loads on buildings (see EN 1991-1-4)	0,6	0,2	0	0,6	0,2	0
Temperature (non-fire) in buildings (see EN 1991-1-5)	0,6	0,5	0	0,6	0,5	0

a) For countries not mentioned below, see relevant local conditions.
b) Finland: Snow load on outdoor terraces and balconies: ψ_0 = 0 in connection with categories A, B, F and G
c) Finland: when s_k < 2,75 kN/m^2
d) Finland: when $s_k \geq$ 2,75 kN/m^2
e) Added to the Finnish National Annex.

In this table the combination factors are presented according to Eurocode (EN 1990) and Finnish National Annex. **The values valid in other countries must be checked from the National Annex of the relevant country**.

Eurocode 3 defines the partial safety factors for resistance as well as the designations and recommended numerical values for them separately in each Part of Eurocode. In respect of buildings, the most essential Parts of Eurocode and the partial safety factors given in them are presented in Table 2.5. The presented values are the recommended values of Eurocode. **The values valid in each country must be checked from the National Annex of the relevant country.** In the table also the values valid in Finland are presented.

It can be seen in Table 2.5 that although the definitions, designations and numerical values in different Parts of Eurocode 3 are generally the same, there do exist also some differences. For example, EN 1993-1-1 uses designation γ_{M2} as the partial safety factor for tension resistance of net section, whereas EN 1993-1-8 uses the same symbol as the partial safety factor for the joints presented in the table. Also the numerical values may differ in different Parts of Eurocode: EN 1993-1-1 gives a recommended value 1,0 for the partial safety factor γ_{M1} to stability of a member, whereas EN 1993-2 (Bridges) gives a recommended value 1,1. <u>The partial safety factors to be used shall always be chosen according to the context and according to the Part of Eurocode to be applied.</u>

Table 2.5 Partial safety factors for resistance [3...25]

Standard	Design case	Partial factor	Value of the partial factor	
			Recommen-ded value of Eurocode	Finland
EN 1993-1-1:	General rules for steel structures:			
	Resistance of cross-section (whatever the cross-section Class is), also for local buckling and distortional buckling. **Note: Class 4 circular hollow sections: see EN 1993-1-6**	γ_{M0}	1,0	1,0
	Resistance of a member to instability, when the calculations are made as a member check.	γ_{M1}	1,0	1,0
	Resistance of net cross-section in tension to fracture (holes deducted from the gross cross-section)	γ_{M2}	1,25	1,25
	Resistance of joints	see EN 1993-1-8		
EN 1993-1-2:	Structural fire design:			
	Resistance in fire design	$\gamma_{M.fi}$	1,0	1,0
EN 1993-1-3:	Supplementary rules for cold-formed members and sheeting:			
	Partial safety factors γ_{M0} and γ_{M1} and γ_{M2}: as for EN 1993-1-1 above			
EN 1993-1-5:	Plated structural elements:			
	Partial safety factors γ_{M0} and γ_{M1} are chosen according to the applied Part of Eurocode (EN 1993-1-1...EN 1993-6)			
EN 1993-1-6:	Shell structures:			
	Resistance to local buckling	γ_{M1}	1,1	1,1
EN 1993-1-8:	Joints:			
	Resistance of members and cross-sections	see EN 1993-1-1		
	Resistance of bolts			
	Resistance of rivets			
	Resistance of pins	γ_{M2}	1,25	1,25
	Resistance of welds			
	Resistance of plates in bearing			
	Slip resistance - at ultimate limit state (Category C) - at serviceability limit state (Category B)	 γ_{M3} $\gamma_{M3.ser}$	 1,25 1,1	 1,25 1,1
	Bearing resistance of an injection bolt	γ_{M4}	1,0	1,0
	Resistance of joints of structural hollow sections in lattice structures	γ_{M5}	1,0	1,0
	Resistance of pins at serviceability limit state	$\gamma_{M6.ser}$	1,0	1,0
	Preload of high strength bolts	γ_{M7}	1,1	1,1
EN 1993-1-9:	Fatigue:			
	'Damage tolerant' principle - low consequences of failure - high consequences of failure	 γ_{Mf} γ_{Mf}	 1,0 1,15	 1,0 1,15
	'Safe life' principle - low consequences of failure - high consequences of failure	 γ_{Mf} γ_{Mf}	 1,15 1,35	 1,15 1,35
In this table the partial safety factors for resistance are presented according to the above mentioned Parts of Eurocode 3 (EN 1993) and the corresponding Parts of Finnish National Annex. **In other Parts of Eurocode 3 other values may be presented for the partial safety factors of resistance. Partial safety factors must always be chosen according to the applied Part of Eurocode and the National Annex of the relevant country.**				

2.2 Co-ordinate system of a member

The co-ordinate system of a member is according to Figure 2.1.

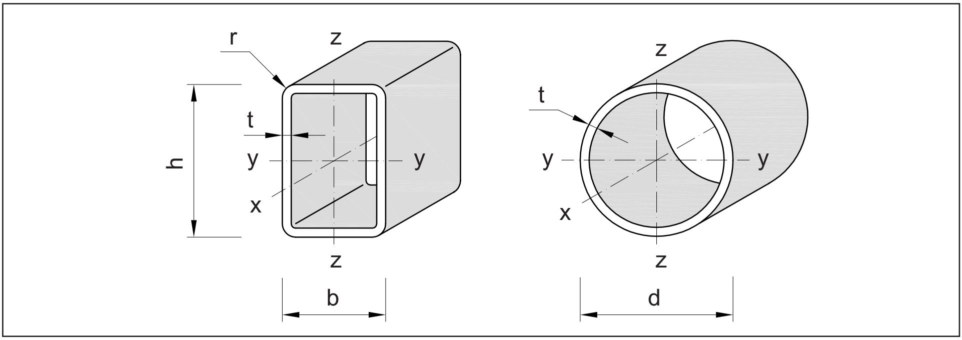


Figure 2.1 Co-ordinate system of a member and designations for the cross-section's dimensions

2.3 Classification of cross-sections

The role of cross-section classification is to identify in which extent local buckling of the cross-section limits its resistance and rotation capacity.

Cross-sections are divided into four Classes (Table 2.6). A single structure may contain structural members with cross-sections of different Classes. Different parts of a single cross-section (flanges and webs) may also belong to different Classes. The Class depends on the width-to-thickness ratio and the stress state of the compressed parts. The compressed part of the cross-section includes every part where the load combination to be considered causes either full or partial compression. The Class of a single structural hollow section may be different in bending than in compression. The Class may vary in the longitudinal direction of the member depending on the ratios between bending moment and normal force [3,4,5].

Class 1: The whole cross-section is effective. The cross-section can form a plastic hinge with plastic bending resistance with the rotation capacity needed for plastic theory.

Class 2: The whole cross-section is effective. The cross-section can form a plastic hinge with plastic bending resistance, but rotation capacity is limited due to local buckling.

Class 3: The whole cross-section is effective. In a bended cross-section the compressive stress can reach the yield strength in the extreme fibre of the cross-section. Local buckling takes place before the internal bending moment has reached the plastic bending resistance. In a uniformly compressed cross-section the whole cross-section can reach the yield strength. (Note: In calculations the stress maximum is in praxis limited to the design value of the yield strength $f_{yd} = f_y / \gamma_{M0}$.)

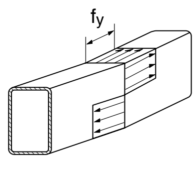
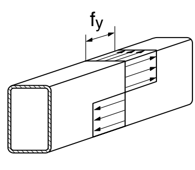
Class 4: Only part of the cross-section is effective. In the cross-section local buckling takes place before the maximum compressive stress in some plane element has reached the yield strength. The calculation of bending resistance and compression resistance is based on the effective cross-section. The resistances of the cross-section are then calculated according to only the effective areas of the elements. When calculating the resistance for Class 4 circular hollow sections, effective cross-section cannot be used, but the calculation is based on the full cross-section (gross cross-section) with respect to local buckling stress of a cylindrical shell [3,4,5,16].

The forces, moments and resistances of the structure can be calculated in all Classes according to theory of elasticity, if the effect of local buckling to the resistance of the cross-section is taken into account. Theory of plasticity can be used to calculate resistances in Class 1 and 2, and forces and moments in Class 1. In practice, for simplicity, the forces and moments can be calculated according to the most governing (i.e. highest) Class [3,4,5].

As a relief to the aforementioned requirements, in case of continuous beams in Class 2, EN 1993-1-1 gives the possibility to utilise the redistribution of moments (so-called levelling of moments) according to theory of plasticity by modifying the bending moments calculated according to theory of elasticity at highest 15 % (as calculated from the peak moment) provided that:

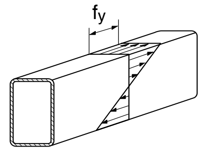
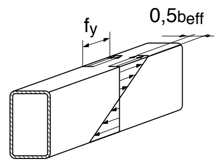
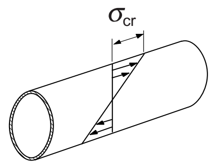
- the internal forces and moments of the frame remain in equilibrium with the external loads and
- all members where the moments are reduced belong to Class 1 or 2 and
- lateral-torsional buckling of the members is prevented.

Table 2.6 Design methods in different cross-section Classes [3,4,5,16]

Cross-section Class	Method for calculating resistance	Method for calculating forces and moments	Stress distribution when resistance is reached
1 square, rectangular and circular hollow sections	theory of plasticity	theory of plasticity	
2 square, rectangular and circular hollow sections	theory of plasticity	theory of elasticity	

(continues)

Table 2.6 Design methods in different cross-section Classes [3,4,5,16] *(continued)*

Cross-section Class	Method for calculating resistance	Method for calculating forces and moments	Stress distribution when resistance is reached
3 square, rectangular and circular hollow sections	theory of elasticity	theory of elasticity	
4 square and rectangular hollow sections	effective cross-section	theory of elasticity	
4 circular hollow sections	local buckling stress of gross cross-section	theory of elasticity	

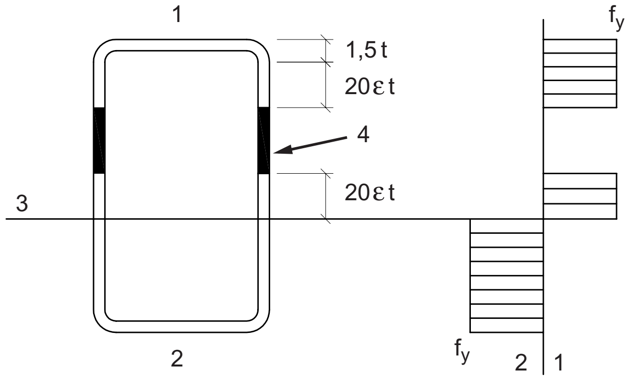
The entire cross-section is usually classified on the basis of its compressed part into the most governing (i.e. highest) Class. Alternatively the Class can be specified for the flange and web separately. If the web is designed for shear forces only and the web is not designed for bending and normal force, the cross-section may be classified into Class 2, 3 or 4 on the basis of classification of the flanges alone [13,14].

In Tables 2.7 and 2.8 the limits for different cross-section Classes are presented according to EN 1993-1-1. The limits for Class 3 calculated according to EN 1993-1-5 differ slightly from the limits presented in EN 1993-1-1 (see Table 2.7). The difference is a consequence of incomplete harmonisation between different Parts of Eurocode [26,27].

The determination of the Class is favourable to perform ‘from top to bottom’ such that first the conditions for Class 1 are checked, then the conditions for Class 2 etc. This way the limit for Class 3 will be checked according to EN 1993-1-1, which is slightly more favourable than the limit calculated according to EN 1993-1-5. The cross-sections that do not meet the requirements of Class 3, belong to Class 4.

When checking the conditions for Classes 1 or 2, the distribution of stresses and the location of neutral axis (plastic neutral axis) are assumed to be according to theory of plasticity. Only the final result of the cross-section verification shows, whether the cross-section has deformation capacity high enough to develop the plastic stress distribution that was initially assumed. Determination of the location of plastic neutral axis is presented in clause 2.9.1.5.1.

A cross-section having its web in Class 3 and the compressed flange in Class 1 or 2, can be classified into Class 2 using effective web (so-called Class 2 effective cross-section). In this case the depth of the compressed portion of the web is chosen to be $20\varepsilon t$ under the compressed flange, and $20\varepsilon t$ above the plastic neutral axis of the effective cross-section, as presented in Figure 2.2 [3,4,5].



1 Compression
2 Tension
3 Plastic neutral axis of the effective cross-section
4 Non-effective part of the web to be neglected

Figure 2.2 Effective cross-section in Class 2 [3,4,5]

The limits for Class 3 given in the expressions of Tables 2.7 and 2.8 can be eased by replacing the factor ε by factor ε^*, except when calculating the buckling resistance of a member [3,4,5]:

$$\varepsilon^* = \sqrt{\frac{f_y/\gamma_{M0}}{\sigma_{com.Ed}}} \cdot \varepsilon = \sqrt{\frac{235}{\gamma_{M0} \cdot \sigma_{com.Ed}}} \qquad (2.5)$$

where $\varepsilon = \sqrt{235/f_y} \quad [f_y] = N/mm^2$

f_y is the nominal yield strength of the material

γ_{M0} is the partial safety factor for resistance (Table 2.5)

$\sigma_{com.Ed}$ is the design value of maximum compressive stress in the considered plane element

This allowance is favourable when the maximum compressive stress of the considered plane element falls under the yield level. The maximum allowable value of the compressive stress is commonly $\sigma_{com.Ed} = f_y / \gamma_{M0}$, in which case expression (2.5) is returned back to the original expression of factor ε presented in Tables 2.7 and 2.8.

When calculating the buckling resistance of a member, the aforementioned allowance shall not be done, but the limits for Class 3 are determined according to the original expressions given in Tables 2.7 and 2.8.

EN 1993-1-1 gives also provisions how to determine the cross-section Class for a web or flange subject to simultaneous compression and bending (Tables 2.7 and 2.8). However, for calculating the resistance of a member in simultaneous compression and bending, unambiguous instructions are not given. As an interpretation it has been concluded, that the cross-section Class can be determined for pure compression and for pure bending separately, whereafter the resistances are calculated respectively for pure compression and for pure bending separately, and then applied in the interaction formulae. Alternatively, when verifying the interaction, the cross-section Class and one effective cross-section are determined using the real (resulting) stress distribution calculated from the simultaneously acting forces and moments. This way the resistances to be applied in the interaction formulae will be based on one cross-section and one Class. However, if using this procedure in case of Class 4, the additional moment ($\Delta M = N_{Ed} \times e_N$) resulting from the shift of the neutral axis location, changes the stress distribution in the cross-section causing the need to calculate a new effective cross-section iteratively [13,14].

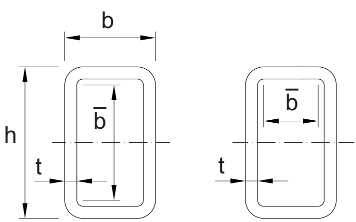
The limits of Classes 1 and 2 for a web or flange subject to simultaneous compression and bending, depend on factor α which represents the compressed part of the considered plane element (Table 2.7). In this case the plastic stress distribution of the cross-section is determined in relation to its plastic neutral axis. Determination of the location of plastic neutral axis is presented in clause 2.9.1.5.1.

When having simultaneous compression and bending, if the relative portion of compression is very large and bending small, the situation approaches uniform compression in regard to stress distribution. In this case the results obtained as mathematical limits (when $\alpha \rightarrow 1,0$) from the expressions given in Table 2.7 for the plane elements subject to simultaneous compression and bending, are the same results as presented for limits of purely compressed Class 1 and 2 plate elements. The corresponding result can be obtained from Table 2.7 also for Class 3.

The limits of cross-section Classes for different steel grades of structural hollow sections are obtained from Table 2.9. The limits given for uniformly compressed Classes 1...3 can be applied as conservative limits for a cross-section subject to simultaneous compression and bending.

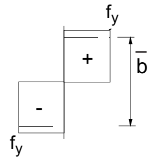
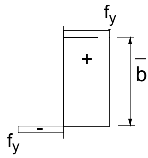
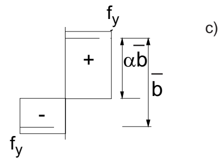
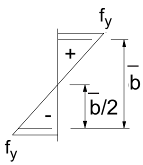
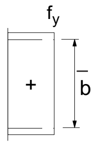
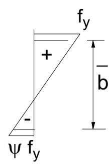
Table 2.7 Limits for cross-section Classes.
Square and rectangular hollow sections [3,4,5]

Internal compression parts (= compressed plane elements restrained on two edges)



$\bar{b} = h - 3t$ or $b - 3t$ a)

Axis of bending

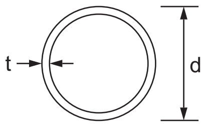
Class	Part subject to bending	Part subject to compression	Part subject to bending and compression b)
Stress distribution in parts (compression positive)			
1	$\bar{b}/t \le 72\varepsilon$	$\bar{b}/t \le 33\varepsilon$	when $\alpha > 0,5$: $\bar{b}/t \le \dfrac{396\varepsilon}{13\alpha - 1}$ when $\alpha \le 0,5$: $\bar{b}/t \le \dfrac{36\varepsilon}{\alpha}$
2	$\bar{b}/t \le 83\varepsilon$	$\bar{b}/t \le 38\varepsilon$	when $\alpha > 0,5$: $\bar{b}/t \le \dfrac{456\varepsilon}{13\alpha - 1}$ when $\alpha \le 0,5$: $\bar{b}/t \le \dfrac{41,5\varepsilon}{\alpha}$
Stress distribution in parts (compression positive)			
3	$\bar{b}/t \le 124\varepsilon$ d)	$\bar{b}/t \le 42\varepsilon$ e)	when $\psi > -1$: $\bar{b}/t \le \dfrac{42\varepsilon}{0,67 + 0,33\psi}$ f) when $\psi \le -1$ g): $\bar{b}/t \le 62\varepsilon(1-\psi)\sqrt{(-\psi)}$

$\varepsilon = \sqrt{235/f_y}$	f_y	235	275	**355**	**420**	460
	ε	1,0000	0,9244	**0,8136**	**0,7480**	0,7148

a) The calculatory width $\bar{b}$ drawn in the picture has not been determined unambiguously in EN 1993-1-1. The herein presented definition according to expression given in EN 1993-1-5 is recommended.
b) A conservative estimate is obtained when the cross-section Class is determined acc. to pure compression.
c) Determination of depth for the compressed part (factor α), see clause 2.9.1.5.1
d) If calculated according to EN 1993-1-5 the limit value will be: $\bar{b}/t \le 121,43\varepsilon$
e) If calculated according to EN 1993-1-5 the limit value will be: $\bar{b}/t \le 38,25\varepsilon$
f) According to EN 1993-1-5 the limit value depends on the local buckling factor k_σ and stress ratio ψ
g) Case $\psi \le -1$ applies, where either the compression stress $\sigma < f_y$ or the tensile strain $\varepsilon_y > f_y/E$

Table 2.8 Limits for cross-section Classes.
Circular hollow sections [3,4,5]

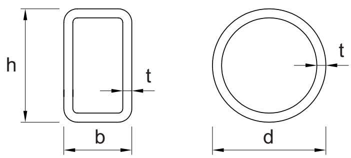
Circular hollow section

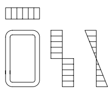
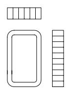


Class	Cross-section in bending and / or compression
1	$d/t \leq 50\varepsilon^2$
2	$d/t \leq 70\varepsilon^2$
3	$d/t \leq 90\varepsilon^2$ **Note.** For $d/t > 90\varepsilon^2$, see EN 1993-1-6.

$\varepsilon = \sqrt{235/f_y}$	f_y	235	275	**355**	**420**	460
	ε	1,0000	0,9244	**0,8136**	**0,7480**	0,7148
	ε^2	1,0000	0,8545	**0,6620**	**0,5595**	0,5109

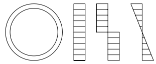
Table 2.9 Limits for cross-section Classes for grades S235-S460



Load case			Bending			Compression a)		
								
Class			1	2	3	1	2	3
f_y (N/mm^2)								
235	b / t h / t	(flange) (web)	36,0 75,0	41,0 86,0	45,0 127,0	36,0 36,0	41,0 41,0	45,0 45,0
275	b / t h / t	(flange) (web)	33,5 69,6	38,1 79,7	41,8 117,6	33,5 33,5	38,1 38,1	41,8 41,8
355	**b / t** **h / t**	**(flange)** **(web)**	**29,8** **61,6**	**33,9** **70,5**	**37,2** **103,9**	**29,8** **29,8**	**33,9** **33,9**	**37,2** **37,2**
420	**b / t** **h / t**	**(flange)** **(web)**	**27,7** **56,9**	**31,4** **65,1**	**34,4** **95,8**	**27,7** **27,7**	**31,4** **31,4**	**34,4** **34,4**
460	b / t h / t	(flange) (web)	26,6 54,5	30,2 62,3	33,0 91,6	26,6 26,6	30,2 30,2	33,0 33,0

(continues)

Table 2.9 Limits for cross-section Classes for grades S235-S460
(continued)

Load case	Bending or compession or simultaneous bending and compression		
			
Class	1	2	3
f_y (N/mm^2)			
235 d / t (entire cross-section)	50,0	70,0	90,0
275 d / t (entire cross-section)	42,7	59,8	76,9
355 **d / t** (entire cross-section)	**33,1**	**46,3**	**59,6**
420 **d / t** (entire cross-section)	**28,0**	**39,2**	**50,4**
460 d / t (entire cross-section)	25,5	35,8	46,0
a) The limit values for different cross-section Classes given for uniformly compressed cross-section can be applied for web and flange of cross-section as conservative limits of that Class also in case of simultaneous compression and bending. The exact limits of Classes for a web simultaneously in compression and bending can be determined according to Table 2.7.			

2.4 Effective cross-section in Class 4 (square and rectangular hollow sections)

For square and rectangular Class 4 hollow sections, local buckling is taken into account using effective widths of the compressed parts, which then form the effective cross-section (Figure 2.3). The effective properties of the cross-section (A_{eff}, I_{eff}, W_{eff}) are determined by the effective cross-section formed this way. The effective widths are calculated from Table 2.10. The reduction factor ρ needed in the table is calculated according to clauses 2.4.1 and 2.4.2. When determining the effective width of the flange, the stress ratio ψ is calculated on the basis of the properties of the gross cross-section. When determining the effective width of the web, the stress ratio is calculated on the basis of the effective width of the compressed flange and gross cross-section of the web.

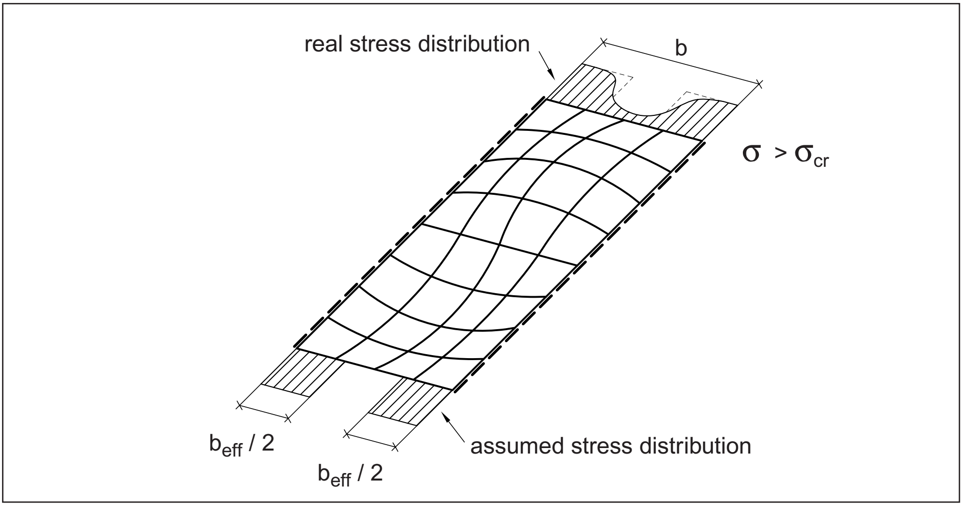


Figure 2.3 Schematic picture of effective width method, when local buckling takes place in case of uniformly compressed (ψ = 1) plane element restrained on two edges

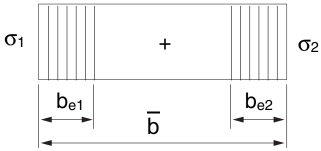
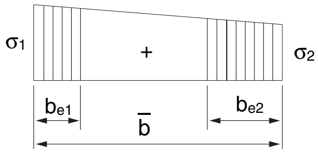
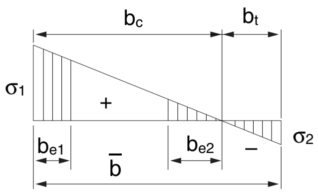
In Class 4 local buckling can be taken into account, instead of effective width, alternatively by using method basing on reduced stress presented in EN 1993-1-5. The method basing on reduced stress is equivalent with the effective width method in case of single plane elements, but not necessarily in relation to the entire cross-section, because in the method of reduced stresses the load distribution between the plane elements is not taken into account. The method of reduced stresses is not presented in this handbook, because therein the stress limits of the weakest part of the cross-section may govern the resistance of the whole cross-section, and because the effective width method allows the use of more slender plane elements [13,14,28].

In global analysis (i.e. when calculating the internal forces and moments of the structure) the influence of local buckling of the plane elements on the stiffness may be neglected when the effective cross-section of the compressed part is greater than ρ_{lim} times gross cross-sectional area of the same part [13,14]. The value for the aforementioned limit can be defined in National Annex. The recommended value given in EN 1993-1-5 is $\rho_{lim} = 0{,}5$. If the plane element is partially in compression and partially in tension, like a web subject to bending, the rule is applied to the compressed part [28].

Finnish National Annex to standard EN 1993-1-5 [15]:
The recommended value of Eurocode $\rho_{lim} = 0{,}5$ *is used.*

If the aforementioned condition is not satisfied, the influence of local buckling on the stiffness shall be taken into account in global analysis by determining the effective cross-section according to Table 2.10 and clause 2.4.1 using the loads at ultimate limit state. The second moment of area I_{eff} of the effective cross-section can be converted into a constant value I_{fic} for the considered length according to expression (2.13) (the expression shall be applied using load combination at ultimate limit state or at serviceability limit state, whichever is relevant).

Table 2.10 Effective width of internal compression elements (= compressed plane elements restrained on two edges) [13,14]

Stress distribution (compression positive)	Effective width b_{eff}
	$\underline{\psi = 1}$ $b_{eff} = \rho \bar{b}$ $b_{e1} = 0{,}5 b_{eff}$ $b_{e2} = 0{,}5 b_{eff}$
	$\underline{0 \le \psi < 1}$ $b_{eff} = \rho \bar{b}$ $b_{e1} = 2 b_{eff} / (5 - \psi)$ $b_{e2} = b_{eff} - b_{e1}$
	$\underline{\psi < 0}$ $b_{eff} = \rho b_c = \rho \bar{b} / (1 - \psi)$ $b_{e1} = 0{,}4 b_{eff}$ $b_{e2} = 0{,}6 b_{eff}$

$\psi = \sigma_2 / \sigma_1$	$-3 \le \psi < -1$ a)	-1	$-1 < \psi < 0$	0	$0 < \psi < 1$	1
Local buckling factor k_σ	$5{,}98 (1 - \psi)^2$	$23{,}9$	$7{,}81 - 6{,}29\psi + 9{,}78\psi^2$	$7{,}81$	$\dfrac{8,2}{1,05 + \psi}$	$4{,}0$

a) The expression is corrected according to [14].

Dimension $\bar{b}$ is determined according to Table 2.7.

2.4.1 Effective cross-section at ultimate limit state

In the effective width method the reduction factor ρ presented in Table 2.10 is calculated for the compressed plane element of the square or rectangular hollow section as follows [13,14]:

$$\rho = 1,0 \qquad \text{for } \bar{\lambda}_p \le 0,5 + \sqrt{0,085 - 0,055\psi} \tag{2.6a}$$

$$\rho = \frac{\bar{\lambda}_p - 0,055(3 + \psi)}{\bar{\lambda}_p^2} \le 1,0 \qquad \text{for } \bar{\lambda}_p > 0,5 + \sqrt{0,085 - 0,055\psi} \tag{2.6b}$$

The non-dimensional slenderness of a plane element is calculated as follows [13,14]:

$$\bar{\lambda}_p = \sqrt{\frac{f_y}{\sigma_{cr}}} \tag{2.7}$$

The elastic critical local buckling stress σ_{cr} of a plane element is calculated as follows:

$$\sigma_{cr} = k_\sigma \cdot \sigma_E \tag{2.8}$$

where σ_E is Euler stress that is calculated as follows:

$$\sigma_E = \frac{\pi^2 E}{12(1-\nu^2)}(t/\bar{b})^2 = 190000 \cdot (t/\bar{b})^2 \tag{2.9}$$

When the Young's modulus of elasticity for steel is E = 2,1 · 10^5 N/mm^2 and Poisson's ratio ν = 0,3 , the expression presented in EN 1993-1-5 for the non-dimensional slenderness of a plane element can be derived from expressions (2.7) - (2.9) as follows:

$$\bar{\lambda}_p = \frac{\bar{b}/t}{28,4\varepsilon\sqrt{k_\sigma}} \tag{2.10}$$

where

$\bar{b}$	is the calculatory width of the hollow section (= b - $3t$) or the calculatory depth of the web (= h - $3t$)
t	is the wall thickness
ε	$= \sqrt{235/f_y} \quad [f_y] = N/mm^2$
f_y	is the nominal yield strength of the material
k_σ	is the buckling factor corresponding to stress ratio ψ and boundary conditions on the edges of the plane element (Table 2.10)

When the maximum compressive stress of the plane element to be considered falls under the yield level, the non-dimensional slenderness $\bar{\lambda}_p$ can be replaced by a reduced value of the non-dimensional slenderness as follows [13,14]:

$$\bar{\lambda}_{p.red} = \bar{\lambda}_p \sqrt{\frac{\sigma_{com.Ed}}{f_y/\gamma_{M0}}} \tag{2.11}$$

where

$\bar{\lambda}_p$	is the non-dimensional slenderness of the plane element according to expression (2.10)
f_y	the nominal yield strength of the material
γ_{M0}	is the partial safety factor for resistance (Table 2.5)
$\sigma_{com.Ed}$	is the design value of the maximum compressive stress of the plane element, determined on the basis of effective cross-section when all the simultaneously acting loads are taken into account

The aforementioned method is conservative, but it requires iterative calculations where the stress ratio ψ is determined on each iteration cycle from stresses that are calculated on the basis of the effective cross-section obtained from the previous iteration cycle.

When calculating in Class 4 the buckling resistance or lateral-torsional buckling resistance the non-dimensional slenderness of plane elements may be calculated as a reduced value $\bar{\lambda}_{p.red}$ according to expression (2.11) only if $\sigma_{com.Ed}$ is determined according to second order analysis taking into account the global imperfections. Otherwise the normal value of the non-dimensional slenderness $\bar{\lambda}_p$ according to expression (2.10) shall be used [13,14].

If the maximum compressive stress reaches the yield level ($\sigma_{com.Ed} = f_y / \gamma_{M0}$), the value of $\bar{\lambda}_{p.red}$ according to expression (2.11) returns back to non-dimensional slenderness $\bar{\lambda}_p$.

In Annex E of EN 1993-1-5 also an alternative method to calculate the non-dimensional slenderness of a plane element is given. It will not be presented in this handbook, because it is anyways not allowed for determining the effective cross-section when calculating the buckling resistance or lateral-torsional buckling resistance.

2.4.2 Effective cross-section at serviceability limit state

To calculate the second moment of area needed at serviceability limit state, the effective cross-section is determined as at ultimate limit state but using the non-dimensional slenderness $\bar{\lambda}_{p.ser}$, which is calculated as follows [13,14]:

$$\bar{\lambda}_{p.ser} = \bar{\lambda}_p \sqrt{\frac{\sigma_{com.Ed.ser}}{f_y}} \qquad (2.12)$$

where $\bar{\lambda}_p$ is the non-dimensional slenderness of the plane element according to expression (2.10)

f_y is the nominal yield strength of the material

$\sigma_{com.Ed.ser}$ is the maximum compressive stress in the considered plane element, based on the effective cross-section applying the loads at the serviceability limit state when all the simultaneously acting loads are taken into account

The effective second moment of area I_{eff} calculated on the basis of the effective cross-section varies as a function of span. Instead of a variable value it is possible to use one fictive constant value I_{fic} for the whole span (or for the considered length) that is calculated using the loads at serviceability limit state to determine the maximum absolute value of the span moment, and using the following interpolation [11,13,14]:

$$I_{fic} = I_{gr} - \frac{\sigma_{com.gr}}{\sigma_{com.Ed.ser}} [I_{gr} - I_{eff}(\sigma_{com.Ed.ser})] \qquad (2.13)$$

where

I_{gr} is the second moment of area of the gross cross-section (Annex 11.1)

$\sigma_{com.gr}$ is the maximum compressive bending stress in the considered span (or in the considered length) using the loads at serviceability limit state

$I_{eff}(\sigma_{com.Ed.ser})$ is the second moment of area of the effective cross-section that is calculated on the basis of the maximum compressive stress $\sigma_{com.Ed.ser} \geq \sigma_{com.gr}$ acting in the considered span

Serviceability limit state will be discussed further in Chapter 7.

2.5 Resistance of a structural hollow section subject to normal force

2.5.1 Tension resistance

Any cross-section subject to tension is fully effective no matter what the cross-section Class is, hence the slenderness of the cross-section has therein no importance. The design condition for a member subject to tension is [3,4,5]:

$$N_{Ed} \leq N_{t.Rd} \tag{2.14}$$

where N_{Ed} is the design value of the tension force applied to an axially loaded member at ultimate limit state

$N_{t.Rd}$ is the design tension resistance of the cross-section

The tension resistance of the cross-section $N_{t.Rd}$ is the smaller of the following values [3,4,5]:

$$N_{t.Rd} = N_{pl.Rd} = \frac{Af_y}{\gamma_{M0}} \tag{2.15}$$

$$N_{t.Rd} = N_{u.Rd} = \frac{0,9 A_{net} f_u}{\gamma_{M2}} \tag{2.16}$$

where $N_{pl.Rd}$ is the design plastic resistance of the gross cross-section according to plastic theory

$N_{u.Rd}$ is the design ultimate resistance of the net cross-section at fastener holes

A is the gross cross-section area (Annex 11.1)

A_{net} is the net cross-section area at fastener holes

f_y is the nominal yield strength of the material

f_u is the nominal ultimate tensile strength of the material

γ_{M0} is the partial safety factor for resistance to yielding (Table 2.5)

γ_{M2} is the partial safety factor for resistance to fracture (Table 2.5)

Where the so-called capacity design is requested (e.g. in seismic design, EN 1998), the design plastic resistance of the gross cross-section $N_{pl.Rd}$ is not allowed to be greater than the design ultimate resistance of the net section to fracture at fastener holes $N_{u.Rd}$. In other words, the net cross-section is not allowed to experience fracture before the gross cross-section yields [3,4,5]. This requirement can be presented using expressions (2.15) and (2.16) as follows:

$$\frac{A_{net}}{A} \geq \frac{1}{0,9} \cdot \frac{\gamma_{M2}}{\gamma_{M0}} \cdot \frac{f_y}{f_u} \tag{2.17}$$

With bolted connections in Category C (see EN 1993-1-8), the design tension resistance of the net cross-section at fastener holes $N_{net.Rd}$ is calculated as follows [3,4,5]:

$$N_{net.Rd} = \frac{A_{net} f_y}{\gamma_{M0}} \tag{2.18}$$

Net area of cross-section:

The net area of a cross-section should be taken as its gross area less appropriate deductions for all holes and other openings. The reduction for a single fastener hole should be the gross cross-section area of the hole in the plane of its axis. For countersunk holes, the countersunk portion shall be taken into account [3,4,5].

If the fastener holes are not staggered, the total area to be deduced for fastener holes should be the maximum sum of the sectional areas of the holes in any cross-section perpendicular to the member axis (Figure 2.4, fracture line 2).

In case of staggered holes, the net area of the cross-section is the smaller of the following values [3,4,5]:

- **direct fracture line (Figure 2.4, fracture line 2):**

$$A_{net} = A - \sum t d_0 \tag{2.19}$$

- **staggered fracture line (Figure 2.4, fracture line 1):**

$$A_{net} = A - \sum t d_0 + \sum \frac{ts^2}{4p} \tag{2.20}$$

where A_{net} is the net cross-section area at the fastener holes
A is the gross cross-section area (Annex 11.1)
t is the wall thickness
d_0 is the diameter of the hole
s is the distance of the centres of two adjacent staggered holes in the chain measured parallel to the member axis
p is the distance of the centres of two adjacent staggered holes measured perpendicular to the member axis

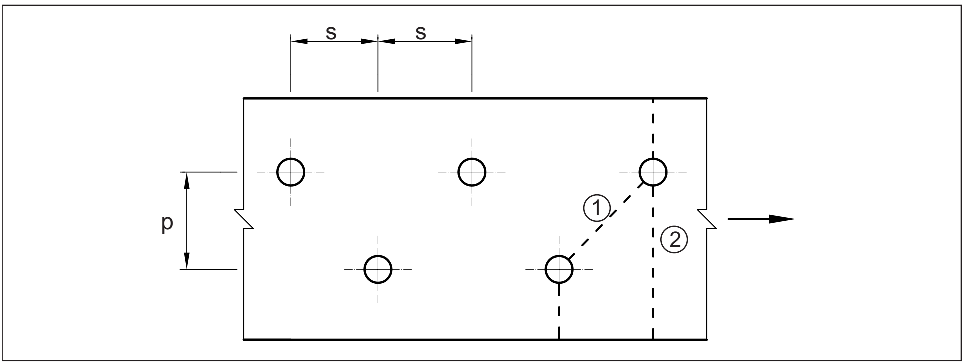


Figure 2.4 Determination of the net cross-section. Fracture lines 1 and 2 [3,4,5]

2.5.2 Compression resistance (buckling excluded)

2.5.2.1 Compression resistance of square and rectangular hollow sections and Class 1, 2 and 3 circular hollow sections (buckling excluded)

When determining the resistance of the cross-section of a compressed member, only the local stability phenomenon are taken into account, i.e. the effect of possible local buckling of the plane elements to the resistance. When determining the compression resistance of the entire member, also the global loss of stability is taken into account, i.e. possible global buckling in its different forms (clause 2.5.3).

The design condition for the cross-section of compressed members is [3,4,5]:

$$N_{Ed} \leq N_{c.Rd} \tag{2.21}$$

where N_{Ed} is the design value of the compression force at ultimate limit state
$N_{c.Rd}$ is the design compression resistance of the cross-section

The design compression resistance of the cross-section $N_{c.Rd}$ to concentric uniform compression is calculated as follows [3,4,5]:

$$N_{c.Rd} = N_{pl.Rd} = \frac{A f_y}{\gamma_{M0}} \quad \textit{for Class 1, 2 and 3 (all hollow section forms)} \tag{2.22}$$

$$N_{c.Rd} = \frac{A_{eff} f_y}{\gamma_{M0}} \quad \textit{for Class 4 (square and rectangular)} \tag{2.23}$$

where $N_{pl.Rd}$ is the design plastic resistance of the gross cross-section according to plastic theory
A is the gross cross-section area (Annex 11.1)
A_{eff} is the effective cross-section area in concentric compression
f_y is the nominal yield strength of the material
γ_{M0} is the partial safety factor for resistance (Table 2.5)

Provided the fastener is placed in the hole, the fastener holes need not be taken into account when calculating the resistance of a member subject to compression and bending, except for oversized or slotted holes according to EN 1090 [3,4,5,29].

The resistance of the cross-section for members subject to eccentric compression is determined using interaction formulae for combined effect of compression and bending according to clause 2.9.1.5.

2.5.2.2 Compression resistance of Class 4 circular hollow section (buckling excluded)

The design condition for the compression resistance of Class 4 circular hollow sections is:

$$N_{Ed} \le N_{c.Rd} \tag{2.24}$$

where N_{Ed} is the design value of the compression force at ultimate limit state
$N_{c.Rd}$ is the design compression resistance of the cross-section

The design compression resistance of the cross-section $N_{c.Rd}$ for concentric uniform compression is determined on the basis of local buckling strength $\chi_x f_y$ as follows, when there is an end-plate or a rigid ring welded at both ends of the hollow section that prevent the deformation of the cross-section (or the hollow section is welded at its ends to the surrounding structure in the corresponding way) [16]:

$$N_{c.Rd} = \frac{\chi_x A f_y}{\gamma_{M1}} \tag{2.25}$$

where χ_x is the reduction factor for elastic-plastic local buckling of a shell
A is the gross cross-section area (Annex 11.1)
f_y the nominal yield strength of the material
γ_{M1} is the partial safety factor for resistance **according to EN 1993-1-6** (Table 2.5)

The reduction factor χ_x for elastic-plastic local buckling is calculated as follows [16]:

$$\chi_x = 1,0 \qquad for\ \bar{\lambda}_x \le 0,2 \tag{2.26a}$$

$$\chi_x = 1 - 0,6\frac{\bar{\lambda}_x - 0,2}{\bar{\lambda}_{pl.x} - 0,2} \qquad for\ 0,2 < \bar{\lambda}_x < \bar{\lambda}_{pl.x} \tag{2.26b}$$

$$\chi_x = \frac{\alpha_x}{\bar{\lambda}_x^2} \qquad for\ \bar{\lambda}_x \ge \bar{\lambda}_{pl.x} \tag{2.26c}$$

where α_x is the elastic imperfection reduction factor for local buckling
$\bar{\lambda}_x$ is the non-dimensional slenderness
$\bar{\lambda}_{pl.x}$ is the value of plastic limit non-dimensional slenderness

The elastic imperfection reduction factor α_x is obtained from the formula [16]:

$$\alpha_x = \frac{0,62}{1 + 1,91(\Delta w_k / t)^{1,44}} \tag{2.27}$$

where Δw_k is the characteristic value of the imperfection amplitude
t is the wall thickness

The characteristic value of the imperfection amplitude Δw_k needed in the expression (2.27) is calculated as follows [16]:

$$\Delta w_k = \frac{\sqrt{r_m t}}{Q} \tag{2.28}$$

$$r_m = \frac{d - t}{2} \tag{2.29}$$

where r_m is the radius of the centreline of the hollow section wall thickness
t is the wall thickness
d is the external diameter of the hollow section
Q is the fabrication quality parameter according to the tolerances of the hollow section, that is obtained from Tables 2.11 and 2.12.

The fabrication quality parameter Q is determined according to fabrication tolerance quality classes A...C (A is the best) given in EN 1090-2 for circular cylinders [29]. On circular hollow sections the most essential geometrical tolerance is usually the tolerance for out-of-roundness. When the external diameter is $d \leq$ 400 mm and $d/t \leq$ 100, the SSAB hollow sections meet the out-of-roundness quality class B (Tables 1.2 and 2.12). Thereby parameter Q obtains the value Q = 25 (Table 2.11). With other dimensions, the applicable fabrication tolerance quality class shall be checked case by case.

The plastic limit non-dimensional slenderness $\bar{\lambda}_{pl.x}$ is [16]:

$$\bar{\lambda}_{pl.x} = \sqrt{\frac{\alpha_x}{0,4}} \tag{2.30}$$

The non-dimensional slenderness $\bar{\lambda}_x$ is [16]:

$$\bar{\lambda}_x = \sqrt{\frac{f_y}{\sigma_{cr}}} \tag{2.31}$$

where f_y is the nominal yield strength of the material
σ_{cr} is the elastic critical local buckling stress

The elastic critical local buckling stress of a circular hollow section σ_{cr} is calculated as follows [16]:

$$\sigma_{cr} = \frac{E}{\sqrt{3(1-\nu^2)}} \cdot C_x \frac{t}{r_m} = 0,605 E C_x \frac{t}{r_m} \tag{2.32}$$

where E is the Young's modulus of elasticity
t is the wall thickness
r_m is the radius of the centreline of the hollow section wall thickness

EN 1993-1-6 defines the factor C_x as a function of the length of the cylinder-like structure considered. In practice, in hollow section structures the length of a circular hollow section is usually more than $(1...2)\times d$. In such case the value obtained from Eurocode is C_x = 0,6. This value may be used also for shorter lengths as a conservative simplification.

Table 2.11 Fabrication quality parameter Q for different fabrication tolerance quality classes of circular hollow sections [16]

Fabrication tolerance quality class	Description	Q
Class A	Excellent	40
Class B	High	25
Class C	Normal	16

Table 2.12 Permitted out-of-roundness in different fabrication tolerance quality classes [29]

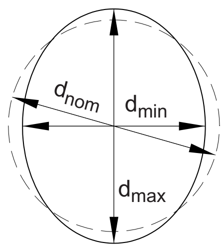
Fabrication tolerance quality class	Out-of-roundness: permitted deviation Δ a)		
	Internal diameter		
	$d_i \le 0{,}50$ m b)	$0{,}5 \text{ m} < d_i < 1{,}25$ m b)	$d_i \ge 1{,}25$ m b)
Class A	$\Delta = \pm\, 0{,}014$	$\Delta = \pm\,[0{,}007 + 0{,}0093\,(1{,}25 - d_i)]$	$\Delta = \pm\, 0{,}007$
Class B	$\Delta = \pm\, 0{,}020$	$\Delta = \pm\,[0{,}010 + 0{,}0133\,(1{,}25 - d_i)]$	$\Delta = \pm\, 0{,}010$
Class C	$\Delta = \pm\, 0{,}030$	$\Delta = \pm\,[0{,}015 + 0{,}0200\,(1{,}25 - d_i)]$	$\Delta = \pm\, 0{,}015$

a) Out-of-roundness:
Difference between the maximum and minimum values of the measured internal diameter, relative to the nominal internal diameter:

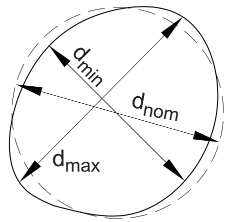
$$\Delta = \frac{d_{max} - d_{min}}{d_{nom}}$$

b) d_i is the nominal internal diameter in metres

flattening:



unsymmetrical:



Example 2.1

Calculate the cross-section compression resistance of a hollow section 200×100×5. The cross-section is subjected to uniform compression only.

The steel grade is SSAB Domex Tube Double Grade, which fulfills the EN 10219 requirements for both steel grades S420MH and S355J2H. Thereby the design calculations may be performed at designer's own choice either according to grade S420 or grade S355. Grade S420 is chosen in this Example as design basis.

$A = 2836\ mm^2$ *(Annex 11.1)*
$f_y = 420\ N/mm^2$
$\gamma_{M0} = 1{,}0$

Check the cross-section classification (Table 2.9):

Flange:

$b/t = 100/5 = 20,0 \leq 27,7 \qquad \Rightarrow Class\ 1$

Web:

$h/t = 200/5 = 40,0 > 34,4 \qquad \Rightarrow Class\ 4$

The whole cross-section shall be classified into Class 4 according to its most critical plane element.

Calculate the effective cross-section area:
Flanges are fully effective.

Webs:

$$\bar{b} = h - 3t = 200 - 3 \cdot 5 = 185\ mm$$

$$\psi = 1\ ,\ \ k_\sigma = 4{,}0$$

$$\bar{\lambda}_p = \frac{\bar{b}/t}{28,4\varepsilon\sqrt{k_\sigma}} = \frac{(200 - 3 \cdot 5)/5}{28,4 \cdot \sqrt{235/420} \cdot \sqrt{4,0}} = 0,8709$$

$$\bar{\lambda}_p = 0,8709 > 0,5 + \sqrt{0,085 - 0,055\psi} = 0,5 + \sqrt{0,085 - 0,055 \cdot 1} = 0,6732$$

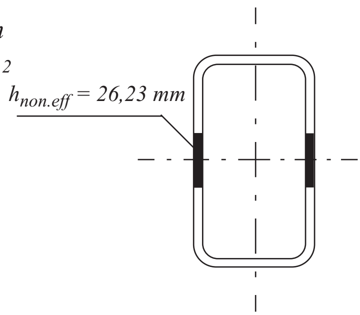
$$\rho = \frac{\bar{\lambda}_p - 0,055(3 + \psi)}{\bar{\lambda}_p^2} = \frac{0,8709 - 0,055 \cdot 4}{0,8709^2} = 0,8582 \leq 1,0$$

$$h_{eff} = \rho\bar{b} = 0,8582 \cdot 185 = 158,8\ mm$$

The non-effective part and non-effective area of the web is hence:

$$h_{non.eff} = (1 - \rho)\bar{b} = (1 - 0,8582) \cdot 185 = 26,23\ mm$$

$$A_{non.eff} = 2 \cdot h_{non.eff} \cdot t = 2 \cdot 26,23 \cdot 5 = 262,3\ mm^2$$



The effective cross-section area:

$$A_{eff} = A - A_{non.eff} = 2836 - 262,3 = 2574\ mm^2$$

The compression resistance of the cross- section:

$$N_{c.Rd} = \frac{A_{eff} f_y}{\gamma_{M0}} = \frac{2574 \cdot 420}{1,0} = 1081\ kN$$

Comparison S420 vs S355:
In case the design calculations would be performed according to grade S355, the compression resistance of the cross-section would be 945,0 kN (Class 4, A_{eff} = 2662 mm^2). In both cases the cross-section belongs to Class 4, telling the cross-section is not fully effective. Increase of the material strength S355 → S420 further decreases the cross-section's effectiveness. Nevertheless, increase of the yield strength by 18 % improves the compression resistance in this Example by 14 %.

Example 2.2

Calculate the cross-section compression resistance of a circular hollow section 323,9×5. The cross-section is subjected to uniform compression only.

The steel grade is SSAB Domex Tube Double Grade, which fulfills the EN 10219 requirements for both steel grades S420MH and S355J2H. Thereby the design calculations may be performed at designer's own choice either according to grade S420 or grade S355. Grade S420 is chosen in this Example as design basis.

$A = 5009\ mm^2$ *(Annex 11.1)*
$f_y = 420\ N/mm^2$
$\gamma_{M1} = 1,1$ *(Class 4 circular hollow section, EN 1993-1-6)*

Check the cross-section classification (Table 2.9):

$$d/t = 323,9/5 = 64,8 > 50,4 \qquad \Rightarrow Class\ 4$$

The elastic critical local buckling stress and non-dimensional slenderness of the hollow section are:

$$\sigma_{cr} = 0,605EC_x\frac{t}{r_m} = 0,605 \cdot 2,1 \cdot 10^5 \cdot 0,6 \cdot \frac{5}{159,5} = 2390\ N/mm^2$$

$$\bar{\lambda}_x = \sqrt{f_y/\sigma_{cr}} = \sqrt{420/2390} = 0,4192$$

The hollow section has $d \leq 400$ mm and $d/t \leq 100$ whereby it satisfies the requirements specified for the fabrication tolerance quality class B (Tables 1.2 and 2.11)

$$\Rightarrow Q = 25 \qquad (Table\ 2.11)$$

$$\Delta w_k = \frac{\sqrt{r_m t}}{Q} = \frac{\sqrt{159,5 \cdot 5}}{25} = 1,130$$

$$\alpha_x = \frac{0,62}{1 + 1,91(\Delta w_k/t)^{1,44}} = \frac{0,62}{1 + 1,91 \cdot (1,130/5)^{1,44}} = 0,5064$$

$$\bar{\lambda}_{pl.x} = \sqrt{\frac{\alpha_x}{0,4}} = \sqrt{\frac{0,5064}{0,4}} = 1,125$$

$$0,2 < \bar{\lambda}_x < \bar{\lambda}_{pl.x}$$

$\Rightarrow$

$$\chi_x = 1 - 0,6\frac{\bar{\lambda}_x - 0,2}{\bar{\lambda}_{pl.x} - 0,2} = 1 - 0,6 \cdot \frac{0,4192 - 0,2}{1,125 - 0,2} = 0,8578$$

The compression resistance of the cross-section is finally:

$$N_{c.Rd} = \frac{\chi_x A f_y}{\gamma_{M1}} = \frac{0,8578 \cdot 5009 \cdot 420}{1,1} = 1641\ kN$$

Comparison S420 vs S355:
In case the design calculations would be performed according to grade S355, the compression resistance of the cross-section would be 1422 kN. In both cases we have Class 4 circular cross-section, and increase of the material strength S355 → S420 further increases the cross-section's non-dimensional slenderness. Nevertheless, increase of the yield strength by 18 % improves the compression resistance in this Example by 15 %.

2.5.3 Buckling resistance

2.5.3.1 Buckling resistance of square and rectangular hollow sections and Class 1, 2 and 3 circular hollow sections

A structural hollow section is superior as a compressed member, because its material is efficiently located far from the centre point of the cross-section. Due to high torsional stiffness, the torsional buckling or torsional-flexural buckling need not be taken into account. With structural hollow sections, only flexural buckling needs to be checked with respect to both principal axes.

The design condition for a compressed member against buckling is [3,4,5]:

$$N_{Ed} \le N_{b.Rd} \qquad (2.33)$$

where N_{Ed} is the design value of the compression force in an axially compressed member at ultimate limit state
$N_{b.Rd}$ is the design buckling resistance of an axially compressed member

The buckling resistance need not to be determined if the non-dimensional slenderness of the member is $\bar{\lambda} \le 0{,}2$ or if $N_{Ed}/N_{cr} \le 0{,}04$ [3,4,5]. The determination of the non-dimensional slenderness $\bar{\lambda}$ and the elastic critical buckling force N_{cr} is presented later on.

The resistance of eccentrically compressed members shall be determined using the interaction formulae for combined compression and bending as presented in clause 2.10.1.

The design buckling resistance $N_{b.Rd}$ for each related buckling mode and direction, is calculated as follows [3,4,5]:

$$N_{b.Rd} = \frac{\chi A f_y}{\gamma_{M1}} \qquad \textit{for Class 1, 2 and 3 (all hollow section forms)} \qquad (2.34)$$

$$N_{b.Rd} = \frac{\chi A_{eff} f_y}{\gamma_{M1}} \qquad \textit{for Class 4 (square and rectangular)} \qquad (2.35)$$

where χ is the reduction factor for the related buckling resistance
A is the gross cross-section area (Annex 11.1)
A_{eff} is the effective cross-section area in concentric compression
f_y is the nominal yield strength of the material
γ_{M1} is the partial safety factor for resistance (Table 2.5)

When calculating the areas A and A_{eff} the fastener holes at the ends of the column need not be taken into account [3,4,5].

The reduction factor χ in the expressions (2.34) and (2.35) is calculated as follows:

$$\chi = 1,0 \qquad \text{for} \quad \bar{\lambda} \leq 0,2 \tag{2.36a}$$

$$\chi = \frac{1}{\Phi + \sqrt{\Phi^2 - \bar{\lambda}^2}} \leq 1,0 \qquad \text{for} \quad \bar{\lambda} > 0,2 \tag{2.36b}$$

$$\Phi = 0,5 \cdot [1 + \alpha(\bar{\lambda} - 0,2) + \bar{\lambda}^2] \tag{2.37}$$

where α is the imperfection factor of the related buckling curve (cold-formed structural hollow sections: α = 0,49)
$\bar{\lambda}$ is the non-dimensional slenderness of the member

The imperfection factor α is determined on the basis of the buckling curve (i.e. buckling class) to be applied. According to Eurocode 3, for cold-formed structural hollow sections the buckling curve c (whereby α = 0,49) shall always be applied for all directions and for all grades S235-S460 [3,4,5]. The initial geometrical curvature of the compressed member needs then meet the tolerances for execution as specified in EN 1090-2 (Chapter 8).

The non-dimensional slenderness $\bar{\lambda}$ needed in expressions (2.36) - (2.37) is calculated as follows [3,4,5]:

$$\bar{\lambda} = \sqrt{\frac{A f_y}{N_{cr}}} \qquad \text{for Class 1, 2 and 3 (all hollow section forms)} \tag{2.38}$$

$$\bar{\lambda} = \sqrt{\frac{A_{eff} f_y}{N_{cr}}} \qquad \text{for Class 4 (square and rectangular)} \tag{2.39}$$

where A is the gross cross-section area (Annex 11.1)
A_{eff} is the effective cross-section area in concentric compression
f_y is the nominal yield strength of the material
N_{cr} is the elastic critical buckling force for the related buckling mode and direction

The elastic critical buckling force N_{cr} is calculated for the flexural buckling as follows, wherein <u>the properties of the cross-section are determined for all cross-section classes according to gross cross-section</u> [3,4,5]:

$$N_{cr} = \frac{\pi^2 EI}{L_{cr}^2} = \frac{\pi^2 EA}{(L_{cr}/i)^2} \tag{2.40}$$

where E is the Young's modulus of elasticity

I is the second moment of area of the gross cross-section with regard to the relevant axis (Annex 11.1)

L_{cr} is the buckling length of the member in flexural buckling to the relevant direction

A is the gross cross-section area (Annex 11.1)

i is the radius of gyration of the gross cross-section with regard to the relevant axis (Annex 11.1)

The buckling length of the member L_{cr} for different buckling modes and directions is determined by taking into account the degree of restraint at the ends of the member with regard to the related buckling mode and direction. However, in lattice structures it is usually conservative to use as buckling length the real length of the member, and in frame structures the theoretical buckling length, ignoring the stiffness of the joints in both aforementioned cases. The determination of the buckling length is presented in more details in Chapter 7.

2.5.3.2 Buckling resistance of Class 4 circular hollow section

The design condition for the buckling resistance of Class 4 circular hollow sections is:

$$N_{Ed} \leq N_{b.Rd} \tag{2.41}$$

where N_{Ed} is the design value of the compression force in an axially compressed member at ultimate limit state

$N_{b.Rd}$ is the design buckling resistance of an axially compressed member

The design buckling resistance $N_{b.Rd}$ is calculated for Class 4 circular hollow sections as follows:

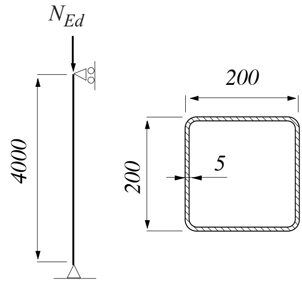
$$N_{b.Rd} = \chi N_{c.Rd} \tag{2.42}$$

where $N_{c.Rd}$ is the compression resistance of the cross-section according to clause 2.5.2.2

χ is the reduction factor for flexural buckling

The reduction factor χ for flexural buckling is calculated in the same way as for Classes 1...3 in clause 2.5.3.1 (the cross-section of Class 4 circular hollow section is always fully effective), but now in expression (2.38) the yield strength f_y is replaced by local buckling strength $\chi_x f_y$, where the reduction factor χ_x for elastic-plastic local buckling is calculated according to clause 2.5.2.2.

Example 2.3



Calculate the compression resistance of a hollow section 200×200×5. The member is simply supported at both ends and the buckling length for flexural buckling is 4 m about both axes.

The steel grade is SSAB Domex Tube Double Grade, which fulfills the EN 10219 requirements for both steel grades S420MH and S355J2H. Thereby the design calculations may be performed at designer's own choice either according to grade S420 or grade S355. Grade S420 is chosen in this Example as design basis.

$A = 3836\ mm^2$
$I = 2410 \cdot 10^4\ mm^4$ *(Annex 11.1)*
$f_y = 420\ N/mm^2$
$\gamma_{M0} = 1{,}0$
$\gamma_{M1} = 1{,}0$

Check the cross-section classification (Table 2.9):

Web and flange:
$h/t = b/t = 200/5 = 40{,}0 > 34{,}4 \qquad \Rightarrow$ *Class 4*

Since the cross-section is classified into Class 4, calculate the effective cross-section area:

$$\psi = 1\ ,\ k_\sigma = 4,0$$

$$\bar{\lambda}_p = \frac{\bar{b}/t}{28,4\varepsilon\sqrt{k_\sigma}} = \frac{(b-3t)/t}{28,4\varepsilon\sqrt{k_\sigma}} = \frac{(200-3\cdot 5)/5}{28,4\cdot\sqrt{235/420}\cdot\sqrt{4,0}} = 0,8709$$

$$\bar{\lambda}_p = 0,8709 > 0,5+\sqrt{0,085-0,055\psi} = 0,5+\sqrt{0,085-0,055\cdot 1} = 0,6732$$

$$\rho = \frac{\bar{\lambda}_p - 0,055(3+\psi)}{\bar{\lambda}_p^2} = \frac{0,8709-0,055\cdot 4}{0,8709^2} = 0,8582 \le 1,0$$

The effective part and non-effective part of the cross-section are hence:

$$b_{eff} = \rho\bar{b} = 0,8582\cdot(200-3\cdot 5) = 158,8\ mm$$

$$b_{non.eff} = (b-3t)-b_{eff} = (200-3\cdot 5)-158,8 = 26,2\ mm$$

Calculate effective area based on above obtained effective cross-section:

$$A_{eff} = A-(4\cdot b_{non.eff}\cdot t) = 3836-(4\cdot 26,2\cdot 5) = 3312\ mm^2$$

The compression resistance of the cross-section:

$$N_{c.Rd} = \frac{A_{eff}f_y}{\gamma_{M0}} = \frac{3312\cdot 420}{1,0} = 1391\ kN$$

Since local buckling of the cross-section has now been considered, flexural buckling of the member can be checked next.

Although the cross-section belongs to Class 4, the elastic critical force N_{cr} for flexural buckling shall be determined by using gross cross-section properties:

$$N_{cr} = \frac{\pi^2 EI}{L_{cr}^2} = \frac{\pi^2 \cdot 210000 \cdot 2410 \cdot 10^4}{4000^2} = 3122\ kN$$

Non-dimensional slenderness of the member:

$$\bar{\lambda} = \sqrt{\frac{A_{eff} f_y}{N_{cr}}} = \sqrt{\frac{3312 \cdot 420}{3122 \cdot 10^3}} = 0,6675 > 0,2$$

For flexural buckling, buckling curve c (α = 0,49) shall be used for cold-formed structural hollow sections. Hence:

$$\Phi = 0,5 \cdot [1 + \alpha(\bar{\lambda} - 0,2) + \bar{\lambda}^2] = 0,5 \cdot [1 + 0,49 \cdot (0,6675 - 0,2) + 0,6675^2] = 0,8373$$

$$\chi = \frac{1}{\Phi + \sqrt{\Phi^2 - \bar{\lambda}^2}} = \frac{1}{0,8373 + \sqrt{0,8373^2 - 0,6675^2}} = 0,7447 \le 1,0$$

The buckling resistance of the member is finally:

$$N_{b.Rd} = \frac{\chi A_{eff} f_y}{\gamma_{M1}} = \frac{0,7447 \cdot 3312 \cdot 420}{1,0} = 1036\ kN$$

Comparison S420 vs S355:
In case the design calculations would be performed according to grade S355, the buckling resistance of the member would be 950,5 kN. In both cases the cross-section belongs to Class 4, telling the cross-section is not fully effective. Increase of the material strength S355 → S420 further decreases the cross-section's effectiveness and increases the member's non-dimensional slenderness for flexural buckling. Nevertheless, increase of the yield strength by 18 % improves the buckling resistance in this Example by 9 %.

Example 2.4

Calculate the compression resistance of a hollow section 200×200×8. The member is simply supported at both ends and the buckling length for flexural buckling is 4 m about both axes.

The steel grade is SSAB Domex Tube Double Grade, which fulfills the EN 10219 requirements for both steel grades S420MH and S355J2H. Thereby the design calculations may be performed at designer's own choice either according to grade S420 or grade S355. Grade S420 is chosen in this Example as design basis.

$A = 5924\ mm^2$ *(Annex 11.1)*
$I = 3566 \cdot 10^4\ mm^4$ *(Annex 11.1)*
$f_y = 420\ N/mm^2$
$\gamma_{M0} = 1,0$
$\gamma_{M1} = 1,0$

Check the cross-section classification (Table 2.9):

Web and flange:
$h / t = b / t = 200 / 8 = 25{,}0 \leq 27{,}7 \qquad \Rightarrow$ *Class 1*

Calculations for effective cross-section are not needed in cross-section Classes 1, 2 and 3, since therein the cross-section is fully effective.

The compression resistance of the cross-section:

$$N_{c.Rd} = \frac{A f_y}{\gamma_{M0}} = \frac{5924 \cdot 420}{1,0} = 2488\ kN$$

Next, check flexural buckling of the member.

The elastic critical force for flexural buckling:

$$N_{cr} = \frac{\pi^2 EI}{L_{cr}^2} = \frac{\pi^2 \cdot 210000 \cdot 3566 \cdot 10^4}{4000^2} = 4619\ kN$$

Non-dimensional slenderness of the member:

$$\bar{\lambda} = \sqrt{\frac{A f_y}{N_{cr}}} = \sqrt{\frac{5924 \cdot 420}{4619 \cdot 10^3}} = 0,7339 > 0,2$$

The reduction factor for flexural buckling (buckling curve c: $\alpha = 0{,}49$):

$$\Phi = 0,5 \cdot [1 + \alpha(\bar{\lambda} - 0,2) + \bar{\lambda}^2] = 0,5 \cdot [1 + 0,49 \cdot (0,7339 - 0,2) + 0,7339^2] = 0,9001$$

$$\chi = \frac{1}{\Phi + \sqrt{\Phi^2 - \bar{\lambda}^2}} = \frac{1}{0,9001 + \sqrt{0,9001^2 - 0,7339^2}} = 0,7036 \leq 1,0$$

The buckling resistance of the member is finally:

$$N_{b.Rd} = \frac{\chi A f_y}{\gamma_{M1}} = \frac{0,7036 \cdot 5924 \cdot 420}{1,0} = 1751\ kN$$

Comparison S420 vs S355:
In case the design calculations would be performed according to grade S355, the buckling resistance of the member would be 1557 kN. In both cases the cross-section belongs to Class 1, telling the cross-section is fully effective. However, increase of the material strength S355 → S420 increases the member's non-dimensional slenderness for flexural buckling. Nevertheless, increase of the yield strength by 18 % improves the buckling resistance in this Example by 12 %.

Example 2.5

Calculate the compression resistance of the circular hollow section 323,9×5 from Example 2.2. The cross-section is classified into Class 4. The member is simply supported at both ends and the buckling length for flexural buckling is 4 m about both axes.

The steel grade is SSAB Domex Tube Double Grade, which fulfills the EN 10219 requirements for both steel grades S420MH and S355J2H. Thereby the design calculations may be performed at designer's own choice either according to grade S420 or grade S355. Grade S420 is chosen in this Example as design basis.

$A = 5009\ mm^2$ (Annex 11.1)
$I = 6369 \cdot 10^4\ mm^4$ (Annex 11.1)
$f_y = 420\ N/mm^2$
$\gamma_{M1} = 1{,}1$ (Class 4 circular hollow section, EN 1993-1-6)

The compression resistance $N_{c.Rd}$ of the cross-section and reduction factor χ_x for elastic-plastic local buckling are the same as calculated in Example 2.2:

$\chi_x = 0{,}8578$
$N_{c.Rd} = 1641\ kN$

Next, check flexural buckling of the member.

The elastic critical force for flexural buckling:

$$N_{cr} = \frac{\pi^2 EI}{L_{cr}^2} = \frac{\pi^2 \cdot 210000 \cdot 6369 \cdot 10^4}{4000^2} = 8250\ kN$$

Non-dimensional slenderness of the member:

$$\bar{\lambda} = \sqrt{\frac{\chi_x A f_y}{N_{cr}}} = \sqrt{\frac{0,8578 \cdot 5009 \cdot 420}{8250 \cdot 10^3}} = 0,4677 > 0,2$$

The reduction factor for flexural buckling (buckling curve c: $\alpha = 0{,}49$):

$$\Phi = 0,5 \cdot [1 + \alpha(\bar{\lambda} - 0,2) + \bar{\lambda}^2] = 0,5 \cdot [1 + 0,49 \cdot (0,4677 - 0,2) + 0,4677^2] = 0,6750$$

$$\chi = \frac{1}{\Phi + \sqrt{\Phi^2 - \bar{\lambda}^2}} = \frac{1}{0,6750 + \sqrt{0,6750^2 - 0,4677^2}} = 0,8608 \le 1,0$$

The buckling resistance of the member is finally:

$$N_{b.Rd} = \chi \cdot N_{c.Rd} = 0,8608 \cdot 1641 = 1413\ kN$$

Comparison S420 vs S355:
In case the design calculations would be performed according to grade S355, the buckling resistance of the member would be 1249 kN. In both cases we have Class 4 circular cross-section, and increase of the material strength S355 → S420 further increases the cross-section's non-dimensional slenderness and increases the member's non-dimensional slenderness for flexural buckling. Nevertheless, increase of the yield strength by 18 % improves the buckling resistance in this Example by 13 %.

2.6 Resistance of a structural hollow section subject to bending moment

2.6.1 Bending resistance of square and rectangular and Class 1, 2 and 3 circular hollow sections

A structural hollow section applies well to structures subject to bending. It is especially advantageous, when subject to multiaxial loading. In bending about the weaker axis, the bending resistance of the structural hollow section is better compared to I-sections. With structural hollow sections, the lateral bracing needed for a structure can be performed by using a longer distance between the bracing.

The design condition for a cross-section subject to bending moment is [3,4,5]:

$$M_{Ed} \leq M_{c.Rd} \tag{2.43}$$

where M_{Ed} is the design value of the bending moment at ultimate limit state, on a circular hollow section subject to $M_{y.Ed}$ and $M_{z.Ed}$ the total moment is $M_{Ed} = \sqrt{M_{y.Ed}^2 + M_{z.Ed}^2}$

$M_{c.Rd}$ is the design bending resistance of the cross-section

The design bending resistance of the cross-section $M_{c.Rd}$ is calculated in different cross-section Classes as follows [3,4,5]:

$$M_{c.Rd} = M_{pl.Rd} = \frac{W_{pl} f_y}{\gamma_{M0}} \quad \textit{Class 1 and 2} \quad \textit{(all hollow section shapes)} \tag{2.44}$$

$$M_{c.Rd} = M_{el.Rd} = \frac{W_{el} f_y}{\gamma_{M0}} \quad \textit{Class 3} \quad \textit{(all hollow section shapes)} \tag{2.45}$$

$$M_{c.Rd} = M_{eff.Rd} = \frac{W_{eff} f_y}{\gamma_{M0}} \quad \textit{Class 4} \quad \textit{(square and rectangular)} \tag{2.46}$$

where W_{pl} is the plastic section modulus of the cross-section (Annex 11.1)
W_{el} is the elastic section modulus of the cross-section (Annex 11.1)
W_{eff} is the section modulus of the effective cross-section
f_y is the nominal yield strength of the material
γ_{M0} is the partial safety factor for resistance (Table 2.5)

The plastic and elastic section moduli for structural hollow sections are obtained from the tables presented in Annex 11.1. The effective section modulus for Class 4 is calculated as follows:

$$W_{eff.c} = \frac{I_{eff}}{e_c} \tag{2.47}$$

$$W_{eff.t} = \frac{I_{eff}}{e_t} \tag{2.48}$$

$$W_{eff} = min[W_{eff.c}, W_{eff.t}] \tag{2.49}$$

where
$W_{eff.c}$ is the section modulus for the compressed side of the effective cross-section
$W_{eff.t}$ is the section modulus for the tension side of the effective cross-section
I_{eff} is the second moment of area of the effective cross-section
e_c is the distance at effective cross-section from the outer surface of the compressed flange to the neutral axis of the effective cross-section
e_t is the distance at effective cross-section from the outer surface of the tension flange to the neutral axis of the effective cross-section

The determination of the effective cross-section is presented in clause 2.4.

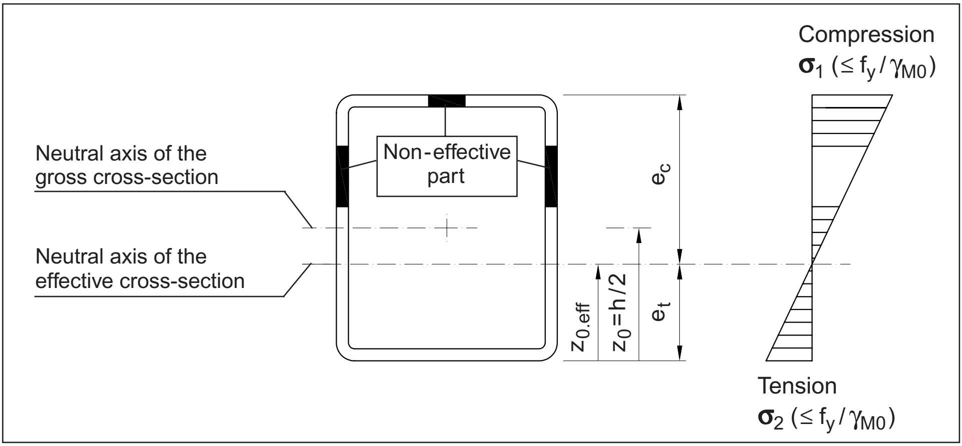


Figure 2.5 Class 4 effective cross-section subject to bending moment

When the cross-section is subject to bending only, the neutral axis of the effective cross-section is located in the centre of gravity of the effective (as if 'perforated') cross-section, Figure 2.5. In Class 4, the bending moment causes a shift of the neutral axis even in case of double symmetrical cross-section. On square and rectangular hollow sections, the location of the neutral axis at effective cross-section can be calculated in the easiest way as follows, when the cross-section is subject to bending only (in other case the location of the neutral axis has to be determined on the basis of force equilibrium condition of the effective cross-section $\Sigma F = N_{Ed}$):

$$z_{0.eff} = \frac{Az_0 - \sum[A_{non.eff.i} \cdot z_{non.eff.i}]}{A - \sum A_{non.eff.i}} \tag{2.50}$$

where A is the gross cross-section area (Annex 11.1)
z_0 is the z-coordinate of the neutral axis at the gross cross-section ($z_0 = h/2$)
$A_{non.eff.i}$ is the area of a considered non-effective part of the cross-section
$z_{non.eff.i}$ is the z-coordinate for centre of gravity of the considered non-effective part of the cross-section

When the place of the effective cross-section is known, the effective second moment of area can be calculated using the following expression that bases on the Steiner rule:

$$I_{eff} = I + A(z_0 - z_{0.eff})^2 - \sum I_{non.eff.i} - \sum[A_{non.eff.i}(z_{non.eff.i} - z_{0.eff})^2] \tag{2.51}$$

where I is the second moment of area of the gross cross-section (Annex 11.1)
A is the gross cross-section area (Annex 11.1)
z_0 is the z-coordinate of the neutral axis at the gross cross-section ($z_0 = h/2$)
$z_{0.eff}$ is the z-coordinate of the neutral axis at the effective cross-section from expression (2.50)
$I_{non.eff.i}$ is the second moment of area of a considered non-effective part about its own centre of gravity (for a rectangular part of cross-section $I = ab^3/12$, where a is the width of the rectangle and b is the depth of the rectangle)
$A_{non.eff.i}$ is the area of a considered non-effective part of the cross-section
$z_{non.eff.i}$ is the z-coordinate for centre of gravity of the considered non-effective part of the cross-section

2.6.2 Effect of holes to the bending resistance

The fastener holes located in the tension flange need not be taken into account, if the design ultimate resistance of the net cross-section of the flange is at least equal to the design plastic resistance of the gross cross-section of the flange [3,4,5]:

$$\frac{0,9A_{f.net}f_u}{\gamma_{M2}} \geq \frac{A_f f_y}{\gamma_{M0}} \tag{2.52}$$

where $A_{f.net}$ is the net cross-section area of the flange in tension
f_u is the nominal ultimate tensile strength of the material

γ_{M2}	is the partial safety factor for resistance to fracture (Table 2.5)
A_f	is the gross cross-section area of the tension flange, on structural hollow section $A_f = 0,5 \cdot [A - 2(h - 3t)t]$
f_y	is the nominal yield strength of the material
γ_{M0}	is the partial safety factor for resistance to yielding (Table 2.5)

Expression (2.52) can also be presented in the following form (cf. expression (2.17)):

$$\frac{A_{f.net}}{A_f} \geq \frac{1}{0,9} \cdot \frac{\gamma_{M2}}{\gamma_{M0}} \cdot \frac{f_y}{f_u} \tag{2.53}$$

This condition ensures that the so-called capacity design principle will be met, cf. clause 2.5.1.

If the aforementioned condition is not met, the cross-section area of the tension flange shall be reduced in the calculations in such an amount, that the condition is satisfied. Alternatively, as a conservative simplification a reduced bending resistance can be applied in the calculations for the gross cross-section as follows:

$$M_{red.c.Rd} = k_{red} \cdot M_{c.Rd} \tag{2.54}$$

$$k_{red} = 0,9 \cdot \frac{\gamma_{M0}}{\gamma_{M2}} \cdot \frac{f_u}{f_y} \cdot \frac{A_{f.net}}{A_f} \qquad but\ \ k_{red} \leq 1,0 \tag{2.55}$$

where $M_{c.Rd}$ is the design bending resistance of the gross cross-section

The fastener holes in the tension region of the web need not be taken into account, if the design condition (2.52) of the tension flange is satisfied for the entire tension region respectively. The tension region consists of the flange in tension and the tension part of the web.

Fastener holes on the compressed region, if the fastener is placed in the hole, need not be taken into account, except when oversized or slotted holes according to EN 1090 [3,4,5,29].

2.6.3 Bending resistance of Class 4 circular hollow section

For a Class 4 circular hollow section, the design bending resistance of the cross-section $M_{c.Rd}$ is calculated on the basis of the local buckling strength $\chi_x f_y$ as follows:

$$M_{c.Rd} = \frac{\chi_x W_{el} f_y}{\gamma_{M1}} \tag{2.56}$$

where

χ_x	is the reduction factor for elastic-plastic local buckling according to clause 2.5.2.2
W_{el}	is the elastic section modulus of the cross-section (Annex 11.1)
f_y	is the nominal yield strength of the material
γ_{M1}	is the partial safety factor for resistance **according to EN 1993-1-6** (Table 2.5)

2.6.4 Bending resistance in biaxial bending

2.6.4.1 Class 1 and 2

When there is no shear force or normal force present, the design condition for Class 1 and 2 in biaxial bending is [3,4,5]:

$$\left[\frac{M_{y.Ed}}{M_{pl.y.Rd}}\right]^{\alpha} + \left[\frac{M_{z.Ed}}{M_{pl.z.Rd}}\right]^{\beta} \leq 1,0 \qquad (2.57)$$

where $\alpha = \beta = 1{,}66$ (square and rectangular hollow sections)
$\alpha = \beta = 2$ (circular hollow sections)
and $M_{pl.y.Rd}$ and $M_{pl.z.Rd}$ are calculated as presented in clause 2.6.1.

2.6.4.2 Class 3

When there is no shear force present, the axial stress $\sigma_{x.Ed}$ is not allowed to exceed the design value of yield strength at any point of the cross-section [3,4,5]:

$$\sigma_{x.Ed} \leq \frac{f_y}{\gamma_{M0}} \qquad (2.58)$$

where f_y is the nominal yield strength of the material
γ_{M0} is the partial safety factor for resistance (Table 2.5)

The fastener holes shall be taken into account when needed, see clause 2.6.2.

On square and rectangular hollow sections, $\sigma_{x.Ed}$ is calculated at the point to be considered first for each bending moment $M_{y.Ed}$ and $M_{z.Ed}$ separately, after which the stresses caused by each moment are summed up. In praxis, the condition (2.58) can be replaced by linear summation of utilisation ratios as follows:

$$\frac{M_{y.Ed}}{M_{el.y.Rd}} + \frac{M_{z.Ed}}{M_{el.z.Rd}} \leq 1,0 \qquad \textit{square and rectangular hollow sections} \qquad (2.59)$$

On circular hollow sections, $\sigma_{x.Ed}$ can be calculated directly using the total moment caused by the moments $M_{y.Ed}$ and $M_{z.Ed}$. The total moment is calculated as a vector sum (in which case the situation is returned to uniaxial bending):

$$M_{Ed} = \sqrt{M_{y.Ed}^2 + M_{z.Ed}^2} \qquad \textit{circular hollow sections} \qquad (2.60)$$

2.6.4.3 Class 4

When there is no shear force present, on square and rectangular hollow sections it shall be verified that the axial stress $\sigma_{x.Ed}$ does not exceed the design value of the yield stress at any point of the cross-section [3,4,5]:

$$\sigma_{x.Ed} \leq \frac{f_y}{\gamma_{M0}} \tag{2.61}$$

where f_y is the nominal yield strength of the material
γ_{M0} is the partial safety factor for resistance (Table 2.5)

The axial stress $\sigma_{x.Ed}$ is determined in Class 4 on the basis of the effective cross-section.

The fastener holes shall be taken into account when needed, see clause 2.6.2.

On circular hollow sections, it shall be verified using the gross cross-section and the total moment $M_{tot.Ed}$ according to expression (2.60), that the axial stress $\sigma_{x.Ed}$ does not exceed the design value of the local buckling strength of the hollow section at any point of the cross-section:

$$\sigma_{x.Ed} \leq \frac{\chi_x f_y}{\gamma_{M1}} \tag{2.62}$$

where χ_x is the reduction factor for elastic-plastic local buckling according to clause 2.5.2.2
f_y is the nominal yield strength of the material
γ_{M1} is the partial safety factor for resistance **according to EN 1993-1-6** (Table 2.5)

Example 2.6
Class 1.

Calculate the bending resistance of a hollow section $140 \times 140 \times 6$.

The steel grade is SSAB Domex Tube Double Grade, which fulfills the EN 10219 requirements for both steel grades S420MH and S355J2H. Thereby the design calculations may be performed at designer's own choice either according to grade S420 or grade S355. Grade S420 is chosen in this Example as design basis.

$W_{pl} = 155{,}3 \cdot 10^3\ mm^3$ *(Annex 11.1)*
$f_y = 420\ N/mm^2$
$\gamma_{M0} = 1{,}0$

Check the cross-section classification (Table 2.9):

Flange:

$b/t = 140/6 = 23,3 \leq 27,7$ *(compression)* $\Rightarrow$ *Class 1*

Web:

$$h/t = 140/6 = 23,3 \leq 56,9 \text{ (bending)} \qquad \Rightarrow \text{Class 1}$$

The whole cross-section shall be classified into Class 1. Thereby the bending resistance can be determined using the plastic section modulus of the cross-section.

The bending resistance of the cross-section:

$$M_{pl.Rd} = \frac{W_{pl} f_y}{\gamma_{M0}} = \frac{155,3 \cdot 10^3 \cdot 420}{1,0} = 65,2 \text{ kNm}$$

Comparison S420 vs S355:
In case the design calculations would be performed according to grade S355, the bending resistance would be 55,1 kNm. In both cases the cross-section classification for bending is Class 1, telling the bending resistance can be determined according to plastic bending resistance $M_{pl.Rd}$. Consequently the increase of the yield strength S355 → S420 can be fully utilised for the bending resistance (= + 18 %).

The values for bending resistance and section modulus are also given in the tables presented in Annex 11.1.

Example 2.7
Class 2.

Calculate the bending resistance of a hollow section 150×150×5.

The steel grade is SSAB Domex Tube Double Grade, which fulfills the EN 10219 requirements for both steel grades S420MH and S355J2H. Thereby the design calculations may be performed at designer's own choice either according to grade S420 or grade S355. Grade S420 is chosen in this Example as design basis.

$W_{pl} = 153,0 \cdot 10^3 \text{ mm}^3$ (Annex 11.1)
$f_y = 420 \text{ N/mm}^2$
$\gamma_{M0} = 1,0$

Check the cross-section classification (Table 2.9):

Flange:

$$b/t = 150/5 = 30,0 \leq 31,4 \text{ (compression)} \qquad \Rightarrow \text{Class 2}$$

Web:

$$h/t = 150/5 = 30,0 \leq 56,9 \text{ (bending)} \qquad \Rightarrow \text{Class 1}$$

The whole cross-section shall be classified into Class 2 according to its most critical plane element. Thereby the bending resistance can be determined using the plastic section modulus of the cross-section.

The bending resistance of the cross-section:

$$M_{pl.Rd} = \frac{W_{pl} f_y}{\gamma_{M0}} = \frac{153,0 \cdot 10^3 \cdot 420}{1,0} = 64,3\ kNm$$

Comparison S420 vs S355:
In case the design calculations would be performed according to grade S355, the bending resistance would be 54,3 kNm. In both cases the cross-section classification for bending is Class 2, telling the bending resistance can be determined according to plastic bending resistance $M_{pl.Rd}$. Consequently the increase of the yield strength S355 → S420 can be fully utilised for the bending resistance (= + 18 %).

Example 2.8
Class 3.

Calculate the bending resistance of a hollow section $160 \times 160 \times 5$.

The steel grade is SSAB Domex Tube Double Grade, which fulfills the EN 10219 requirements for both steel grades S420MH and S355J2H. Thereby the design calculations may be performed at designer's own choice either according to grade S420 or grade S355. Grade S420 is chosen in this Example as design basis.

$W_{el} = 150,3 \cdot 10^3\ mm^3$ *(Annex 11.1)*
$f_y = 420\ N/mm^2$
$\gamma_{M0} = 1,0$

Check the cross-section classification (Table 2.9):

Flange:

$b/t = 160/5 = 32,0 \leq 34,4$ (compressed) $\Rightarrow Class\ 3$

Web:

$h/t = 160/5 = 32,0 \leq 56,9$ (bended) $\Rightarrow Class\ 1$

The whole cross-section shall be classified into Class 3 according to its most critical plane element. Thereby the bending resistance shall be determined using the elastic section modulus of the cross-section.

The bending resistance of the cross-section:

$$M_{el.Rd} = \frac{W_{el} f_y}{\gamma_{M0}} = \frac{150,3 \cdot 10^3 \cdot 420}{1,0} = 63,1\ kNm$$

Comparison S420 vs S355:
In case the design calculations would be performed according to grade S355, the cross-section would turn out to satisfy the conditions for cross-section Class 2. This means that the cross-section's deformation capability (as S355) would be good enough to enable calculations for bending resistance according to plastic bending resistance $M_{pl.Rd}$, resulting in the value 62,2 kNm. Hence we can notice, that in such cases where a change in yield strength

causes a Class2/Class3-change in cross-section classification, the difference of bending resistance between grades S420 and S355 is quite small. However, in certain applications the structural member may need to satisfy cross-section classification for Class 1 or Class 2 (e.g. welded joints of hollow section trusses, see Chapter 3). In such cases, if a hollow section with desired cross-section dimensions does not meet the needed cross-section classification when treated as S420, it still may be possible (when having SSAB Domex Tube Double Grade) to satisfy the requirements by designing the member according to grade S355.

Example 2.9
Class 4 square hollow section.

Calculate the bending resistance of a hollow section 200×200×5.

The steel grade is SSAB Domex Tube Double Grade, which fulfills the EN 10219 requirements for both steel grades S420MH and S355J2H. Thereby the design calculations may be performed at designer's own choice either according to grade S420 or grade S355. Grade S420 is chosen in this Example as design basis.

$A = 3836\ mm^2$ (Annex 11.1)
$I = 2410 \cdot 10^4\ mm^4$ (Annex 11.1)
$f_y = 420\ N/mm^2$
$\gamma_{M0} = 1{,}0$

Check the cross-section classification (Table 2.9):

Flange:

$b/t = 200/5 = 40{,}0 > 34{,}4$ (compression) $\Rightarrow$ Class 4

Web:

$h/t = 200/5 = 40{,}0 \le 56{,}9$ (bending) $\Rightarrow$ Class 1

Since the web belongs to Class 1, it is fully effective.

The whole cross-section shall be classified into Class 4 according to its most critical plane element.

Calculate the effective width of the compressed flange:

$$\bar{b} = h - 3t = 200 - 3 \cdot 5 = 185\ mm$$

$$\psi = 1\ ,\ \ k_\sigma = 4{,}0$$

$$\bar{\lambda}_p = \frac{\bar{b}/t}{28{,}4\varepsilon\sqrt{k_\sigma}} = \frac{(200 - 3 \cdot 5)/5}{28{,}4 \cdot \sqrt{235/420} \cdot \sqrt{4{,}0}} = 0{,}8709$$

$$\bar{\lambda}_p = 0{,}8709 > 0{,}5 + \sqrt{0{,}085 - 0{,}055\psi} = 0{,}5 + \sqrt{0{,}085 - 0{,}055 \cdot 1} = 0{,}6732$$

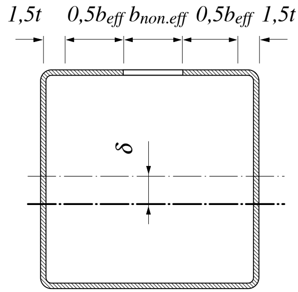
$$\rho = \frac{\bar{\lambda}_p - 0{,}055(3 + \psi)}{\bar{\lambda}_p^2} = \frac{0{,}8709 - 0{,}055 \cdot 4}{0{,}8709^2} = 0{,}8582 \le 1{,}0$$

$$b_{eff} = \rho\bar{b} = 0{,}8582 \cdot 185 = 158{,}8\ mm$$

The non-effective part and non-effective area of the flange are hence:

$$b_{non.eff} = (1-\rho)\bar{b} = (1-0,8582)\cdot 185 = 26,23\ mm$$

$$A_{non.eff} = h_{non.eff}\cdot t = 26,23\cdot 5 = 131,15\ mm^2$$



The neutral axis for the effective cross-section is shifted downwards. The new location of the neutral axis is calculated with expression (2.50):

$$z_{0.eff} = \frac{Az_0 - \sum[A_{non.eff.i}\cdot z_{non.eff.i}]}{A - \sum A_{non.eff.i}}$$

$$= \frac{3836\cdot 100 - 131,15\cdot(200-5/2)}{3836-131,15} = 96,55\ mm$$

The second moment of area for the effective cross-section is calculated with expression (2.51):

$$I_{eff} = I + A(z_0 - z_{0.eff})^2 - \sum I_{non.eff.i} - \sum[A_{non.eff.i}(z_{non.eff.i} - z_{0.eff})^2]$$

$$= 2410\cdot 10^4 + 3836\cdot(100-96,55)^2 - \frac{26,23\cdot 5^3}{12} - 131,15\cdot[(200-5/2)-96,55]^2$$

$$= 2281\cdot 10^4\ mm^4$$

The maximum edge distance is greater on the compressed side of the cross-section. The section modulus for the effective cross-section is therefore:

$$W_{eff.c} = \frac{I_{eff}}{e_c} = \frac{2281\cdot 10^4}{200-96,55} = 220,5\cdot 10^3\ mm^3$$

The bending resistance of the effective cross-section:

$$M_{eff.Rd} = \frac{W_{eff}f_y}{\gamma_{M0}} = \frac{220,5\cdot 10^3\cdot 420}{1,0} = 92,6\ kNm$$

Comparison S420 vs S355:

In case the design calculations would be performed according to grade S355, the bending resistance would be 80,7 kNm (Class 4, $W_{eff} = 227,3\cdot 10^3\ mm^3$). In both cases the cross-section belongs to Class 4, telling the cross-section is not fully effective. Increase of the material strength S355 → S420 further decreases the cross-section's effectiveness. Nevertheless, increase of the yield strength by 18 % improves the bending resistance in this Example by 15 %.

Example 2.10

Class 4 circular hollow section.

Calculate the bending resistance of the circular hollow section 323,9×5 from Example 2.2.

The steel grade is SSAB Domex Tube Double Grade, which fulfills the EN 10219 requirements for both steel grades S420MH and S355J2H. Thereby the design calculations may be performed at designer's own choice either according to grade S420 or grade S355. Grade S420 is chosen in this Example as design basis.

$W_{el} = 393{,}3 \cdot 10^3\ mm^3$ *(Annex 11.1)*
$f_y = 420\ N/mm^2$
$\gamma_{M1} = 1{,}1$ *(Class 4 circular hollow section, EN 1993-1-6)*

The procedure to check the cross-section classification has been presented in Example 2.2.

The buckling strength for hollow section 323,9×5 has been calculated in Example 2.2:

$$\chi_x f_y = 0,8578 \cdot 420 = 360,3\ N/mm^2$$

The bending resistance of the cross-section:

$$M_{c.Rd} = \frac{\chi_x W_{el} f_y}{\gamma_{M1}} = \frac{0,8578 \cdot 393,3 \cdot 10^3 \cdot 420}{1,1} = 128,8\ kNm$$

Comparison S420 vs S355:
In case the design calculations would be performed according to grade S355, the bending resistance would be 111,7 kNm. In both cases we have Class 4 circular cross-section, and increase of the material strength S355 → S420 further increases the cross-section's non-dimensional slenderness. Nevertheless, increase of the yield strength by 18 % improves the bending resistance in this Example by 15 %.

2.6.5 Lateral-torsional buckling resistance

When a member is subject to bending about the stronger axis, the compressed flange may loose its stability by buckling into the lateral direction. At the same time the member rotates about its longitudinal axis. The phenomenon is called lateral-torsional buckling.

Lateral-torsional buckling is influenced by the length of the member, loading, cross-sectional properties, support conditions of the member and the material properties. The higher the cross-section is compared to its width, the more sensitive the beam is to lateral-torsional buckling. A member subject to bending about the weaker axis does not experience lateral-torsional buckling. Within different load types, uniform moment is the most severe.
When compared to open sections such as I- or H-sections, the structural hollow sections have especially good lateral-torsional buckling resistance due to the high torsional stiffness of their box-like cross-section.

In practice, circular or square hollow sections are not susceptible to lateral-torsional buckling, hence their lateral-torsional buckling need not be taken into account [3,4,5].

On rectangular hollow sections however, the lateral-torsional buckling resistance can become governing in design, if long structural hollow sections are used and if their ratio b/h is low. The calculation of lateral-torsional buckling resistance presented here, can be alternatively substituted by checking the distance between the lateral restraints of the compressed flange of the member. The method is presented in clause 2.6.6.

The design condition for lateral-torsional buckling of a member subject to bending is [3,4,5]:

$$M_{Ed} \leq M_{b.Rd} \tag{2.63}$$

where M_{Ed} is the design value of the bending moment at ultimate limit state
$M_{b.Rd}$ is the design lateral-torsional buckling resistance

Part EN 1993-1-1 of Eurocode gives two different methods to calculate the lateral-torsional buckling resistance. Here the more favourable one is presented (see EN 1993-1-1: 6.3.2.3):

The lateral-torsional buckling resistance need not to be determined (i.e. lateral-torsional buckling does not reduce the bending resistance), if the non-dimensional slenderness of the member in lateral-torsional buckling is $\bar{\lambda}_{LT} \leq \bar{\lambda}_{LT.0}$ or if $M_{Ed}/M_{cr} \leq \bar{\lambda}_{LT.0}^2$ [3,4,5]. Calculation of the non-dimensional slenderness $\bar{\lambda}_{LT}$ and the elastic critical lateral-torsional buckling moment M_{cr} is presented later on. The value for the parameter $\bar{\lambda}_{LT.0}$ can be presented in the National Annex. The recommended value given in Eurocode is $\bar{\lambda}_{LT.0} = 0,4$ [3,4,5].

Finnish National Annex to standard EN 1993-1-1 [6]:
For cold-formed structural hollow sections the value $\bar{\lambda}_{LT.0} = 0,4$ is used.

The design value of lateral-torsional buckling resistance $M_{b.Rd}$ is calculated for a laterally unrestrained member subject to bending about stronger axis as follows [3,4,5]:

$$M_{b.Rd} = W_y \frac{\chi_{LT} f_y}{\gamma_{M1}} \tag{2.64}$$

where $W_y = W_{pl.y}$ for Class 1 and 2 (Annex 11.1)
$W_y = W_{el.y}$ for Class 3 (Annex 11.1)
$W_y = W_{eff.y}$ for Class 4
χ_{LT} is the reduction factor for lateral-torsional buckling resistance
f_y is the nominal yield strength of the material
γ_{M1} is the partial safety factor for resistance (Table 2.5)

When calculating the section modulus the fastener holes at the ends of the member need not be taken into account [3,4,5].

The reduction factor χ_{LT} for lateral-torsional buckling resistance is calculated as follows [3,4,5]:

$$\chi_{LT} = \frac{1}{\Phi_{LT} + \sqrt{\Phi_{LT}^2 - \beta\bar{\lambda}_{LT}^2}} \qquad but \begin{cases} \chi_{LT} \leq 1,0 \\ \chi_{LT} \leq \dfrac{1,0}{\bar{\lambda}_{LT}^2} \end{cases} \tag{2.65}$$

$$\Phi_{LT} = 0,5 \cdot [1 + \alpha_{LT}(\bar{\lambda}_{LT} - \bar{\lambda}_{LT.0}) + \beta\bar{\lambda}_{LT}^2] \tag{2.66}$$

where α_{LT} is the imperfection factor of the related lateral-torsional buckling curve
$\bar{\lambda}_{LT}$ is the non-dimensional slenderness of the member in lateral-torsional buckling

The values for parameters $\bar{\lambda}_{LT.0}$ and β needed in expressions (2.65) and (2.66), as well as the lateral-torsional buckling curve and the corresponding imperfection factor α_{LT} to be applied, can be determined in the National Annex. Eurocode does not present any recommended values specifically for structural hollow sections.

Finnish National Annex to the standard EN 1993-1-1 [6]:
For cold-formed structural hollow sections the following values are used:
$\bar{\lambda}_{LT.0} = 0{,}4$
$\beta = 0{,}75$
Lateral-torsional buckling curve and the imperfection factor α_{LT} *are chosen according to Tables 2.13 and 2.14.*

The steel grade (S235-S460) has thus no influence to the determination of the lateral-torsional buckling curve.

Table 2.13 Imperfection factor for lateral-torsional buckling curves [3,4,5]

Lateral-torsional buckling curve	a	b	c	d
Imperfection factor α_{LT}	0,21	0,34	0,49	0,76

Table 2.14 Determination of lateral-torsional buckling curve according to Finnish National Annex [6]

Cross-section	Limits	Lateral-torsional buckling curve
Cold-formed structural hollow sections	$h/b \le 2$ $2 < h/b < 3{,}1$	c d

The reduction factor χ_{LT} for lateral-torsional buckling resistance can be improved by taking into account the form of the moment diagram between the lateral restraints as follows [3,4,5]:

$$\chi_{LT.mod} = \frac{\chi_{LT}}{f} \quad but \quad \begin{cases} \chi_{LT.mod} \le 1,0 \\ \chi_{LT.mod} \le \dfrac{1,0}{\bar{\lambda}_{LT}^2} \end{cases} \tag{2.67}$$

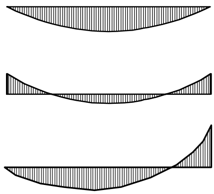
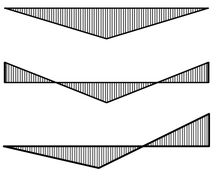
where the value for factor f can be presented in the National Annex. Eurocode recommends the following maximum value [3,4,5]:

$$f = 1 - 0,5(1 - k_c)[1 - 2(\bar{\lambda}_{LT} - 0,8)^2] \le 1,0 \tag{2.68}$$

where k_c is the correction factor according to Table 2.15.

Finnish National Annex to standard EN 1993-1-1 [6]:
The value $f = 1{,}0$ *is used.*

Table 2.15 Correction factor k_c [3,4,5]

Form of the moment plane	k_c
 $\psi = 1$  $-1 \leq \psi \leq 1$	$1,0$ $\dfrac{1}{1,33 - 0,33\,\psi}$
	$0,94$ $0,90$ $0,91$
	$0,86$ $0,77$ $0,82$

The non-dimensional slenderness $\bar{\lambda}_{LT}$ of the member needed in expressions (2.65) - (2.66) is calculated as follows [3,4,5]:

$$\bar{\lambda}_{LT} = \sqrt{\frac{W_y f_y}{M_{cr}}} \qquad (2.69)$$

where W_y is the section modulus according to the cross-section Class (see expression 2.64)
f_y is the nominal yield strength of the material
M_{cr} is the elastic critical lateral-torsional buckling moment

The elastic critical lateral-torsional buckling moment M_{cr} is calculated for the rectangular hollow section subject to bending about the stronger axis as follows. The properties of the cross-section are determined in all cross-section Classes according to gross cross-section [3,4,5,30]:

$$M_{cr} = C_1 \frac{\pi^2 EI_z}{(kL)^2}\left[\sqrt{\frac{(kL)^2 GI_t}{\pi^2 EI_z} + (C_2 z_g)^2} - C_2 z_g\right] \qquad (2.70)$$

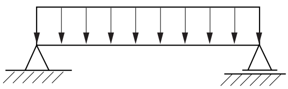
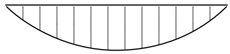
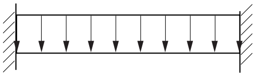
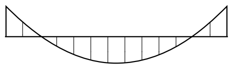
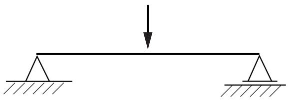
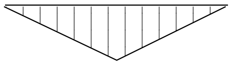
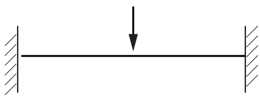
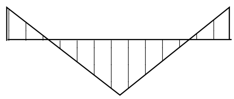
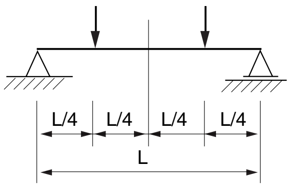
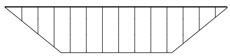
where	C_1 and C_2	are coefficients that depend on the loading (Table 2.17)
	I_z	is the second moment of area about the weaker axis (Annex 11.1)
	I_t	is the St. Venant torsional constant (Annex 11.1)
	E	is the Young's modulus of elasticity
	G	is the shear modulus
	L	is the length of the member between the laterally supported points (distance between the lateral restraints)
	k	is the effective length factor depending on the support conditions at the ends of the member (Table 2.16)
	$z_g = + h/2$	when the load acts downwards at the top flange of the hollow section (i.e. towards the torsional center, destabilizing effect)
	$z_g = 0$	when the member is subject to only a bending moment, which is uniform or linearly varying between the ends of the member
	$z_g = - h/2$	when the load acts downwards at the bottom flange of the hollow section (i.e. away from the torsional center, stabilizing effect

In the formula (2.70) it is assumed that both ends of the member are supported so that the lateral displacement and the rotation about the longitudinal axis are prevented (so-called fork bearing). Other support conditions and the type of loading (i.e. form of the moment diagram) are taken into account in expression (2.70) by the various parameters appearing in it (see Tables 2.16 and 2.17). The effect of warping has not been taken into account, because its effect on hollow sections is small.

Table 2.16 Lateral-torsional buckling of a rectangular hollow section. Effective length factors for different support conditions [30,31]

Support conditions at the ends of the member against the rotation about the vertical axis	
Rotation prevented at both ends of the member	k = 0,5
Rotation free at both ends of the member	k = 1,0
Rotation prevented at one end of the member, the other end is free to rotate	k = 0,7

Table 2.17 Values of the factors C_1 and C_2 corresponding to the effective length factor k [30]

Loading and support conditions	Bending moment diagram	k	Values of factors	
			C_1	C_2
		1,0 0,5	1,132 0,972	0,459 0,304
		1,0 0,5	1,285 0,712	1,562 0,652
		1,0 0,5	1,365 1,070	0,553 0,432
		1,0 0,5	1,565 0,938	1,267 0,715
		1,0 0,5	1,046 1,010	0,430 0,410

2.6.6 Prevention of lateral-torsional buckling by using lateral bracing of the member

2.6.6.1 Bracing distance (non-braced maximum length of the member)

As stated before, circular or square hollow sections are in practice not susceptible to lateral-torsional buckling, hence their lateral-torsional buckling need not be taken into account [3,4,5].

On rectangular hollow sections however, the lateral-torsional buckling resistance can become governing in design. Calculation of the lateral-torsional buckling resistance can be alternatively substituted by checking the distance between the lateral restraints of the compressed flange of the member as follows:

According to Eurocode, lateral-torsional buckling does not reduce the bending resistance of the member, if its non-dimensional slenderness in lateral-torsional buckling is $\bar{\lambda}_{LT} \leq 0{,}4$. Based on that condition, a conservative value can be derived from expressions (2.69) and (2.70) for the maximum distance L_c between the lateral restraints for a rectangular hollow section:

$$L_c \leq \frac{0,4^2}{f_y W_y} \cdot \frac{\pi}{k}\sqrt{(EI_z)(GI_t)} \qquad (2.71)$$

where k is the effective length factor depending on the support conditions at the ends of the member (Table 2.16)
E is the Young's modulus of elasticity
G is the shear modulus
I_z is the second moment of area about the weaker axis (Annex 11.1)
I_t is the St. Venant torsional constant (Annex 11.1)
f_y is the nominal yield strength of the material
W_y is the section modulus according to cross-section Class (see expression 2.64)

In expression (2.71) it is assumed that there is fork bearing at the ends of the member and the member is subject to uniform moment, which is within the different loadings the most critical one when considering lateral-torsional buckling.

From expression (2.71), the following approximation can be derived for Class 3 hollow sections:

$$\frac{L_c}{h-t} \leq 0,4^2 \cdot \frac{\pi}{k} \cdot \frac{\sqrt{3EG}}{f_y} \cdot \frac{\gamma^2}{1+3\gamma} \cdot \sqrt{\frac{3+\gamma}{1+\gamma}} \qquad (2.72)$$

$$\gamma = \frac{b-t}{h-t} \qquad (2.73)$$

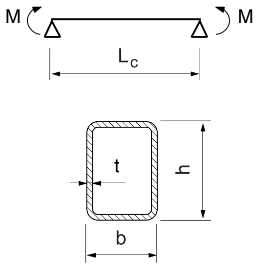
where k is the effective length factor depending on the support conditions at the ends of the member (Table 2.16)
E is the Young's modulus of elasticity
G is the shear modulus
b is the width of the rectangular hollow section
h is the depth of the rectangular hollow section
t is the wall thickness

The limit values for the distance between the bracing of the compressed flange according to expression (2.72) are presented in Table 2.18 and in Figure 2.6 for Class 3 rectangular hollow sections, wherein the effective length factor k has conservatively been taken as k = 1,0 .

In Classes 1 and 2, an approximation corresponding to expression (2.72) and Table 2.18 is obtained by reducing the limit value obtained from them by the ratio of the elastic and plastic section moduli $W_{el.y} / W_{pl.y}$.

If the above presented condition for the bracing distance L_c according to the related cross-section Class is not satisfied, the lateral-torsional buckling resistance of rectangular hollow sections need to be checked more accurately according to clause 2.6.5.

Table 2.18 Limit values for distance L_c between lateral restraints of compressed flange of Class 3 rectangular hollow sections, below which lateral-torsional buckling need not be taken account



$\frac{b-t}{h-t}$	$\frac{L_c}{h-t}$				
	S235	S275	**S355**	**S420**	S460
0,25	27,8	23,8	**18,4**	**15,6**	14,2
0,33	41,8	35,8	**27,7**	**23,4**	21,4
0,40	54,8	46,8	**36,3**	**30,6**	28,0
0,50	73,8	63,1	**48,9**	**41,3**	37,7
0,60	93,2	79,6	**61,7**	**52,1**	47,6
0,70	112,7	96,3	**74,6**	**63,0**	57,6
0,80	132,2	112,9	**87,5**	**73,9**	67,5
0,90	151,5	129,5	**100,3**	**84,8**	77,4
1,00 [a)]	170,8	146,0	**113,1**	**95,6**	87,3

a) According to EN 1993-1-1, square hollow sections are not susceptible to lateral-torsional buckling.

The values in the table have been determined for a fork-bearing-supported member, and by using a conservative value k = 1,0 for the effective length factor.
Additionally it has been assumed that the member is subject to uniform moment which is within the different loadings the most critical one when considering lateral-torsional buckling.
For Classes 1 and 2 the limit values are obtained by multiplying the values in the table by the ratio $W_{el.y} / W_{pl.y}$.

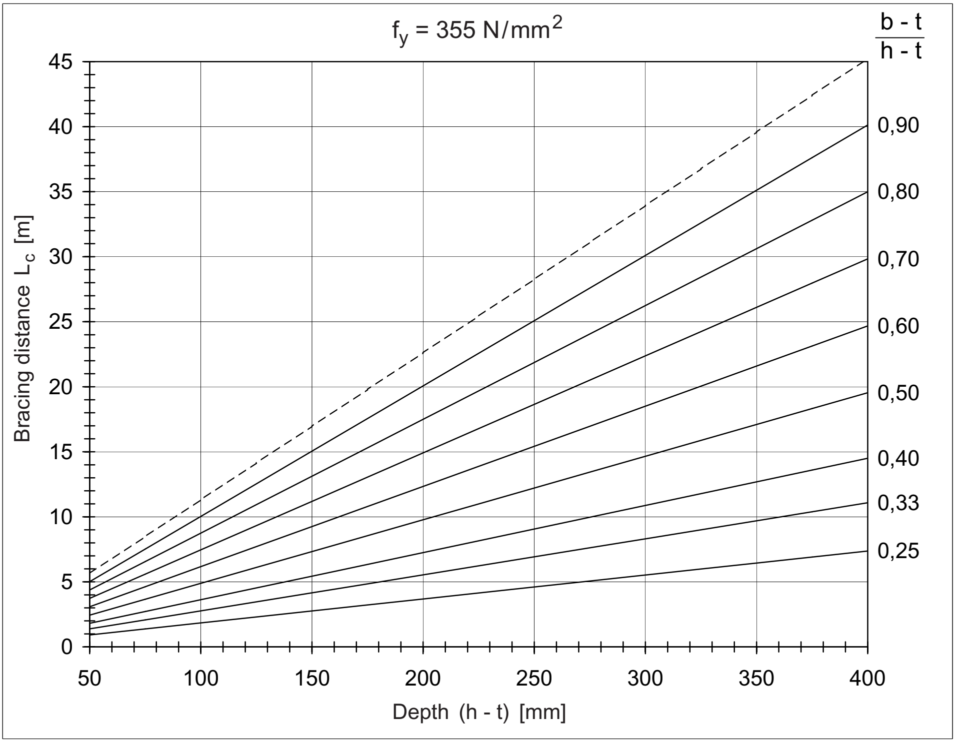


Figure 2.6a Limit value curves for lateral-torsional buckling corresponding to Table 2.18 (f_y = 355 N/mm^2)

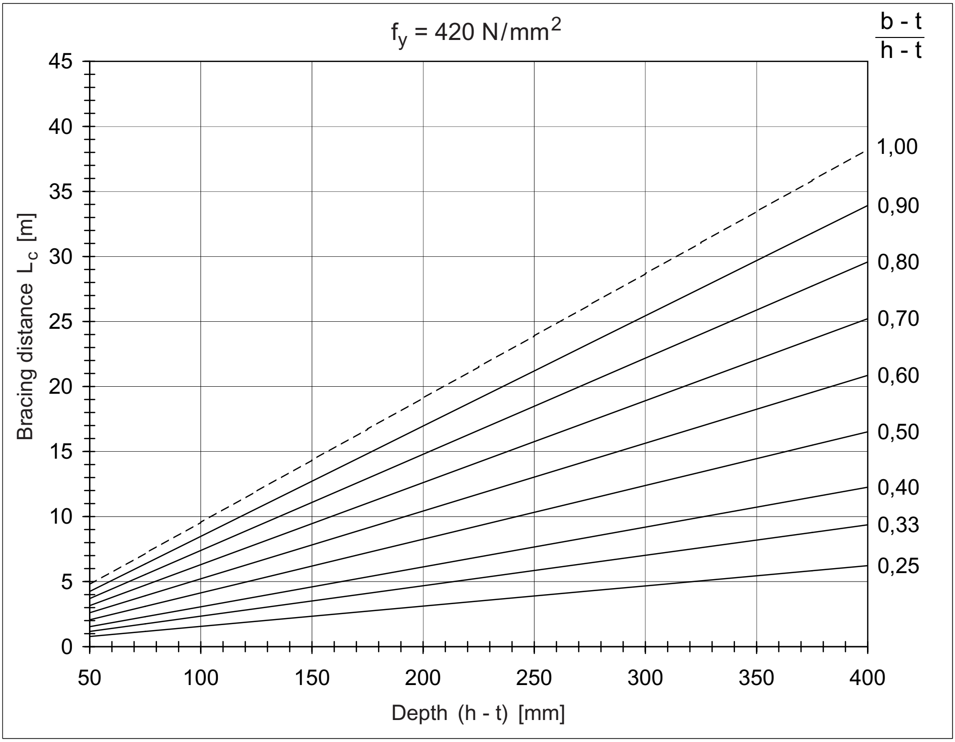


Figure 2.6b Limit value curves for lateral-torsional buckling corresponding to Table 2.18 (f_y = 420 N/mm^2)

2.6.6.2 Lateral bracing when the structure contains plastic hinges

The global analysis of the structure may be performed on the basis of the plastic theory provided that lateral-torsional buckling of the different parts of the frame is prevented in the following ways [3,4,5]:

- lateral restraints are placed at 'rotated' plastic hinges, and

- it shall be verified that the distance between the aforementioned lateral restraints and other lateral restraints does not exceed the stable length L_m. Because there are no instructions given in Eurocode to calculate the stable length L_m for structural hollow sections, it is recommended to use the limit value of the lateral bracing distance according to expression (2.71) (as a section modulus the plastic section modulus of Class 1 is now applied).

In case that it can be verified, that at every ultimate limit state load combination the plastic hinge does not 'rotate', the lateral restraints are not needed in that particular hinge [3,4,5].

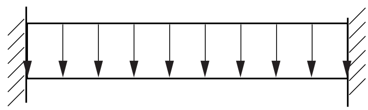
If it is not practical to place the lateral restraint directly at the plastic hinge, it may be placed at a distance of not more than $h/2$ from the plastic hinge in the direction of the member axis [3,4,5].

The design of lateral bracing is presented more thoroughly in EN 1993-1-1.

Example 2.11

Check the lateral-torsional buckling of a hollow section 300×100×5. The length of the beam is 7 m and it has a rigid connection to a hollow section column at both ends in such a way, that the displacements and rotations are fully restrained about all axes. The beam is subjected to uniform load on the top flange about the major axis of the section.

The steel grade is SSAB Domex Tube Double Grade, which fulfills the EN 10219 requirements for both steel grades S420MH and S355J2H. Thereby the design calculations may be performed at designer's own choice either according to grade S420 or grade S355. Grade S420 is chosen in this Example as design basis.



$I_t = 2044 \cdot 10^4\ mm^4$ *(Annex 11.1)*
$I_z = 722{,}8 \cdot 10^4\ mm^3$ *(Annex 11.1)*
$W_{el.y} = 271{,}0 \cdot 10^3\ mm^3$ *(Annex 11.1)*
$W_{pl.y} = 348{,}2 \cdot 10^3\ mm^3$ *(Annex 11.1)*
$f_y = 420\ N/mm^2$
$\gamma_{M0} = 1{,}0$
$\gamma_{M1} = 1{,}0$

Check the cross-section classification (Table 2.9):

Flange:

$b/t = 100/5 = 20,0 \leq 27,7$ *(compression)* $\Rightarrow$ *Class 1*

Web:

$h/t = 300/5 = 60,0 \leq 65,1$ *(bending)* $\Rightarrow$ *Class 2*

The whole cross-section shall be classified into Class 2.

The bending resistance of the cross-section:

$$M_{pl.Rd} = \frac{W_{pl} f_y}{\gamma_{M0}} = \frac{348,2 \cdot 10^3 \cdot 420}{1,0} = 146,2\ kNm$$

Calculate the elastic critical moment for lateral-torsional buckling:
$k = 0{,}5$ *(Table 2.16)*
$C_1 = 0{,}712$ *(Table 2.17)*
$C_2 = 0{,}652$ *(Table 2.17)*
$z_g = h/2 = 300/2 = 150\ mm$ *(expression 2.70)*

$$M_{cr} = C_1 \frac{\pi^2 EI_z}{(kL)^2}\left[\sqrt{\frac{(kL)^2 GI_t}{\pi^2 EI_z} + (C_2 z_g)^2} - C_2 z_g\right]$$

$$= 0,712 \cdot \frac{\pi^2 \cdot 210000 \cdot 722,8 \cdot 10^4}{(0,5 \cdot 7000)^2} \cdot$$

$$\left[\sqrt{\frac{(0,5 \cdot 7000)^2 \cdot 81000 \cdot 2044 \cdot 10^4}{\pi^2 \cdot 210000 \cdot 722,8 \cdot 10^4} + (0,652 \cdot 150)^2} - (0,652 \cdot 150)\right]$$

$$= 931,7\ kNm$$

Non-dimensional slenderness of the member for lateral-torsional buckling:

$$\bar{\lambda}_{LT} = \sqrt{\frac{W_y f_y}{M_{cr}}} = \sqrt{\frac{348,2 \cdot 10^3 \cdot 420}{931,7 \cdot 10^6}} = 0,3962 \le 0,4 \quad \Rightarrow$$ *lateral-torsional buckling doesn't reduce the bending resistance*

Corresponding result can be obtained also by using the maximum bracing distance L_c:

A conservative approximation for maximum bracing distance L_c is obtained from Table 2.18 by applying it for cross-section Class 2 as follows:

$$\gamma = \frac{b-t}{h-t} = \frac{100-5}{300-5} = 0,32 \sim 0,33$$

$$\frac{L_c}{h-t} = 23,4 \cdot \frac{W_{el.y}}{W_{pl.y}} = 23,4 \cdot \frac{271,0 \cdot 10^3}{348,2 \cdot 10^3} = 18,2$$

$$L_c = 18,2 \cdot (h-t) = 18,2 \cdot (300-5) = 5,4\ m \le L = 7\ m$$

$\Rightarrow$ *since the actual length exceeds the approximated value of the maximum bracing distance, a more accurate value for the maximum bracing distance needs to be calculated from expression (2.71). This way also the restraints at beam ends (factor k) can be taken into account according to actual situation:*

$$L_c = \frac{0,4^2}{f_y W_y} \cdot \frac{\pi}{k}\sqrt{(EI_z)(GI_t)}$$

$$= \frac{0,16}{420 \cdot 348,2 \cdot 10^3} \cdot \frac{\pi}{0,5} \cdot \sqrt{(210000 \cdot 722,8 \cdot 10^4) \cdot (81000 \cdot 2044 \cdot 10^4)}$$

$$= 10,9\ m \ge L = 7\ m \quad \Rightarrow$$ *lateral torsional-buckling doesn't reduce the bending resistance*

Comparison S420 vs S355:

According to grade S420 the hollow section shall be classified into cross-section Class 2, and according to grade S355 into cross-section Class 1 respectively. In both cases the bending resistance of the cross-section can be determined according to plastic bending resistance. Consequently the increase of the yield strength S355 $\rightarrow$ S420 can be fully utilised for the *cross-section's bending resistance (= + 18 %). Regarding lateral-torsional buckling, expression (2.71) shows that when designing according to grade S420, the distance between lateral restraints needs to be shortened in same respect with the yield strengths, in order to avoid the lateral-torsional buckling to reduce the cross-section's increased bending resistance obtained by the higher yield strength.*

2.7 Resistance of a structural hollow section subject to shear force

The provisions presented herein are assigned for determining the shear resistance of beam elements. The shear resistance in respect to beam-to-column joints or lattice joints; see EN 1993-1-8 and Chapter 3 of this handbook.

The design condition for shear resistance can be presented in the form:

$$V_{Ed} \leq V_{Rd} \tag{2.74}$$

where V_{Ed} is the design value of the shear force at ultimate limit state, on circular hollow sections subject to shear forces $V_{y.Ed}$ and $V_{z.Ed}$ the total shear force is $V_{Ed} = \sqrt{V_{y.Ed}^2 + V_{z.Ed}^2}$

V_{Rd} is the smaller of the following values: plastic shear resistance $V_{pl.Rd}$ or shear buckling resistance $V_{b.Rd}$

2.7.1 Shear resistance of square and rectangular hollow sections

The procedure to determine the shear resistance depends on the slenderness of those webs in the cross-section, which are parallel to the applied load. The web's slenderness limit given in Parts EN 1993-1-1 and EN 1993-1-5 of Eurocode, is presented therein using the cross-sectional measures applied for welded sections. The corresponding web's slenderness limit is applied for structural hollow sections as follows:

$$\frac{h}{t} \leq \frac{72\varepsilon}{\eta} + 3 \quad \Rightarrow \quad \textit{calculate plastic shear resistance (clause 2.7.1.1)} \tag{2.75a}$$

$$\frac{h}{t} > \frac{72\varepsilon}{\eta} + 3 \quad \Rightarrow \begin{cases} \text{- calculate shear buckling resistance (clause 2.7.1.2)} \\ \text{- the web shall be provided with transverse stiffeners} \\ \quad \text{at the supports} \end{cases} \tag{2.75b}$$

where h is the depth of the hollow section in the direction of the shear force

t is the wall thickness

$\varepsilon = \sqrt{235/f_y} \quad [f_y] = N/mm^2$

f_y is the nominal yield strength of the material

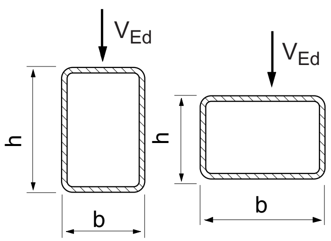
η is the factor taking the strain hardening into account

Strain hardening is not utilised when calculating shear resistance for structural hollow sections. Therefore, when determining the web's slenderness limit using expression (2.75), the slenderness of the web will not be reduced with the factor η, i.e. the value $\eta = 1,0$ can be applied [3,4,5].

The web's slenderness limit according to expression (2.75) is presented in Table 2.19 based on the above presented value $\eta = 1,0$. In practice only on a few structural hollow sections shear buckling turns out to be governing in design.

If the slenderness limit of the web does not require the section to be equipped with transverse stiffeners at supports, the resistance of the unstiffened web to the support reaction force shall be checked at end support according to clause 2.11.

Table 2.19 Limit values for web's slenderness. If exceeded, the shear buckling resistance will become governing

	$\frac{h}{t}$				
	S235	S275	**S355**	**S420**	S460
	75,0	69,6	**61,6**	**56,9**	54,5
The values in the table have been determined by using $\eta = 1,0$ for structural hollow sections [3,4,5].					

2.7.1.1 Plastic shear resistance of square and rectangular hollow sections

When there is no torsion present, the shear resistance according to the theory of plasticity $V_{pl.Rd}$ is calculated from the formula [3,4,5]:

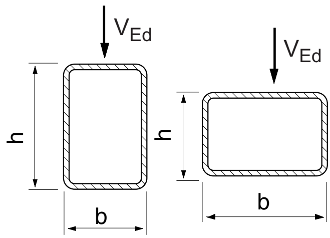
$$V_{pl.Rd} = A_V \cdot \frac{f_y / \sqrt{3}}{\gamma_{M0}} \tag{2.76}$$

where A_V is the shear area
f_y is the nominal yield strength of the material
γ_{M0} is the partial safety factor for resistance (Table 2.5)

The shear area is [3,4,5]:

$$A_V = A \cdot \frac{h}{b+h} \tag{2.77}$$

where A is the cross-section area (Annex 11.1)
h is the depth of the hollow section in the direction of the shear force
b is the width of the hollow section



According to EN 1993-1-1 the fastener holes need not be taken into account when calculating the shear resistance, except when calculating the shear resistance at connection zones according to EN 1993-1-8 [3,4,5]. In EN 1993-1-8 there are, however, no general instructions how to take the hole deduction into account when calculating the shear resistance. In this handbook it is recommended, that regarding the hole deduction, the following method can be used:

At the fastener holes expression (2.77) is substituted by the following expression when the fastener hole is in the plane element carrying the shear load (i.e. in the plane element parallel to the shear load):

$$A_V = A \cdot \frac{h}{b+h} - \sum(d_0 t) \tag{2.78}$$

where A is the cross-section area (Annex 11.1)
h is the depth of the hollow section in the direction of the shear force
b is the width of the hollow section
d_0 is the diameter of the hole in the plane element carrying the shear force
t is the wall thickness

2.7.1.2 Shear buckling resistance of square and rectangular hollow sections

The determination of shear buckling resistance is presented in Part EN 1993-1-5 of Eurocode [13,14]. Shear buckling resistance consists of the contribution from the web and the contribution from the flanges. The contribution from the web, on its behalf, depends on the slenderness of the web and the way of stiffening. Thereby the procedure becomes rather complex.

Because on hollow sections the shear buckling resistance becomes critical very rarely, the presentation is limited herein to an EN 1993-1-5 based conservative and simplified method, where the contribution from the flanges has not been taken into account, and the contribution from the vertical stiffeners of the web has been considered according to the weakest stiffening category.

Thereby the shear buckling resistance of unstiffened and stiffened webs $V_{b.Rd}$ can be calculated from the following formula:

$$V_{b.Rd} = V_{bw.Rd} = A_V \cdot \frac{\chi_w f_y}{\sqrt{3}\,\gamma_{M1}} \tag{2.79}$$

where $V_{bw.Rd}$ is the contribution from the web to shear buckling resistance
χ_w is the reduction factor for the contribution from the web (Table 2.20)
A_V is the shear area according to expression (2.77)
f_y is the nominal yield strength of the material
γ_{M1} is the partial safety factor for resistance (Table 2.5)

The reduction factor χ_w for the contribution from the web is determined from Table 2.20.

Table 2.20 Contribution from the web to shear buckling resistance. Reduction factor χ_w

$\bar{\lambda}_w < 0{,}83$	1,00
$0{,}83 \leq \bar{\lambda}_w < 1{,}08$	$0{,}83 / \bar{\lambda}_w$
$\bar{\lambda}_w \geq 1{,}08$	$0{,}83 / \bar{\lambda}_w$

The non-dimensional slenderness of the web $\bar{\lambda}_w$ needed in Table 2.20 is calculated as follows:

$$\bar{\lambda}_w = \frac{\bar{b}/t}{86{,}4\varepsilon} = \frac{(h-3t)/t}{86{,}4\varepsilon} \tag{2.80}$$

where h is the depth of the hollow section in the direction of shear force
t is the wall thickness
ε $= \sqrt{235/f_y} \quad [f_y] = N/mm^2$
f_y is the nominal yield strength of the material

2.7.2 Shear resistance of circular hollow sections

The procedure to calculate shear resistance of circular hollow sections is determined as follows:

Class 1, 2 and 3 $\Rightarrow$ calculate plastic shear resistance (clause 2.7.2.1)
Class 4 $\Rightarrow$ calculate shear buckling resistance (clause 2.7.2.2)

2.7.2.1 Plastic shear resistance of circular hollow sections

When there is no torsion present, the shear resistance according to the theory of plasticity $V_{pl.Rd}$ is calculated from the formula [3,4,5]:

$$V_{pl.Rd} = A_V \cdot \frac{f_y/\sqrt{3}}{\gamma_{M0}} \tag{2.81}$$

where A_V is the shear area
f_y is the nominal yield strength of the material
γ_{M0} is the partial safety factor for resistance (Table 2.5)

The shear area is [3,4,5]:

$$A_V = A\frac{2}{\pi} \tag{2.82}$$

where A is the cross-section area (Annex 11.1)

2.7.2.2 Shear buckling resistance of circular hollow sections

The design shear buckling resistance of the cross-section $V_{b.Rd}$ is determined on the basis of shear buckling strength $\chi_\tau f_y/\sqrt{3}$ as follows, when there is an end-plate or a rigid ring welded at both ends of the hollow section that prevent the deformation of the cross-section (or the hollow section is welded at its ends to the surrounding structure in the corresponding way) [16]:

$$V_{b.Rd} = A_{bV} \cdot \frac{\chi_\tau f_y/\sqrt{3}}{\gamma_{M1}} \tag{2.83}$$

$$A_{bV} = \pi r_m t \tag{2.84}$$

where
- A_{bV} is the calculatory area for shear buckling
- r_m is the radius of the centreline of the hollow section wall thickness
- t is the wall thickness
- χ_τ is the reduction factor for elastic-plastic shear buckling of a shell
- f_y is the nominal yield strength of the material
- γ_{M1} is the partial safety factor for resistance **according to EN 1993-1-6** (Table 2.5)

The reduction factor χ_τ for elastic-plastic shear buckling is calculated as follows, cf. clause 2.5.2.2 [16]:

$$\chi_\tau = 1,0 \qquad \text{for } \bar{\lambda}_\tau \le 0,4 \tag{2.85a}$$

$$\chi_\tau = 1 - 0,6\frac{\bar{\lambda}_\tau - 0,4}{\bar{\lambda}_{pl.\tau} - 0,4} \qquad \text{for } 0,4 < \bar{\lambda}_\tau < \bar{\lambda}_{pl.\tau} \tag{2.85b}$$

$$\chi_\tau = \frac{\alpha_\tau}{\bar{\lambda}_\tau^2} \qquad \text{for } \bar{\lambda}_\tau \ge \bar{\lambda}_{pl.\tau} \tag{2.85c}$$

where
- α_τ is the elastic imperfection reduction factor for shear buckling (Table 2.21)
- $\bar{\lambda}_\tau$ is the non-dimensional slenderness for shear buckling
- $\bar{\lambda}_{pl.\tau}$ is the value of plastic limit non-dimensional slenderness for shear buckling

The elastic imperfection reduction factor α_τ is determined from the fabrication tolerance quality classes A...C (A is the best) given in EN 1090-2 [29]. On circular hollow sections the most essential geometrical tolerance is usually the tolerance for out-of-roundness. When the external diameter is $d \leq$ 400 mm and $d/t \leq$ 100, the SSAB hollow sections meet the out-of-roundness quality class B (Tables 1.2 and 2.12). Thereby the elastic imperfection reduction factor α_τ obtains the value α_τ = 0,65 (Table 2.21). With other dimensions, the applicable fabrication tolerance quality class shall be checked case by case.

Table 2.21 Elastic imperfection reduction factor α_τ for shear buckling [16]

Fabrication tolerance quality class	Description	α_τ
Class A	Excellent	0,75
Class B	High	0,65
Class C	Normal	0,50

The plastic limit non-dimensional slenderness $\bar{\lambda}_{pl.\tau}$ for shear buckling is [16]:

$$\bar{\lambda}_{pl.\tau} = \sqrt{\frac{\alpha_\tau}{0,4}} \tag{2.86}$$

Non-dimensional slenderness $\bar{\lambda}_\tau$ for shear buckling is [16]:

$$\bar{\lambda}_\tau = \sqrt{\frac{f_y / \sqrt{3}}{\tau_{cr}}} \tag{2.87}$$

where f_y is the nominal yield strength of the material
τ_{cr} is the elastic critical shear buckling stress

The elastic critical shear buckling stress of circular hollow section τ_{cr} is calculated as follows [16]:

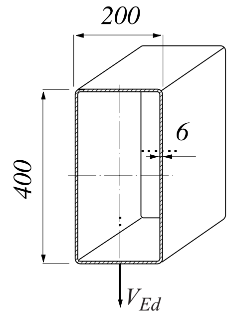
$$\tau_{cr} = 0,75EC_\tau \cdot \sqrt{\frac{t}{L}} \cdot \left[\frac{t}{r_m}\right]^{3/4} \tag{2.88}$$

where E is the Young's modulus of elasticity
L is the length of the hollow section
t is the wall thickness
r_m is the radius of the centreline of the hollow section wall thickness

EN 1993-1-6 defines the factor C_τ as a function of the length of the cylinder-like structure considered. As a conservative simplification the value C_τ = 1,0 can be used.

Example 2.12

Calculate the shear resistance of a hollow section 400×200×6.



The steel grade is SSAB Domex Tube Double Grade, which fulfills the EN 10219 requirements for both steel grades S420MH and S355J2H. Thereby the design calculations may be performed at designer's own choice either according to grade S420 or grade S355. Grade S420 is chosen in this Example as design basis.

$A = 6963\ mm^2$ *(Annex 11.1)*
$f_y = 420\ N/mm^2$
$\gamma_{M1} = 1,0$

First, check whether the shear buckling of the web needs to be considered. For the hollow section chosen in this Example we get:

$$\frac{h}{t} = \frac{400}{6} = 66,67 > \frac{72\varepsilon}{\eta} + 3 = \frac{72\sqrt{235/420}}{1,0} + 3 = 56,86$$

⇒ *shear buckling of the web needs to be considered and the web has to be provided with transverse stiffeners at the supports.*

Non-dimensional slenderness of the web:

$$\bar{\lambda}_w = \frac{\bar{b}/t}{86,4\varepsilon} = \frac{(h-3t)/t}{86,4\varepsilon} = \frac{(400-3\cdot 6)/6}{86,4\cdot\sqrt{235/420}} = 0,9851$$

$$0,83 \le \bar{\lambda}_w < 1,08$$

The reduction factor of the web to buckling is obtained from Table 2.20:

$$\chi_w = 0,83/\bar{\lambda}_w = 0,83/0,9851 = 0,8426$$

The shear area of the hollow section:

$$A_V = A\cdot\frac{h}{b+h} = 6963\cdot\frac{400}{200+400} = 4642\ mm^2$$

The shear buckling resistance of the hollow section:

$$V_{b.Rd} = V_{bw.Rd} = A_V\cdot\frac{\chi_w f_y}{\sqrt{3}\,\gamma_{M1}} = 4642\cdot\frac{0,8426\cdot 420}{\sqrt{3}\cdot 1,0} = 948,5\ kN$$

Comparison S420 vs S355:
In case the design calculations would be performed according to grade S355, the shear buckling resistance of the hollow section would be 871,9 kN. In both cases shear buckling is the critical failure mode, and increase of the material strength S355 → S420 further increases the web's non-dimensional slenderness. Nevertheless, increase of the yield strength by 18 % improves the shear buckling resistance in this Example by 9 %.

Example 2.13

Calculate the shear resistance of a hollow section 400×200×8 when subjected to a shear load in stronger direction.

The steel grade is SSAB Domex Tube Double Grade, which fulfills the EN 10219 requirements for both steel grades S420MH and S355J2H. Thereby the design calculations may be performed at designer's own choice either according to grade S420 or grade S355. Grade S420 is chosen in this Example as design basis.

$A \quad = 9124\ mm^2$ *(Annex 11.1)*
$f_y \quad = 420\ N/mm^2$
$\gamma_{M0} = 1,0$

$$\frac{h}{t} = \frac{400}{8} = 50 \le \frac{72\varepsilon}{\eta} + 3 = \frac{72 \cdot \sqrt{235/420}}{1,0} + 3 = 56,86$$

⇒ *the shear buckling of the web does not need to be considered*

Calculate the plastic shear resistance of the hollow section:

The shear area of the hollow section:

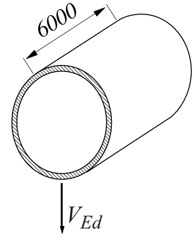
$$A_V = A \cdot \frac{h}{b+h} = 9124 \cdot \frac{400}{200+400} = 6083\ mm^2$$

The plastic shear resistance:

$$V_{pl.Rd} = A_V \cdot \frac{f_y/\sqrt{3}}{\gamma_{M0}} = 6083 \cdot \frac{420/\sqrt{3}}{1,0} = 1475\ kN$$

Comparison S420 vs S355:
In case the design calculations would be performed according to grade S355, the shear resistance would be 1247 kN. In both cases the shear resistance can be determined according to plastic shear resistance $V_{pl.Rd}$. Consequently the increase of the yield strength S355 → S420 can be fully utilised for the shear resistance (= + 18 %).

Example 2.14



Calculate the shear resistance of the Class 4 circular hollow section 323,9×5 from Example 2.2, when its length is 6 m. Shear force is assumed to be constant within the whole length.

The steel grade is SSAB Domex Tube Double Grade, which fulfills the EN 10219 requirements for both steel grades S420MH and S355J2H. Thereby the design calculations may be performed at designer's own choice either according to grade S420 or grade S355. Grade S420 is chosen in this Example as design basis.

f_y = 420 N/mm^2
γ_{M1} = 1,1 (Class 4 circular hollow section, EN 1993-1-6)

On circular hollow section, the shear resistance shall be determined according to shear buckling resistance if the cross-section is classified into Class 4.

The critical shear buckling stress and non-dimensional slenderness:

$$\tau_{cr} = 0,75EC_\tau \cdot \sqrt{\frac{t}{L}} \cdot \left[\frac{t}{r_m}\right]^{3/4} = 0,75 \cdot 210000 \cdot 1,0 \cdot \sqrt{\frac{5}{6000}} \cdot \left[\frac{5}{159,5}\right]^{3/4} = 338,7 N/mm^2$$

$$\bar{\lambda}_\tau = \sqrt{\frac{f_y/\sqrt{3}}{\tau_{cr}}} = \sqrt{\frac{420/\sqrt{3}}{338,7}} = 0,8461$$

The hollow section has d ≤ 400 mm and d/t≤ 100 whereby it satisfies the requirements specified for the fabrication tolerance quality class B (Tables 1.2 and 2.11)

$\Rightarrow \alpha_\tau = 0,65$ (Table 2.21)

$$\bar{\lambda}_{pl.\tau} = \sqrt{\frac{\alpha_\tau}{0,4}} = \sqrt{\frac{0,65}{0,4}} = 1,275$$

$$0,4 < \bar{\lambda}_\tau < \bar{\lambda}_{pl.\tau}$$

$\Rightarrow$

$$\chi_\tau = 1 - 0,6\frac{\bar{\lambda}_\tau - 0,4}{\bar{\lambda}_{pl.\tau} - 0,4} = 1 - 0,6 \cdot \frac{0,8461 - 0,4}{1,275 - 0,4} = 0,6941$$

The calculatory area for shear buckling:

$$A_{bV} = \pi r_m t = \pi \cdot 159,5 \cdot 5 = 2505\ mm^2$$

The shear buckling resistance is finally:

$$V_{b.Rd} = A_{bV} \cdot \frac{\chi_\tau f_y/\sqrt{3}}{\gamma_{M1}} = 2505 \cdot \frac{0,6941 \cdot 420/\sqrt{3}}{1,1} = 383,3\ kN$$

Comparison S420 vs S355:
In case the design calculations would be performed according to grade S355, the shear buckling resistance of the hollow section would be 345,8 kN. In both cases shear buckling is the critical failure mode, and increase of the material strength S355 → S420 further increases the cross-section's non-dimensional slenderness. Nevertheless, increase of the yield strength by 18 % improves the shear buckling resistance in this Example by 11 %.

Example 2.15

Calculate the shear resistance of a circular hollow section 323,9×8.

The steel grade is SSAB Domex Tube Double Grade, which fulfills the EN 10219 requirements for both steel grades S420MH and S355J2H. Thereby the design calculations may be performed at designer's own choice either according to grade S420 or grade S355. Grade S420 is chosen in this Example as design basis.

On circular hollow section, the shear resistance shall be determined according to plastic shear resistance if the cross-section is classified into Class 3 ($d/t = 40,5 \leq 50,4$):

$A = 7939\ mm^2$ *(Annex 11.1)*
$f_y = 420\ N/mm^2$
$\gamma_{M0} = 1,0$

The calculatory shear area:

$$A_V = A\frac{2}{\pi} = 7939 \cdot \frac{2}{\pi} = 5054\ mm^2$$

The plastic shear resistance:

$$V_{pl.Rd} = A_V \cdot \frac{f_y/\sqrt{3}}{\gamma_{M0}} = 5054 \cdot \frac{420/\sqrt{3}}{1,0} = 1226\ kN$$

Comparison S420 vs S355:
In case the design calculations would be performed according to grade S355, the shear resistance would be 1036 kN. In both cases the shear resistance can be determined according to plastic shear resistance $V_{pl.Rd}$. Consequently the increase of the yield strength S355 → S420 can be fully utilised for the shear resistance (= + 18 %).

2.8 Resistance of a structural hollow section subject to torsional moment

The structural hollow sections apply excellently to structures that are subject to torsion. Their torsional stiffness is superior when compared to open sections. Moreover, calculation of the torsional resistance is considerably simpler on structural hollow sections than on open sections.

The design condition for torsional resistance can be presented in the form:

$$M_{x.Ed} \leq M_{x.Rd} \qquad (2.89)$$

where $M_{x.Ed}$ is the design value of the torsional moment at ultimate limit state
$M_{x.Rd}$ is the smaller of the following values:
plastic torsional resistance $M_{x.pl.Rd}$ or
torsional buckling resistance $M_{x.b.Rd}$

According to EN 1993-1-1, on structural hollow sections subject to torsional moment, the calculation is allowed to be simplified by assuming that the whole external moment is carried by

St. Venant torsion alone (i.e. the effects of torsional warping may be neglected regarding the total torsional moment and stresses) [3,4,5]. Thereby the design value of torsional moment can be presented in the form:

$$M_{x.Ed} = M_{t.Ed} \tag{2.90}$$

where $M_{t.Ed}$ is the design value of St. Venant torsion

When calculating the torsional resistance, also the shear resistance and shear buckling resistance of the single plane elements shall be taken into account. [3,4,5]. That is why calculation of the torsional resistance is presented in the following by separating it into different procedures in the similar way as the shear resistance in clause 2.7.

2.8.1 Resistance of square and rectangular hollow sections to torsional moment

The procedure to calculate the resistance to torsion depends on the slenderness of the plane elements in the cross-section correspondingly as the shear resistance in clause 2.7.1:

$$\frac{max(b,h)}{t} \leq \frac{72\varepsilon}{\eta} + 3 \quad \Rightarrow \text{calculate plastic torsional resistance (clause 2.8.1.1)} \tag{2.91a}$$

$$\frac{max(b,h)}{t} > \frac{72\varepsilon}{\eta} + 3 \quad \Rightarrow \text{calculate torsional buckling resistance (clause 2.8.1.2)} \tag{2.91b}$$

where
- b is the width of the cross-section
- h is the depth of the cross-section
- t is the wall thickness
- ε $= \sqrt{235/f_y} \quad [f_y] = N/mm^2$
- f_y is the nominal yield strength of the material
- η is the factor taking the strain hardening into account

For the reduction factor η, on structural hollow sections the value η = 1,0 is used (see clause 2.7.1).

The slenderness limit according to expression (2.91) is presented in Table 2.19 based on the above presented value η = 1,0. As with regard to shear buckling, in practice only on a few structural hollow sections torsional buckling turns out to be governing in design.

2.8.1.1 Plastic torsional resistance of square and rectangular hollow sections

The torsional resistance of a structural hollow section according to the theory of plasticity $M_{x.pl.Rd}$ is [32]:

$$M_{x.pl.Rd} = \frac{f_y/\sqrt{3}}{\gamma_{M0}} W_t \approx \frac{f_y/\sqrt{3}}{\gamma_{M0}} \cdot 2A_t t \tag{2.92}$$

where
- W_t is the torsional section modulus of the cross-section (Annex 11.1)
- A_t is the area limited by the centreline of the hollow section wall thickness

t is the wall thickness
f_y is the nominal yield strength of the material
γ_{M0} is the partial safety factor for resistance (Table 2.5)

2.8.1.2 Torsional buckling resistance of square and rectangular hollow sections

The torsional buckling resistance of a structural hollow section $M_{x.b.Rd}$ can be calculated on the basis of torsional buckling strength $\chi_t f_y / \sqrt{3}$ from the expression:

$$M_{x.b.Rd} = \frac{\chi_t f_y / \sqrt{3}}{\gamma_{M1}} W_t \approx \frac{\chi_t f_y / \sqrt{3}}{\gamma_{M1}} \cdot 2A_t t \qquad (2.93)$$

where χ_t is the reduction factor for torsional buckling
W_t is the torsional section modulus of the cross-section (Annex 11.1)
A_t is the area limited by the centreline of the hollow section wall thickness
t is the wall thickness
f_y is the nominal yield strength of the material
γ_{M1} is the partial safety factor for resistance (Table 2.5)

The reduction factor χ_t for torsional buckling is calculated according to clause 2.7.1.2 like χ_w, but in the expression for slenderness (2.80) now the bigger (i.e. the governing) of the cross-sectional dimensions b or h will be applied.

2.8.2 Resistance of circular hollow sections to torsional moment

The procedure to calculate torsional resistance of circular hollow sections is determined as follows:

Class 1, 2 and 3 $\Rightarrow$ calculate plastic torsional resistance (clause 2.8.2.1)
Class 4 $\Rightarrow$ calculate torsional buckling resistance (clause 2.8.2.2)

2.8.2.1 Plastic torsional resistance of circular hollow sections

The torsional resistance of circular hollow sections according to theory of plasticity $M_{x.pl.Rd}$ is [32]:

$$M_{x.pl.Rd} = \frac{f_y / \sqrt{3}}{\gamma_{M0}} W_t \approx \frac{f_y / \sqrt{3}}{\gamma_{M0}} \cdot 2A_t t \qquad (2.94)$$

where W_t is the torsional section modulus of the cross-section (Annex 11.1)
A_t is the area limited by the centreline of the hollow section wall thickness
t is the wall thickness
f_y is the nominal yield strength of the material
γ_{M0} is the partial safety factor for resistance (Table 2.5)

2.8.2.2 Torsional buckling resistance of circular hollow sections

The torsional buckling resistance of circular hollow sections $M_{x.b.Rd}$ is calculated on the basis of torsional buckling strength $\chi_t f_y / \sqrt{3}$ from the expression:

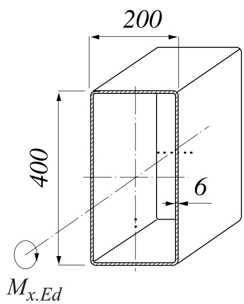
$$M_{x.b.Rd} = \frac{\chi_t f_y / \sqrt{3}}{\gamma_{M1}} W_t \approx \frac{\chi_t f_y / \sqrt{3}}{\gamma_{M1}} \cdot 2A_t t \qquad (2.95)$$

where χ_t is the reduction factor for torsional buckling
W_t is the torsional section modulus of the cross-section (Annex 11.1)
A_t is the area limited by the centreline of the hollow section wall thickness
t is the wall thickness
f_y is the nominal yield strength of the material
γ_{M1} is the partial safety factor for resistance **according to EN 1993-1-6** (Table 2.5)

The reduction factor χ_t for torsional buckling is calculated for circular hollow sections according to clause 2.7.2.2 like χ_τ.

Example 2.16

Study the hollow section $400\times200\times6$ from Example 2.12, when subjected to torsional moment $M_{x.Ed}$ = 170 kNm.



The steel grade is SSAB Domex Tube Double Grade, which fulfills the EN 10219 requirements for both steel grades S420MH and S355J2H. Thereby the design calculations may be performed at designer's own choice either according to grade S420 or grade S355. Grade S420 is chosen in this Example as design basis.

$W_t = 877{,}1 \cdot 10^3\ mm^3$ *(Annex 11.1)*
$f_y = 420\ N/mm^2$
$\gamma_{M1} = 1{,}0$

First, check whether the torsional buckling needs to be considered. For the hollow section chosen in this Example we get:

$$\frac{max(b, h)}{t} = \frac{400}{6} = 66,67 > \frac{72\varepsilon}{\eta} + 3 = \frac{72\sqrt{235/420}}{1,0} + 3 = 56,86$$

$\Rightarrow$ the torsional resistance shall be determined according to the torsional buckling resistance.

The cross-section's most slender plane element is the web, for which the non-dimensional slenderness can be obtained by applying expression (2.80) for torsional buckling:

$$\bar{\lambda}_t = \frac{\bar{b}/t}{86,4\varepsilon} = \frac{(h-3t)/t}{86,4\varepsilon} = \frac{(400-3\cdot6)/6}{86,4\cdot\sqrt{235/420}} = 0,9851$$

$$0,83 \le \bar{\lambda}_t < 1,08$$

The reduction factor for torsional buckling is obtained from Table 2.20 ($\chi_t = \chi_w$):

$$\chi_t = 0,83/\bar{\lambda} = 0,83/0,9851 = 0,8426$$

The torsional buckling resistance:

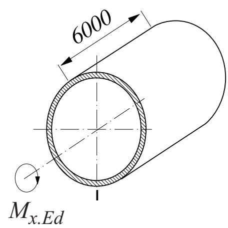
$$M_{x.b.Rd} = \frac{\chi_t f_y / \sqrt{3}}{\gamma_{M1}} W_t = \frac{0,8426 \cdot 420/\sqrt{3}}{1,0} \cdot 877,1 \cdot 10^3 = 179,2 \text{ kNm} \geq M_{x.Ed} \qquad OK$$

Comparison S420 vs S355:

In case the design calculations would be performed according to grade S355, the torsional buckling resistance would be 164,7 kNm, which is not sufficient. In both cases torsional resistance shall be determined according to torsional buckling, and increase of the material strength S355 → S420 further increases the web's non-dimensional slenderness. Nevertheless, increase of the yield strength by 18 % improves the torsional buckling resistance in this Example by 9 %.

Example 2.17

Study the Class 4 circular hollow section 323,9×5 from Example 2.14, when subjected to torsional moment $M_{x.Ed}$ = 110 kNm.



The steel grade is SSAB Domex Tube Double Grade, which fulfills the EN 10219 requirements for both steel grades S420MH and S355J2H. Thereby the design calculations may be performed at designer's own choice either according to grade S420 or grade S355. Grade S420 is chosen in this Example as design basis.

W_t = 786,6 · 10^3 mm^3 (Annex 11.1)
f_y = 420 N/mm^2
γ_{M1} = 1,1 (Class 4 circular hollow section, EN 1993-1-6)

On circular hollow section, the torsional resistance shall be determined according to torsional buckling resistance if the cross-section is classified into Class 4.

On circular hollow section, the reduction factor χ_t for torsional buckling is the same as the reduction factor χ_τ for shear buckling, which is calculated already earlier in Example 2.14:

$$\chi_t = \chi_\tau = 0,6941$$

The torsional buckling resistance:

$$M_{x.b.Rd} = \frac{\chi_t f_y / \sqrt{3}}{\gamma_{M1}} W_t = \frac{0,6941 \cdot 420/\sqrt{3}}{1,1} \cdot 786,6 \cdot 10^3 = 120,4 \text{ kNm} \geq M_{x.Ed} \qquad OK$$

Comparison S420 vs S355:
In case the design calculations would be performed according to grade S355, the torsional buckling resistance would be 108,6 kNm, which is not sufficient. In both cases torsional resistance shall be determined according to torsional buckling, and increase of the material strength S355 → S420 further increases the cross-section's non-dimensional slenderness. Nevertheless, increase of the yield strength by 18 % improves the torsional buckling resistance in this Example by 11 %.

2.9 Cross-sectional resistance of a structural hollow section subject to combined actions, when buckling and lateral-torsional buckling are prevented

Resistance to concentrated load and its interaction with other forces and moments is presented in clause 2.11.

2.9.1 Square and rectangular and Class 1, 2 and 3 circular hollow sections

2.9.1.1 Limitation of stresses (von Mises yield criterion)

Verification of resistance according to theory of elasticity is always allowed in all cross-section Classes, provided that in case of Class 4, the effective cross-section properties are used [3,4,5].

<u>If there is no other applicable formula given to verify the combined effects</u>, the elastic verification of the resistance can be performed at the critical point of the cross-section by using the following yield criterion based on von Mises equivalent stress [3,4,5]:

$$\left[\frac{\sigma_{x.Ed}}{f_y/\gamma_{M0}}\right]^2 + \left[\frac{\sigma_{z.Ed}}{f_y/\gamma_{M0}}\right]^2 - \left[\frac{\sigma_{x.Ed}}{f_y/\gamma_{M0}}\right]\left[\frac{\sigma_{z.Ed}}{f_y/\gamma_{M0}}\right] + 3\left[\frac{\tau_{Ed}}{f_y/\gamma_{M0}}\right]^2 \leq 1,0 \qquad (2.96)$$

where $\sigma_{x.Ed}$ is the design value of the longitudinal stress at the point of consideration (compression is positive)

$\sigma_{z.Ed}$ is the design value of the transverse stress at the point of consideration (compression is positive)

τ_{Ed} is the design value of the shear stress at the point of consideration

f_y is the nominal yield strength of the material

γ_{M0} is the partial safety factor for resistance (Table 2.5)

Verification according to expression (2.96) can be conservative, because partial plastification is therein not taken into account. The latter is permitted also in elastic design in certain situations. Therefore this method is recommended only when the combined effects cannot be verified by the single resistances N_{Rd}, M_{Rd} and V_{Rd} [3,4,5].

2.9.1.2 Shear force and torsional moment

The interaction of torsion and shear force is taken into account in different Parts of Eurocode in slightly different ways. The provisions of Part EN 1993-1-1 shall be applied when the shear resistance of the cross-section is equal to the plastic shear resistance $V_{pl.Rd}$, and the provisions of Part EN 1993-1-5 shall be applied when shear buckling reduces shear resistance, i.e. $V_{b.Rd} < V_{pl.Rd}$.

2.9.1.2.1 Shear force and torsional moment, when shear buckling does not reduce shear resistance

According to EN 1993-1-1, when shear force and torsional moment act simultaneously, the plastic shear resistance $V_{pl.Rd}$ is reduced to the value $V_{pl.T.Rd}$ and the shear force shall satisfy the following condition [3,4,5]:

$$V_{Ed} \leq V_{pl.T.Rd} \tag{2.97}$$

The plastic shear resistance $V_{pl.T.Rd}$ reduced by the torsional moment can be calculated for structural hollow sections from the formula [3,4,5]:

$$V_{pl.T.Rd} = \sqrt{1 - \frac{\tau_{t.Ed}}{(f_y/\sqrt{3})/\gamma_{M0}}} \cdot V_{pl.Rd} \tag{2.98}$$

$$\tau_{t.Ed} = \frac{M_{t.Ed}}{W_t} \approx \frac{M_{t.Ed}}{2A_t t} \tag{2.99}$$

$$M_{t.Ed} = M_{x.Ed} \tag{2.100}$$

where
- $V_{pl.Rd}$ is the plastic shear resistance according to clause 2.7
- $\tau_{t.Ed}$ is the portion of shear stresses due to St. Venant torsion
- $M_{x.Ed}$ is the design value of acting torsional moment
- $M_{t.Ed}$ is the portion of St. Venant torsion from the acting torsional moment (see clause 2.8)
- W_t is the torsional section modulus of the cross-section (Annex 11.1)
- A_t is the area limited by the centreline of the hollow section wall thickness
- t is the wall thickness
- f_y is the nominal yield strength of the material
- γ_{M0} is the partial safety factor for resistance (Table 2.5)

2.9.1.2.2 Shear force and torsional moment, when shear buckling reduces shear resistance

These assessments according to Part EN 1993-1-5 of Eurocode are needed only on some few square and rectangular hollow sections, because on circular hollow sections the shear resistance is always determined in cross-section Classes 1, 2 and 3 on the basis of plastic shear resistance (see clause 2.7.2), in which case the interaction of shear force and torsional moment is checked according to preceding clause 2.9.1.2.1.

In EN 1993-1-5, the effects of torsion are included in shear buckling verifications so, that the shear caused by possible torsion shall be already taken into account in determining the acting shear force V_{Ed} [13,14]. On square and rectangular hollow sections the effect of torsion to shear force can be calculated as follows:

$$\Delta V = \tau_{t.Ed} A_V \qquad (2.101)$$

where $\tau_{t.Ed}$ is the shear stress caused by torsion according to expression (2.99)
A_V is the shear area in the direction of the shear force V_{Ed} according to expression (2.77)

2.9.1.3 Bending moment and shear force

Provisions for the interaction of simultaneous bending and shear are presented in Parts EN 1993-1-1 and EN 1993-1-5 of Eurocode. The provisions of Part EN 1993-1-1 shall be applied when the shear resistance of the cross-section is equal to the plastic shear resistance $V_{pl.Rd}$, and the provisions of Part EN 1993-1-5 shall be applied when shear buckling reduces shear resistance, i.e. $V_{b.Rd} < V_{pl.Rd}$.

2.9.1.3.1 Bending moment and shear force, when shear buckling does not reduce shear resistance

According to Part EN 1993-1-1 of Eurocode, the effect of shear force to bending resistance shall be taken into account if the shear force exceeds half of the plastic shear resistance, i.e. $V_{Ed} > 0{,}5 V_{pl.Rd}$. In this case bending resistance of the related cross-section Class is reduced by applying a reduced yield strength $(1-\rho)f_y$ for the shear area in the direction to be considered. Factor ρ is calculated as [3,4,5]:

$$\rho = \left[\frac{2V_{Ed}}{V_{pl.Rd}} - 1\right]^2 \qquad (2.102)$$

where V_{Ed} is the design value of the shear force in the direction to be considered
$V_{pl.Rd}$ is the plastic shear resistance in the direction to be considered according to clause 2.7.1.1 (square and rectangular hollow sections) or according to clause 2.7.2.1 (circular hollow sections)

Instead of reducing the yield strength, it is alternatively possible to reduce the thickness of the corresponding cross-section element by using thickness $(1-\rho)\,t$ for that element [3,4,5].

Square and rectangular hollow sections (all Classes):

The design value of bending resistance reduced due to shear force can be alternatively calculated in all cross-section Classes as a reduced value of the plastic bending resistance by applying the following formula, but after that it shall be checked that the bending resistance $M_{c.Rd}$ of the cross-section corresponding to its real cross-section Class calculated according to clause 2.6.1, is not exceeded [3,4,5]:

$$M_{V.y.Rd} = \left[W_{pl.y} - \frac{\rho A_V^2}{8t}\right]\left(\frac{f_y}{\gamma_{M0}}\right) \qquad \textit{but } M_{V.y.Rd} \le M_{c.y.Rd} \tag{2.103}$$

where
- W_{pl} is the plastic section modulus of the cross-section (Annex 11.1)
- $M_{c.y.Rd}$ is the bending resistance of the cross-section according to its real cross-section Class according to clause 2.6.1
- ρ is calculated according to expression (2.102)
- A_V is the shear area corresponding to shear force $V_{z.Ed}$ in z-direction according to expression (2.77)
- t is the wall thickness
- f_y is the nominal yield strength of the material
- γ_{M0} is the partial safety factor for resistance (Table 2.5)

By changing the applied axes in expression (2.103), the bending resistance about z-axis reduced by the shear force in y-direction, can be calculated respectively.

For biaxial bending with simultaneous shear forces, EN 1993-1-1 does not assign any specific interaction formula (nor does the preceding prestandard ENV 1993-1-1), only general level instructions are presented. Therefore, in this respect, the Eurocode's provisions have been interpreted in different sources in slightly different ways. Based on the different interpretations the following interaction formulae can be presented [33,34]:

$$\left[\frac{M_{y.Ed}}{M_{V.y.Rd}}\right]^{\alpha} + \left[\frac{M_{z.Ed}}{M_{V.z.Rd}}\right]^{\beta} \le 1,0 \qquad \textit{Class 1 and 2} \tag{2.104}$$

$$\left[\frac{M_{y.Ed}}{M_{V.y.Rd}}\right] + \left[\frac{M_{z.Ed}}{M_{V.z.Rd}}\right] \le 1,0 \qquad \textit{Class 3 and 4} \tag{2.105}$$

where the values for factors α and β are taken as $\alpha = \beta = 1{,}66$ according to clause 2.6.4.1.

All the preceding verifications for (M+V) interaction can be alternatively substituted on Classes 3 and 4 by a stress verification basing on von Mises yield criterion according to clause 2.9.1.1 (whereas no calculatory reductions due to shear forces shall be done to the thicknesses or strengths, and the stresses - and in case of Class 4 one effective cross-section - are calculated having all forces and moments acting simultaneously).

Circular hollow sections (Class 1, 2 and 3):

The design value of bending resistance reduced due to shear force can be calculated as a conservative simplification by using reduced yield strength $(1-\rho)f_y$ for the whole cross-section:

$$M_{V.Rd} = (1-\rho)\, M_{c.Rd} \tag{2.106}$$

where ρ is calculated according to expression (2.102) using for the shear force the value $V_{Ed} = \sqrt{V_{y.Ed}^2 + V_{z.Ed}^2}$

$M_{c.y.Rd}$ is the bending resistance of the cross-section according to its real cross-section Class according to clause 2.6.1

For biaxial bending with simultaneous shear forces, the following interaction formula is obtained by applying expression (2.106) and clause 2.6.4:

$$\left[\frac{M_{y.Ed}}{M_{V.y.Rd}}\right]^{\alpha} + \left[\frac{M_{z.Ed}}{M_{V.z.Rd}}\right]^{\beta} \le 1,0 \tag{2.107}$$

where the values for factors α and β are taken as $\alpha = \beta = 2$. Because on circular hollow sections $M_{V.y.Rd} = M_{V.z.Rd}$, the expression (2.107) simplifies to the form:

$$M_{Ed} \le M_{V.Rd} \tag{2.108}$$

where $M_{Ed} = \sqrt{M_{y.Ed}^2 + M_{z.Ed}^2}$

2.9.1.3.2 Bending moment and shear force, when shear buckling reduces shear resistance

These assessments according to Part EN 1993-1-5 of Eurocode are needed only on some few square and rectangular hollow sections, because on circular hollow sections the shear resistance is always determined in cross-section Classes 1, 2 and 3 on the basis of plastic shear resistance (see clause 2.7.2).

If the cross-section is so designed that the flanges are alone able to carry the acting bending moment (i.e. $M_{Ed} \le M_{f.Rd}$), the interaction of bending and shear need not be taken into account (calculation of bending resistance of the flanges on square and rectangular hollow sections is presented later on).

Otherwise the interaction of bending and shear buckling shall be taken into account if the shear force exceeds half of the contribution from the web to shear buckling resistance according to clause 2.7.1.2, i.e. if $V_{Ed} > 0{,}5\,V_{bw.Rd}$ [3,4,5,13,14].

In this case, on square and rectangular hollow sections, the interaction formula for bending and shear is [13,14]:

$$\bar{\eta}_1 + \left[1 - \frac{M_{f.Rd}}{M_{pl.Rd}}\right](2\bar{\eta}_3 - 1)^2 \leq 1,0 \qquad when\ \bar{\eta}_1 \geq \frac{M_{f.Rd}}{M_{pl.Rd}} \tag{2.109}$$

$$\bar{\eta}_1 = \frac{M_{Ed}}{M_{pl.Rd}} \tag{2.110}$$

$$\bar{\eta}_3 = \frac{V_{Ed}}{V_{bw.Rd}} \tag{2.111}$$

where M_{Ed} is the design value of the bending moment at ultimate limit state

$M_{f.Rd}$ is the design plastic bending resistance of the cross-section, when only the effective flanges are taken into account

$M_{pl.Rd}$ is the design plastic bending resistance of the cross-section, when the cross-section consists according to its cross-section Class of effective flanges, and independently of its cross-section Class of a fully effective web

V_{Ed} is the design value of the shear force at ultimate limit state, including the shear caused by possible torsion

$V_{bw.Rd}$ is the contribution from the web to shear buckling resistance of the cross-section (clause 2.7.1.2)

The interaction formula (2.109) is valid for all cross-section Classes. However, it should be noted that the definition of the plastic bending resistance applied therein according to EN 1993-1-5, differs from the definition given in EN 1993-1-1 (cf. expression (2.44)).

The interaction formula (2.109) need not be checked in a such cross-section that is at a distance of not more than $h_w/2$ from a support having vertical stiffeners of web [13,14].

The bending resistance of the effective flanges $M_{f.Rd}$ alone, can be calculated for square and rectangular hollow sections as follows [13,14,28]:

$$M_{f.Rd} = \frac{A_{f.eff} f_y}{\gamma_{M0}} \cdot (h - t) \tag{2.112}$$

$$A_{f.eff} = A_f - A_{f.non.eff} \tag{2.113}$$

$$A_f = [A - 2(h - 3t)t]/2 \tag{2.114}$$

$$A_{f.non.eff} = (1 - \rho)(b - 3t)t \tag{2.115}$$

where h is the depth of the cross-section in respect to the bending axis

b is the width of the cross-section

A is the gross cross-section area (Annex 11.1)

A_f is the gross area of the flange

$A_{f.eff}$ is the effective area of the compressed flange

$A_{f.non.eff}$ is the non-effective area of the compressed flange

ρ is the reduction factor for the effective width according to clause 2.4.1

f_y is the nominal yield strength of the material
γ_{M0} is the partial safety factor for resistance (Table 2.5)

Expression (2.109) describes the fact, that shear load which is carried mainly by the web, does not influence the capacity of the flanges to carry the bending moment. High shear load reduces thus only the portion of the webs usable for bending resistance (the first parenthetical expression). The shear stress of the flange, caused by possible torsion or shear force parallel to the flange, has more influence. In such case, on expression (2.111), the shear buckling resistance of the web is substituted by the appropriate shear resistance of the flange (i.e. plastic shear resistance or shear buckling resistance), and the shear force V_{Ed} is substituted by the shear force acting in the flange (however at least half of the shear resistance of the flange). At the same time $M_{f.Rd}$ = 0 shall be applied in expression (2.109), and factor $\bar{\eta}_1$ from expression (2.110) shall be substituted by factor η_1 calculated from the expression [13,14]:

$$\eta_1 = \frac{N_{Ed}}{A_{eff} f_y / \gamma_{M0}} + \frac{M_{y.Ed} + N_{Ed} e_{Ny}}{W_{eff.y} f_y / \gamma_{M0}} + \frac{M_{z.Ed} + N_{Ed} e_{Nz}}{W_{eff.z} f_y / \gamma_{M0}} \leq 1,0 \qquad (2.116)$$

where A_{eff} is the effective cross-section area when only uniform compression is acting on the cross-section
W_{eff} is the section modulus of the effective cross-section when only bending moment is acting about the considered axis
e_N is the shift of the neutral axis to the considered direction, when the cross-section is subject to only uniform compression.
Thereby, due to double symmetry, on structural hollow sections e_{Ny} = 0 and e_{Nz} = 0

From the interaction formula (2.109) it is possible to derive the following expression for the reduced bending resistance of the cross-section about y-axis, when the shear force acts in the z-direction only:

$$M_{V.y.Rd} = M_{f.Rd} + (M_{pl.Rd} - M_{f.Rd})(1 - \rho_{bw}) \qquad (2.117)$$

$$\rho_{bw} = \left[\frac{2V_{z.Ed}}{V_{bw.Rd}} - 1\right]^2 \qquad (2.118)$$

The expression (2.117) is valid under the same conditions as the expression (2.109).

The bending resistance of expression (2.117) consists of the sum of the bending resistance of the flanges and the reduced portion of the web. When comparing expressions (2.117) - (2.118) to the preceding provisions presented in clause 2.9.1.3.1, it can be seen that if the slenderness of the web of the cross-section is small enough that $V_{bw.Rd} \rightarrow V_{pl.Rd}$, expression (2.117) resembles the provisions presented in Part EN 1993-1-1 of Eurocode, in which the bending resistance related to the cross-section Class is reduced by applying a reduced yield strength $(1-\rho) f_y$ for the shear area. Despite of the similarity of provisions in Parts EN 1993-1-1 and EN 1993-1-5, the results differ from each other mainly because the provisions of EN 1993-1-5 is applied for all cross-section Classes, while in EN 1993-1-1 a reduction is applied for the bending resistance according to the cross-section Class. Also the definitions of the terms used in the formulae differ slightly from each other.

2.9.1.4 Bending moment, shear force and torsional moment

According to Part EN 1993-1-1 of Eurocode, the effect of torsion to bending resistance shall be taken into account if the shear force exceeds half of the plastic shear resistance reduced due to torsion, i.e. $V_{Ed} > 0{,}5 V_{pl.T.Rd}$. In this case the reduced bending resistance is calculated according to clause 2.9.1.3.1, but ρ is calculated from the following formula [3,4,5]:

$$\rho = \left[\frac{2V_{Ed}}{V_{pl.T.Rd}} - 1\right]^2 \qquad (2.119)$$

where $V_{pl.T.Rd}$ is calculated according to clause 2.9.1.2.1.

The provisions given in EN 1993-1-1 for interaction of torsion, are assigned according to clause 2.9.1.3.1 for a case where shear buckling does not reduce the plastic shear resistance. Otherwise, if shear buckling does reduce shear resistance, the interaction verifications shall be carried out according to clause 2.9.1.3.2. In this case the effect of possible torsion shall be taken into account in the acting shear force V_{Ed} according to clause 2.9.1.2.2 [13,14].

2.9.1.5 Bending moment and normal force

Verification of combined bending moment and normal force is based on verification of the cross-section resistance, when buckling and lateral-torsional buckling of the member are not taken into account. Determination of the cross-section Class depends on the cross-section's stress distribution. In order to simplify, the cross-section Class may be determined on the basis of compression only, which leads to a conservative result.

2.9.1.5.1 Class 1 and 2

In a bended cross-section, a simultaneous normal force causes a change of the stress distribution and thereby a shift of the neutral axis location (Figure 2.7). When designing according to the theory of plasticity (Class 1 and 2), the shift of the neutral axis affects on square and rectangular hollow sections the limits of the cross-section Classes through factor α according to Table 2.7 (on circular hollow sections, the location of the neutral axis has no effect on the cross-section Class, see Table 2.8). The α-factor can be calculated on square and rectangular hollow sections from the formula:

$$\alpha = 0,5 \cdot \left[1 + \frac{N_{Ed}}{2(h-3t)t \cdot f_y/\gamma_{M0}}\right] \qquad (2.120)$$

$$|N_{Ed}| \le 2(h-3t)t \cdot f_y/\gamma_{M0} \qquad (2.121)$$

where N_{Ed} is the design value of the normal force at ultimate limit state (compression is positive)
h is the depth of the cross-section in respect to the bending axis
t is the wall thickness
f_y is the nominal yield strength of the material
γ_{M0} is the partial safety factor for resistance (Table 2.5)

Expression (2.120) can be used when the acting normal force N_{Ed} is at its maximum equal to expression (2.121), indicating the plastic neutral axis is located within the region of the calculatory web ($0 \le \alpha \le 1$).

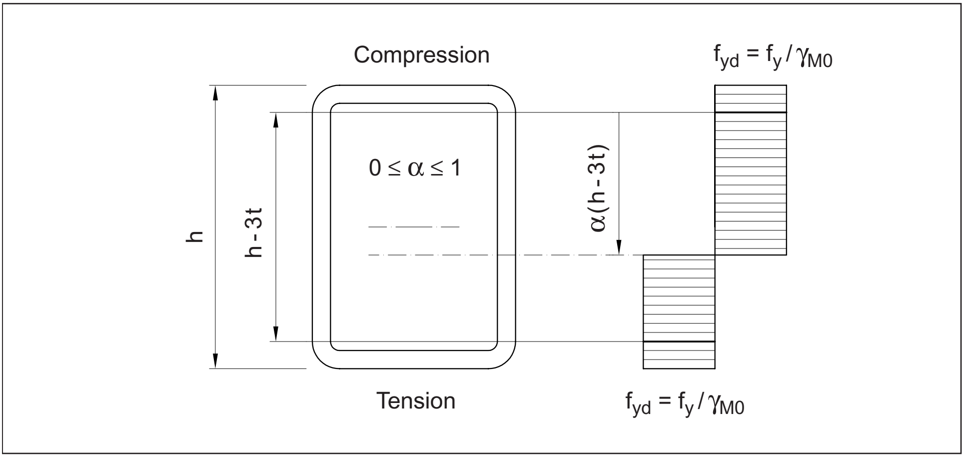


Figure 2.7 Location of the plastic neutral axis defined with factor α for a cross-section subject to simultaneous compression and bending (Class 1 and 2)

The bending resistance of the cross-section decreases due to the influence of the normal force. The design condition is thereby written as [3,4,5]:

$$M_{Ed} \le M_{N.Rd} \tag{2.122}$$

where M_{Ed} is the design value of the bending moment at ultimate limit state
$M_{N.Rd}$ is the reduced bending resistance of the cross-section due to normal force

The design value of reduced bending resistance according to the theory of plasticity depends on the form of the cross-section. The resistances of different cross-sections can be calculated from the following formulae [3,4,5]:

Square and rectangular hollow sections, when the fastener holes need not be taken into account:

- **bending about y-axis:**

$$M_{N.y.Rd} = M_{pl.y.Rd} \quad when \quad N_{Ed} \le \frac{0,5(A-2bt)f_y}{\gamma_{M0}} \quad and \quad N_{Ed} \le 0,25 N_{pl.Rd} \tag{2.123a}$$

in other cases:

$$M_{N.y.Rd} = M_{pl.y.Rd} \frac{1-n}{1-0,5a_w} \qquad but \quad M_{N.y.Rd} \le M_{pl.y.Rd} \tag{2.123b}$$

- **bending about z-axis:**

$$M_{N.z.Rd} = M_{pl.z.Rd} \quad when \quad N_{Ed} \leq \frac{0,5(A-2ht)f_y}{\gamma_{M0}} \quad and \quad N_{Ed} \leq 0,25N_{pl.Rd} \qquad (2.124a)$$

in other cases:

$$M_{N.z.Rd} = M_{pl.z.Rd}\frac{1-n}{1-0,5a_f} \quad but \quad M_{N.z.Rd} \leq M_{pl.z.Rd} \qquad (2.124b)$$

where $n = N_{Ed}/N_{pl.Rd}$

$a_w = (A - 2bt)/A$ but $a_w \leq 0,5$

$a_f = (A - 2ht)/A$ but $a_f \leq 0,5$

In biaxial bending the interaction formula is [3,4,5]:

$$\left[\frac{M_{y.Ed}}{M_{N.y.Rd}}\right]^{\alpha} + \left[\frac{M_{z.Ed}}{M_{N.z.Rd}}\right]^{\beta} \leq 1,0 \qquad (2.125)$$

where the values for factors α and β can be taken on square and rectangular hollow sections conservatively as $\alpha = \beta = 1$, or the values can be calculated as follows [3,4,5]:

$$\alpha = \beta = \frac{1,66}{1-1,13n^2} \quad but \quad \alpha = \beta \leq 6 \qquad (2.126)$$

As a conservative simplification the following interaction formula basing on the linear summation of utilisation ratios can be used [3,4,5]:

$$\frac{N_{Ed}}{Af_y/\gamma_{M0}} + \frac{M_{y.Ed}}{W_{pl.y}f_y/\gamma_{M0}} + \frac{M_{z.Ed}}{W_{pl.z}f_y/\gamma_{M0}} \leq 1,0 \qquad (2.127)$$

Circular hollow sections, when the fastener holes need not be taken into account:

$$M_{N.Rd} = M_{pl.Rd}(1-n^{1,7}) \quad but \quad M_{N.Rd} \leq M_{pl.Rd} \qquad (2.128)$$

where $n = N_{Ed}/N_{pl.Rd}$

In biaxial bending the interaction formula is [3,4,5]:

$$\left[\frac{M_{y.Ed}}{M_{N.y.Rd}}\right]^{\alpha} + \left[\frac{M_{z.Ed}}{M_{N.z.Rd}}\right]^{\beta} \leq 1,0 \qquad (2.129)$$

where the values for factors α and β are taken on circular hollow sections as $\alpha = \beta = 2$. Because on circular hollow sections $M_{N.y.Rd} = M_{N.z.Rd}$, expression (2.129) simplifies to the form:

$$M_{Ed} \leq M_{N.Rd} \qquad (2.130)$$

where $M_{Ed} = \sqrt{M_{y.Ed}^2 + M_{z.Ed}^2}$

2.9.1.5.2 Class 3

When designing according to the theory of elasticity, in a bended cross-section the shift of the neutral axis due to a simultaneous normal force changes the stress ratio ψ of the edge stresses of the cross-section. Thereafter the limits of the cross-section Classes change according to Table 2.7.

When there is no shear force present, the axial stress $\sigma_{x.Ed}$ is not allowed to exceed the design value of yield strength at any point of the cross-section [3,4,5]:

$$\sigma_{x.Ed} \le \frac{f_y}{\gamma_{M0}} \tag{2.131}$$

The fastener holes are taken into account when needed, see clauses 2.5.1 and 2.6.2.

Expression (2.131) can be presented also as a sum of utilisation ratios as follows:

Square and rectangular hollow sections:

$$\frac{N_{Ed}}{A f_y / \gamma_{M0}} + \frac{M_{y.Ed}}{W_{el.y} f_y / \gamma_{M0}} + \frac{M_{z.Ed}}{W_{el.z} f_y / \gamma_{M0}} \le 1,0 \tag{2.132}$$

Circular hollow sections:

$$\frac{N_{Ed}}{A f_y / \gamma_{M0}} + \frac{M_{Ed}}{W_{el} f_y / \gamma_{M0}} \le 1,0 \tag{2.133}$$

where $M_{Ed} = \sqrt{M_{y.Ed}^2 + M_{z.Ed}^2}$

2.9.1.5.3 Class 4

On square and rectangular structural hollow sections, when there is no shear force present, the axial stress $\sigma_{x.Ed}$ is not allowed to exceed the design value of yield strength at any point of the cross-section [3,4,5]:

$$\sigma_{x.Ed} \le \frac{f_y}{\gamma_{M0}} \tag{2.134}$$

The axial stress $\sigma_{x.Ed}$ is determined in Class 4 on the basis of the effective cross-section. The fastener holes shall be taken into account when necessary, see clauses 2.5.1 and 2.6.2.

As an alternative to condition (2.134), the following interaction formula can be applied [3,4,5,13,14]:

$$\eta_1 = \frac{N_{Ed}}{A_{eff} f_y / \gamma_{M0}} + \frac{M_{y.Ed} + N_{Ed} e_{Ny}}{W_{eff.y} f_y / \gamma_{M0}} + \frac{M_{z.Ed} + N_{Ed} e_{Nz}}{W_{eff.z} f_y / \gamma_{M0}} \le 1,0 \tag{2.135}$$

where A_{eff} is the effective cross-section area when only uniform compression is acting on the cross-section

W_{eff} is the section modulus of the effective cross-section when only bending moment is acting about the considered axis

e_N is the shift of the neutral axis to the considered direction, when the cross-section is subject to only uniform compression.
Thereby, due to double symmetry, on structural hollow sections
$e_{Ny} = 0$ and $e_{Nz} = 0$

The signs of the terms N_{Ed}, $M_{y.Ed}$ and $M_{z.Ed}$ depend on the combination of the produced axial stresses.

2.9.1.6 Bending moment, normal force and shear force

As in case of combined bending moment and shear force (clause 2.9.1.3), also in case of (M+N+V) combination the verification is separated in Eurocode into Parts EN 1993-1-1 and EN 1993-1-5 depending on whether shear buckling reduces the plastic shear resistance of the cross-section. The provisions of Part EN 1993-1-1 shall be applied when the shear resistance of the cross-section is equal to the plastic shear resistance $V_{pl.Rd}$, and the provisions of Part EN 1993-1-5 shall be applied when shear buckling reduces shear resistance, i.e. $V_{b.Rd}$ < $V_{pl.Rd}$.

2.9.1.6.1 Bending moment, normal force and shear force, when shear buckling does not reduce shear resistance

According to Part EN 1993-1-1 of Eurocode the effect of shear force to bending resistance need not be taken into account, if the shear force does not exceed half of the plastic shear resistance, i.e. $V_{Ed} \leq 0{,}5 V_{pl.Rd}$. In such case only the combined effect of bending moment and normal force needs to be checked according to clause 2.9.1.5.

In other case the resistances according to combined bending moment and normal force (clause 2.9.1.5) shall be further reduced by applying a reduced yield strength $(1-\rho)f_y$ for the shear area in the direction to be considered (i.e. for the plane elements parallel to the shear force). Factor ρ is calculated according to clause 2.9.1.3.1 [3,4,5].

Instead of reducing the yield strength, it is alternatively possible to reduce the thickness of the corresponding cross-section element by using thickness $(1-\rho)\,t$ for that element [3,4,5].

As a conservative simplification it is possible to use the reduced yield strength $(1-\rho)f_y$ or alternatively the reduced thickness $(1-\rho)\,t$ for the whole cross-section.

2.9.1.6.1.1 Class 1 and 2

The shift of the neutral axis takes place as in the case of simultaneous bending and normal force (see clause 2.9.1.5.1).

The design condition for simultaneous bending moment, normal force and shear force is [3,4,5]:

$$M_{Ed} \leq M_{N.V.Rd} \qquad (2.136)$$

where M_{Ed} is the design value of the bending moment at ultimate limit state
$M_{N.V.Rd}$ is the reduced bending resistance of the cross-section due to normal force and shear force

The design value of reduced bending resistance according to the theory of plasticity depends on the form of the cross-section.

The resistances of different cross-sections can be calculated from the following formulae. The formulae are of their basic structure largely the same as the (M+N) interaction formulae in clause 2.9.1.5.1, but now the reducing effect of the shear force is additionally taken into account in the resistances (areas) of the shear-force-bearing cross-section elements, and that way in the whole cross-section [3,4,5,34]. It should be noted in this context, that EN 1993-1-1 differs herein slightly from its own practice applied in clause 2.7.1.1 on calculation of the shear area.

Square and rectangular hollow sections,
when the fastener holes need not be taken into account:

- **bending about y-axis (N_{Ed} and $V_{z.Ed}$ and $M_{y.Ed}$):**

$$M_{N.V.y.Rd} = M_{V.y.Rd} \qquad when \quad N_{Ed} \le \frac{0,5 A_{w.red} f_y}{\gamma_{M0}} \quad and \quad N_{Ed} \le 0,25 N_{V.Rd} \qquad (2.137a)$$

in other cases:

$$M_{N.V.y.Rd} = M_{V.y.Rd} \frac{1 - n_V}{1 - 0,5 a_V} \qquad but \quad M_{N.V.y.Rd} \le M_{V.y.Rd} \qquad (2.137b)$$

where $n_V = N_{Ed} / N_{V.Rd}$
$a_V = A_{w.red} / A_{tot.red}$ but $a_V \le 0,5$
$A_{w.red} = (1-\rho_z)(A - 2bt)$
$A_{tot.red} = A - \rho_z (A - 2bt)$
$N_{V.Rd} = A_{tot.red} f_y / \gamma_{M0}$
ρ_z and $M_{V.y.Rd}$ are calculated according to clause 2.9.1.3.1

- **bending about z-axis (N_{Ed} and $V_{y.Ed}$ and $M_{z.Ed}$):**

$$M_{N.V.z.Rd} = M_{V.z.Rd} \qquad when \quad N_{Ed} \le \frac{0,5 A_{f.red} f_y}{\gamma_{M0}} \quad and \quad N_{Ed} \le 0,25 N_{V.Rd} \qquad (2.138a)$$

in other cases:

$$M_{N.V.z.Rd} = M_{V.z.Rd} \frac{1 - n_V}{1 - 0,5 a_V} \qquad but \quad M_{N.V.z.Rd} \le M_{V.z.Rd} \qquad (2.138b)$$

where $n_V = N_{Ed} / N_{V.Rd}$
$a_V = A_{f.red} / A_{tot.red}$ *but* $a_V \le 0,5$
$A_{f.red} = (1-\rho_y)(A - 2ht)$
$A_{tot.red} = A - \rho_y (A - 2ht)$
$N_{V.Rd} = A_{tot.red} f_y / \gamma_{M0}$
ρ_y and $M_{V.z.Rd}$ are calculated according to clause 2.9.1.3.1

For biaxial bending with simultaneous shear forces, EN 1993-1-1 does not assign any specific interaction formula (nor does the preceding prestandard ENV 1993-1-1), only non-specific general guidelines are presented. Therefore, in this respect, the Eurocode's provisions have been interpreted in different sources in slightly different ways. Based on the different interpretations the following interaction formula can be presented [33,34]:

$$\left[\frac{M_{y.Ed}}{M_{N.V.y.Rd}}\right]^{\alpha} + \left[\frac{M_{z.Ed}}{M_{N.V.z.Rd}}\right]^{\beta} \leq 1,0 \qquad (2.139)$$

where factors α and β are determined according to clause 2.9.1.5.1, however with the difference that when calculating factors α and β for square and rectangular hollow sections from expression (2.126), factor n_V shall be used instead of factor n. Factor n_V is calculated separately for factors α and β according to the definitions presented with expressions (2.137) and (2.138).

Circular hollow sections, when the fastener holes need not be taken into account:

$$M_{N.V.Rd} = M_{V.Rd}(1 - n^{1,7}) \qquad but \quad M_{N.V.Rd} \leq M_{V.Rd} \qquad (2.140)$$

where $n = N_{Ed} / N_{pl.Rd}$
$M_{V.Rd}$ is calculated according to clause 2.9.1.3.1

For biaxial bending the interaction formula is:

$$\left[\frac{M_{y.Ed}}{M_{N.V.y.Rd}}\right]^{\alpha} + \left[\frac{M_{z.Ed}}{M_{N.V.z.Rd}}\right]^{\beta} \leq 1,0 \qquad (2.141)$$

where for the factors α and β the same values are applied on circular hollow sections as in clause 2.9.1.5.1, i.e. $\alpha = \beta = 2$. Because on circular hollow sections $M_{N.y.Rd} = M_{N.z.Rd}$, expression (2.141) simplifies to the form:

$$M_{Ed} \leq M_{N.V.Rd} \qquad (2.142)$$

where $M_{Ed} = \sqrt{M_{y.Ed}^2 + M_{z.Ed}^2}$

2.9.1.6.1.2 Class 3 and 4

According to Part EN 1993-1-1 of Eurocode, in Class 3 and 4 the combined effect of shear force shall be taken into account by applying a reduced strength $(1-\rho)f_y$ for the shear area, when resistances and cross-sectional properties are calculated. The reduced strength can alternatively be taken into account by applying a reduced thickness $(1-\rho)t$ for the shear area, when using the effective cross-section and calculating the cross-sectional properties. The factor ρ is calculated according to clause 2.9.1.3.1.

This way the calculations may become however rather complex, especially if the shear forces act simultaneously both in y- and z-directions. In practice it is sensible to make a conservative simplification by using the reduced strength $(1-\rho)f_y$ for the whole cross-section. The interaction formula can thereafter be presented in the following forms:

- **Class 3 (all forms):**

$$\frac{N_{Ed}}{Af_y/\gamma_{M0}} + \frac{M_{y.Ed}}{W_{el.y}f_y/\gamma_{M0}} + \frac{M_{z.Ed}}{W_{el.z}f_y/\gamma_{M0}} \leq (1-\rho) \tag{2.143}$$

- **Class 4 (square and rectangular hollow sections):**

$$\frac{N_{Ed}}{A_{eff}f_y/\gamma_{M0}} + \frac{M_{y.Ed}+N_{Ed}\,e_{Ny}}{W_{eff.y}f_y/\gamma_{M0}} + \frac{M_{z.Ed}+N_{Ed}\,e_{Nz}}{W_{eff.z}f_y/\gamma_{M0}} \leq (1-\rho) \tag{2.144}$$

where the left side of the equations is calculated (without the reductions due to shear force) respectively like (M+N) combination in clause 2.9.1.5. If the shear force acts simultaneously both in y- and z-directions, term (1-ρ) on the right side of the equation is substituted by the governing (i.e. smallest) of the terms (1-ρ_y) or (1-ρ_z). Due to double symmetry, on structural hollow sections the shift of the neutral axis in expression (2.144) is $e_{Ny} = 0$ and $e_{Nz} = 0$. Factor ρ is calculated from expression (2.102).

The preceding verifications for (M+N+V) interaction can be alternatively substituted on Classes 3 and 4 by a stress verification basing on von Mises yield criterion according to clause 2.9.1.1 (whereas no calculatory reductions due to shear forces shall be done to the thicknesses or strengths, and the stresses - and in case of Class 4 one effective cross-section - are calculated having all forces and moments acting simultaneously).

2.9.1.6.2 Bending moment, normal force and shear force, when shear buckling reduces shear resistance

These assessments according to Part EN 1993-1-5 of Eurocode are needed only on some few square and rectangular hollow sections, because on circular hollow sections the shear resistance is always determined in cross-section Classes 1, 2 and 3 on the basis of plastic shear resistance (see clause 2.7.2).

If there is in addition to bending moment and shear force also normal force acting in the cross-section, the combined effect shall be verified as presented in clause 2.9.1.3.2 for combined bending and shear, however with the difference that [13,14]:

- $M_{pl.Rd}$ shall be substituted by reduced bending resistance $M_{N.Rd}$ which shall be calculated on the basis of clause 2.9.1.5.1 (by assuming a cross-section that consists according to its cross-section Class of effective flanges and independently of its cross-section Class of fully effective web, as presented in clause 2.9.1.3.2)

- $M_{f.Rd}$ shall be substituted by the reduced bending resistance of the flanges $M_{N.f.Rd}$

The reduced bending resistance of the flanges $M_{N.f.Rd}$ can be calculated for square and rectangular hollow sections from the formula:

$$M_{N.f.Rd} = M_{f.Rd}\left[1 - \frac{N_{Ed}}{2A_f f_y/\gamma_{M0}}\right] \tag{2.145}$$

where the bending resistance of the flanges $M_{f.Rd}$ is calculated from expression (2.112) and the area of the flange A_f from expression (2.114).

If the normal force is so high that the webs are compressed on their entire height $(h - 3t)$, the aforementioned reductions shall not be done. In such case the combined effect of bending moment, normal force and shear force shall be checked directly using expression (2.109) presented in clause 2.9.1.3.2, but the value of the bending resistance of the flanges shall be set in this case as $M_{f.Rd} = 0$ and the value of factor $\bar{\eta}_1$ from expression (2.110) shall be substitued by the value of factor η_1 from expression (2.116) [13,14].

If shear force is present in the flange due to possible torsion or due to shear force parallel to flange, the combined effect shall be checked as presented in clause 2.9.1.3.2 for the corresponding situation. Thereby also the combined effect of the normal force will become taken into account. This procedure shall be followed no matter whether the webs are fully compressed or not [13,14].

2.9.1.7 Bending moment, normal force, shear force and torsional moment

Due to shear force, the verification of the combined effect is separated in Eurocode again into Parts EN 1993-1-1 and EN 1993-1-5 depending on whether shear buckling reduces the plastic resistance of the cross-section. The provisions of Part EN 1993-1-1 shall be applied when the shear resistance of the cross-section is equal to the plastic shear resistance $V_{pl.Rd}$, and the provisions of Part EN 1993-1-5 shall be applied when shear buckling reduces shear resistance, i.e. $V_{b.Rd} < V_{pl.Rd}$.

2.9.1.7.1 Bending moment, normal force, shear force and torsional moment, when shear buckling does not reduce shear resistance

As in clause 2.9.1.4, the additional effect of torsion need not be taken into account, if the shear force does not exceed half of the plastic shear resistance reduced due to torsion according to clause 2.9.1.2.1, i.e. $V_{Ed} \leq 0{,}5 V_{pl.T.Rd}$. In such case only the combined effect of bending moment and normal force needs to be checked according to clause 2.9.1.5.

In other case the combined effect shall be checked as without torsion (i.e. according to clause 2.9.1.6.1), but ρ shall be calculated taking into account the reducing effect of torsion according to clause 2.9.1.4.

2.9.1.7.2 Bending moment, normal force, shear force and torsional moment, when shear buckling reduces shear resistance

These assessments according to Part EN 1993-1-5 of Eurocode are needed only on some few square and rectangular hollow sections, because on circular hollow sections the shear resistance is always determined in cross-section Classes 1, 2 and 3 on the basis of plastic shear resistance (see clause 2.7.2).

The combined effect of bending moment, normal force, shear force and torsional moment can be checked according to clause 2.9.1.6.2, because as stated before, in verifications of Part 1993-1-5 of Eurocode the effects of possible torsion shall be taken into account in the acting shear force V_{Ed} (effect of torsion to the shear force V_{Ed}, see clause 2.9.1.2.2) [13,14].

2.9.2 Class 4 circular hollow section

Cross-section resistance of Class 4 circular hollow section can be checked for all combinations of forces and moments by using following stress-based verifications of axial stress and shear stress in the gross cross-section:

The axial tensile stress is not allowed at any point of the cross-section to exceed the design value of the yield strength, and the axial compressive stress is not allowed to exceed the design value of the buckling strength [16]:

$$\sigma_{x.Ed} = \frac{N_{Ed}}{A} \pm \frac{M_{Ed}}{W_{el}} \tag{2.146}$$

$$\sigma_{x.Ed} \le \frac{f_y}{\gamma_{M0}} \qquad \textit{when } \sigma_{x.Ed} \textit{ is tension} \tag{2.147}$$

$$\sigma_{x.Ed} \le \frac{\chi_x f_y}{\gamma_{M1}} \qquad \textit{when } \sigma_{x.Ed} \textit{ is compression} \tag{2.148}$$

where $\sigma_{x.Ed}$ is the design value of axial stress at the point to be considered
N_{Ed} is the design value of normal force at ultimate limit state (compression is positive)
A is the gross cross-section area (Annex 11.1)
M_{Ed} is the design value of bending moment at ultimate limit state, $M_{Ed} = \sqrt{M_{y.Ed}^2 + M_{z.Ed}^2}$
W_{el} is the elastic section modulus of the cross-section (Annex 11.1)
χ_x is the reduction factor for elastic-plastic local buckling according to clause 2.5.2.2
f_y is the nominal yield strength of the material
γ_{M0} is the partial safety factor for resistance **according to EN 1993-1-1** (Table 2.5)
γ_{M1} is the partial safety factor for resistance **according to EN 1993-1-6** (Table 2.5)

The maximum value of shear stress τ_{Ed} caused by shear force and/or torsion is not allowed to exceed the design value of shear buckling strength [16]:

$$\tau_{Ed} = \frac{V_{Ed}}{\pi r_m t} + \frac{M_{x.Ed}}{W_t} \tag{2.149}$$

$$\tau_{Ed} \le \frac{\chi_\tau f_y / \sqrt{3}}{\gamma_{M1}} \tag{2.150}$$

where τ_{Ed} is the maximum value of elastic shear stress
V_{Ed} is the design value of the shear force at ultimate limit state, $V_{Ed} = \sqrt{V_{y.Ed}^2 + V_{z.Ed}^2}$
r_m is the radius of the centreline of the hollow section wall thickness
t is the wall thickness

$M_{x.Ed}$ is the design value of torsional moment at ultimate limit state
W_t is the torsional section modulus of the cross-section (Annex 11.1)
χ_τ is the reduction factor for elastic-plastic shear buckling according to clause 2.7.2.2
f_y is the nominal yield stength of the material
γ_{M1} is the partial safety factor for resistance **according to EN 1993-1-6** (Table 2.5)

Combined effect of simultaneous normal stress and shear stress can be checked at the critical point of the cross-section by using following simplified stress verification that bases on the local buckling strengths, which has been derived from the interaction formula presented in Part EN 1993-1-6 of Eurocode for shell structures for a general case [16]:

$$\left[\frac{\sigma_{x.Ed}}{\chi_x f_y / \gamma_{M1}}\right]^{k_x} + \left[\frac{\tau_{Ed}}{\chi_\tau (f_y / \sqrt{3}) / \gamma_{M1}}\right]^{k_\tau} \leq 1,0 \qquad (2.151)$$

$$k_x = 1,25 + 0,75\chi_x \qquad (2.152)$$

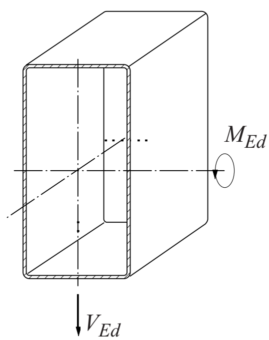
$$k_\tau = 1,75 + 0,25\chi_\tau \qquad (2.153)$$

where $\sigma_{x.Ed}$ is the design value of axial stress at the point to be considered (compression is positive)
τ_{Ed} is the design value of shear stress at the point to be considered
χ_x is the reduction factor for elastic-plastic local buckling according to clause 2.5.2.2
χ_τ is the reduction factor for elastic-plastic shear buckling according to clause 2.7.2.2
f_y is the nominal yield strength of the material
γ_{M1} is the partial safety factor for resistance **according to EN 1993-1-6** (Table 2.5)

If the axial stress is tension, $\sigma_{x.Ed} = 0$ shall be applied in expression (2.151) [16].

Example 2.18

Calculate the resistance of a hollow section 400×200×6 when subjected to combined bending moment and shear force. Calculations of the shear buckling resistance already performed in Example 2.12 for the same hollow section can be utilised here, as well as the observation that shear buckling reduces the cross-section's shear resistance.



The steel grade is SSAB Domex Tube Double Grade, which fulfills the EN 10219 requirements for both steel grades S420MH and S355J2H. Thereby the design calculations may be performed at designer's own choice either according to grade S420 or grade S355. Grade S420 is chosen in this Example as design basis.

Loads:

$V_{Ed} = 650\ kN$
$M_{Ed} = 320\ kNm$

$A \quad = 6963\ mm^2$ (Annex 11.1)
$W_{pl} = 906{,}0 \cdot 10^3\ mm^3$ (Annex 11.1)
$f_y \quad = 420\ N/mm^2$
$\gamma_{M0} = 1{,}0$

$V_{bw.Rd} = 948{,}5\ kN$ *(contribution from the web to shear buckling resistance, obtained from Example 2.12)*

Check the cross-section classification (Table 2.9):

Flange:

$b/t = 200/6 = 33,3 \le 34,4$ (compression) $\Rightarrow$ Class 3

Web:

$h/t = 400/6 = 66,7 \le 95,8$ (bending) $\Rightarrow$ Class 3

The flanges and the whole cross-section are thus fully effective.

Calculate the bending resistance of the effective flanges only:

$$A_f = [A - 2(h - 3t)t]/2 = [6963 - 2 \cdot (400 - 3 \cdot 6) \cdot 6]/2 = 1190\ mm^2 = A_{f.eff}$$

$$M_{f.Rd} = \frac{A_{f.eff} f_y}{\gamma_{M0}} \cdot (h - t) = \frac{1190 \cdot 420}{1,0} \cdot (400 - 6) = 196,9\ kNm < M_{Ed} = 320\ kNm$$

The bending resistance of the flanges alone is not sufficient to carry the acting bending moment
$\Rightarrow$ *the webs are needed to supplement the flanges, consequently the combined effect of bending and shear has to be checked.*

Check the magnitude of the shear force against the web's shear buckling resistance:

$$V_{Ed} = 650\ kN > 0,5 V_{bw.Rd} = 0,5 \cdot 948,5 = 474,3\ kN$$

The shear force is more than half of the web's contribution to shear buckling resistance, thus the shear force reduces the bending resistance.

The plastic bending resistance of the effective flanges supplemented with the web (which is assumed to be fully effective), is here equal to the gross cross-section's plastic bending resistance:

$$M_{pl.Rd} = \frac{W_{pl} f_y}{\gamma_{M0}} = \frac{906,0 \cdot 10^3 \cdot 420}{1,0} = 380,5\ kNm$$

The factors needed when verifying the design condition for combined effect:

$$\bar{\eta}_1 = \frac{M_{Ed}}{M_{pl.Rd}} = \frac{320,0}{380,5} = 0,8410$$

$$\bar{\eta}_3 = \frac{V_{Ed}}{V_{bw.Rd}} = \frac{650,0}{948,5} = 0,6853$$

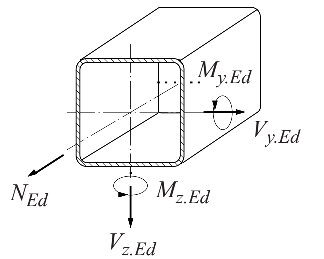
Check the design condition for combined effect:

$$\bar{\eta}_1 + \left[1 - \frac{M_{f.Rd}}{M_{pl.Rd}}\right](2\bar{\eta}_3 - 1)^2 = 0,8410 + \left[1 - \frac{196,9}{380,5}\right](2 \cdot 0,6853 - 1)^2 = 0,9073 \le 1,0 \quad OK$$

Comparison S420 vs S355:
In case the design calculations would be performed according to grade S355, the calculations would follow basically the same principles (i.e. shear buckling determines the shear resistance, and shear force reduces bending resistance). If S355, the cross-section would withstand both loadings separately, but not their combined effect, since the last interaction formula would result in 1,11 > 1,0 telling the resistance is not sufficient. By comparing the 'utilisation ratios' of the interaction formula, we could see that in this Example the increase of the material strength S355 → S420 improves the overall resistance approximately in the same respect with the yield strengths, telling the increase of the yield strength can be fully utilised.

Example 2.19

Check the resistance of the hollow section 200×200×8 from Example 2.4, when subjected to the load combination shown in the figure.



The steel grade is SSAB Domex Tube Double Grade, which fulfills the EN 10219 requirements for both steel grades S420MH and S355J2H. Thereby the design calculations may be performed at designer's own choice either according to grade S420 or grade S355. Grade S420 is chosen in this Example as design basis.

Loads:

$N_{Ed} = 1400$ kN
$M_{y.Ed} = 55$ kNm
$M_{z.Ed} = 50$ kNm
$V_{z.Ed} = 500$ kN
$V_{y.Ed} = 200$ kN

$A = 5924\ mm^2$ (Annex 11.1)
$W_{pl} = 420,9 \cdot 10^3\ mm^3$ (Annex 11.1)
$f_y = 420\ N/mm^2$
$\gamma_{M0} = 1,0$
$\gamma_{M1} = 1,0$

Check the cross-section classification using a conservative simplification by assuming pure uniform compression (Table 2.9):

$h/t = b/t = 200/8 = 25 \leq 27{,}7 \qquad \Rightarrow$ *Class 1*

The procedure to calculate the shear resistance depends on the slenderness of the webs (Table 2.19):

$h/t = 200/8 = 25 \leq 56{,}9$

$\Rightarrow$ *shear buckling does not reduce the shear resistance, thus the shear resistance of the cross-section shall be determined according to the plastic shear resistance (in direction of both axes)*

The plastic shear resistance of the cross-section is here equal in direction of both axes:

$$A_V = A \cdot \frac{h}{b+h} = 5924 \cdot \frac{200}{200+200} = 2962\ mm^2$$

$$V_{pl.Rd} = A_V \cdot \frac{f_y/\sqrt{3}}{\gamma_{M0}} = 2962 \cdot \frac{420/\sqrt{3}}{1,0} = 718,2\ kN$$

Bending about y-axis:

The plastic bending resistance of the cross-section:

$$M_{pl.y.Rd} = \frac{W_{pl.y} f_y}{\gamma_{M0}} = \frac{420,9 \cdot 10^3 \cdot 420}{1,0} = 176,8\ kNm$$

Check whether the shear force has to be considered for combined effect:

$$V_{z.Ed} = 500\ kN > 0,5 V_{z.pl.Rd} = 0,5 \cdot 718,2 = 359,1\ kN$$

$\Rightarrow$ *the shear force needs to be considered for combined effect in z-direction*

The proportional reduction of the bending resistance caused by the shear force is determined by the factor ρ*:*

$$\rho_z = \left[\frac{2V_{z.Ed}}{V_{z.pl.Rd}} - 1\right]^2 = \left[\frac{2 \cdot 500}{718,2} - 1\right]^2 = 0,1540$$

The bending resistance reduced due to shear force:

$$M_{V.y.Rd} = \left[W_{pl.y} - \frac{\rho_z A_V^2}{8t}\right]\left(\frac{f_y}{\gamma_{M0}}\right) = \left[420,9 \cdot 10^3 - \frac{0,1540 \cdot 2962^2}{8 \cdot 8}\right]\left(\frac{420}{1,0}\right) = 167,9\ kNm$$

Next, check the conditions given in expression (2.137a) to find out whether the effect of the normal force has to be considered for the bending resistance:

the calculatory areas reduced due to shear force:

$$A_{w.red} = (1 - \rho_z)(A - 2bt) = (1 - 0,1540) \cdot (5924 - 2 \cdot 200 \cdot 8) = 2305\ mm^2$$

$$A_{tot.red} = A - \rho_z(A - 2bt) = 5924 - 0,1540 \cdot (5924 - 2 \cdot 200 \cdot 8) = 5505\ mm^2$$

the normal force resistance reduced due to shear force:

$$N_{V.Rd} = A_{tot.red} f_y / \gamma_{M0} = 5505 \cdot 420 / 1,0 = 2312 \ kN \geq N_{Ed} \qquad OK$$

$$N_{Ed} = 1400 \ kN > 0,25 N_{V.Rd} = 0,25 \cdot 2312 = 578,0 \ kN$$

$\Rightarrow$ *the acting normal force exceeds at least this limit value of those two limit values given in expression (2.137a), thus the effect of the normal force has to be considered when determining the bending resistance*

$$n_V = N_{Ed} / N_{V.Rd} = 1400 / 2312 = 0,6055$$

$$a_V = A_{w.red} / A_{tot.red} = 2305 / 5505 = 0,4187 \leq 0,5$$

$$M_{N.V.y.Rd} = M_{V.y.Rd} \frac{1 - n_V}{1 - 0,5 a_V} = 167,9 \cdot \frac{1 - 0,6055}{1 - 0,5 \cdot 0,4187} = 83,8 \ kNm \leq M_{V.y.Rd} = 167,9 \ kNm$$

$$M_{N.V.y.Rd} = 83,8 \ kNm \geq M_{y.Ed} \qquad OK$$

Bending about z-axis:

The plastic bending resistance of the cross-section:

$$M_{pl.z.Rd} = \frac{W_{pl.z} f_y}{\gamma_{M0}} = \frac{420,9 \cdot 10^3 \cdot 420}{1,0} = 176,8 \ kNm$$

Check whether the shear force has to be considered for combined effect:

$$V_{y.Ed} = 200 \ kN \leq 0,5 V_{y.pl.Rd} = 0,5 \cdot 718,2 = 359,1 \ kN$$

$\Rightarrow$ *the shear force does not need to be considered for combined effect in y-direction, thus the combined effect about z-axis can be checked according to (M+N) interaction only:*

Next, check the conditions given in expression (2.124a) to find out whether the effect of the normal force has to be considered for the bending resistance:

$$N_{pl.Rd} = \frac{A f_y}{\gamma_{M0}} = \frac{5924 \cdot 420}{1,0} = 2488 \ kN$$

$$N_{Ed} = 1400 \ kN > 0,25 N_{pl.Rd} = 0,25 \cdot 2488 = 622,0 \ kN$$

$\Rightarrow$ *the acting normal force exceeds at least this limit value of those two limit values given in expression (2.124a), thus the effect of the normal force has to be considered when determining the bending resistance*

$$n = N_{Ed} / N_{pl.Rd} = 1400 / 2488 = 0,5627$$

$$a_f = (A - 2ht) / A = (5924 - 2 \cdot 200 \cdot 8) / 5924 = 0,4598 \leq 0,5$$

$$M_{N.z.Rd} = M_{pl.z.Rd} \frac{1 - n}{1 - 0,5 a_f} = 176,8 \cdot \frac{1 - 0,5627}{1 - 0,5 \cdot 0,4598} = 100,4 \ kNm \leq M_{pl.z.Rd} = 176,8 \ kNm$$

$$M_{N.z.Rd} = 100,4 \ kNm$$

As concluded earlier, the shear force in y-direction is so small, that it can be neglected when determining the bending resistance about z-axis. Thus we can take:

$$M_{N.V.z.Rd} = M_{N.z.Rd} = 100,4\ kNm \geq M_{z.Ed} \qquad OK$$

Combined effects in biaxial bending:

Calculate factor α for bending about y-axis (the effect of the shear force is considered by using here factor n_V instead of factor n):

$$\alpha = \frac{1,66}{1 - 1,13n_V^2} = \frac{1,66}{1 - 1,13 \cdot 0,6055^2} = 2,834 \leq 6$$

Calculate factor β for bending about z-axis (as concluded earlier, the effect of the shear force can be neglected in this direction, thus $n_V = n$):

$$\beta = \frac{1,66}{1 - 1,13n_V^2} = \frac{1,66}{1 - 1,13n^2} = \frac{1,66}{1 - 1,13 \cdot 0,5627^2} = 2,585 \leq 6$$

Check the design condition (2.141) for combined effect:

$$\left[\frac{M_{y.Ed}}{M_{N.V.y.Rd}}\right]^{\alpha} + \left[\frac{M_{z.Ed}}{M_{N.V.z.Rd}}\right]^{\beta} = \left[\frac{55}{83,8}\right]^{2,834} + \left[\frac{50}{100,4}\right]^{2,585} = 0,4681 \leq 1,0 \qquad OK$$

Comparison S420 vs S355:
In case the design calculations would be performed according to grade S355, the calculations would follow basically the same principles, but already when doing the verifications about y-axis, it would be concluded that the cross-section is not strong enough to withstand the (M+V) interaction.

2.10 Resistance of a structural hollow section subject to combined actions, when buckling or lateral-torsional buckling can occur

2.10.1 Bending moment and normal force

Unless second-order analysis is performed, the resistance of a member subject to bending moment and normal force shall be checked according to the following provisions when buckling or lateral-torsional buckling can take place. (Additionally the resistances of the cross-section shall be checked according to clause 2.9 at the critical points of the member.)

For the members of a structural system, verification of the resistances can be performed basing on assessment of single members, which are thought to be separated from the structure. On sway structures, second-order effects (P-Δ-effects) are taken into account either by end moments of members or by using appropriate buckling lengths (see clause 7.2.1) [3,4,5].

In the interaction formulae the influence of second-order effects to resistance is taken into account in respect to the considered member itself. The formulae are based on the modelling of simply supported single span members with end-fork-conditions (with or without continuous lateral restraints between the supports), and the member is subject to compression, end moments and/or transverse loads. The combined effect of possible transverse loads and shear force will be taken into account indirectly through the equivalent uniform moment factors which depend on the form of the moment diagram (Table 2.25).

For members subject to combined bending and axial compression the following design conditions shall be checked [3,4,5]:

$$\frac{N_{Ed}}{\chi_y \frac{N_{Rk}}{\gamma_{M1}}} + k_{yy} \frac{M_{y.Ed} + \Delta M_{y.Ed}}{\chi_{LT} \frac{M_{y.Rk}}{\gamma_{M1}}} + k_{yz} \frac{M_{z.Ed} + \Delta M_{z.Ed}}{\frac{M_{z.Rk}}{\gamma_{M1}}} \leq 1,0 \tag{2.154}$$

$$\frac{N_{Ed}}{\chi_z \frac{N_{Rk}}{\gamma_{M1}}} + k_{zy} \frac{M_{y.Ed} + \Delta M_{y.Ed}}{\chi_{LT} \frac{M_{y.Rk}}{\gamma_{M1}}} + k_{zz} \frac{M_{z.Ed} + \Delta M_{z.Ed}}{\frac{M_{z.Rk}}{\gamma_{M1}}} \leq 1,0 \tag{2.155}$$

where N_{Ed}, $M_{y.Ed}$ and $M_{z.Ed}$ are the design values of the compression force and maximum moments of the member

$\Delta M_{y.Ed}$ and $\Delta M_{z.Ed}$ are the moments due to the shift of the neutral axis on Class 4, see also the provisions in Table 2.22

N_{Rk}, $M_{y.Rk}$ and $M_{z.Rk}$ are according to cross-section Class the characteristic values of the compression resistance and bending resistances (Table 2.22)

χ_y and χ_z are reduction factors for flexural buckling according to clause 2.5.3

χ_{LT} is the reduction factor for lateral-torsional buckling according to clause 2.6.5

k_{yy}, k_{yz}, k_{zy} and k_{zz} are interaction factors (Tables 2.23 and 2.24)

The interaction factors k_{yy}, k_{yz}, k_{zy} and k_{zz} are presented in EN 1993-1-1 for two alternative methods. Method 1 is more accurate, but the calculations are more complex. Method 2 gives results which are more conservative, but the calculations are simpler. The National Annex defines which method shall be used.

Finnish National Annex to standard EN 1993-1-1 [6]:

Method 2 is used if it is applicable.
Also Method 1 may be used.

The interaction factors given in EN 1993-1-1 for Method 2 are presented in Tables 2.23 and 2.24. The tabulated values for Method 2 are separated in EN 1993-1-1 on product basis into two groups: I-sections and rectangular hollow sections (including also square hollow sections). In this handbook the tabulated values of Method 2 are presented only in respect to hollow sections.

For circular hollow sections there are no specific guidelines presented regarding Method 2. Nevertheless, according to [35] Method 2 can be applied also for circular hollow sections. Thereby all members, so also circular hollow sections, can be in practice verified according to Method 2. **Also in this handbook it is taken that the provisions given in Method 2 for rectangular hollow sections shall be applied also for circular hollow sections.**

Circular and square hollow sections are in practice not susceptible to lateral-torsional buckling, so their lateral-torsional buckling need not be taken into account, and the value of factor χ_{LT} can be taken as χ_{LT} = 1,0. The interaction factors k_{yy}, k_{yz}, k_{zy} and k_{zz} are obtained from Table 2.23.

On rectangular hollow sections lateral-torsional buckling can, however, turn out to be critical. If it has been verified according to clauses 2.6.5 or 2.6.6 that lateral-torsional buckling is not critical, the value of factor χ_{LT} can be taken as χ_{LT} = 1,0 and the interaction factors k_{yy}, k_{yz}, k_{zy} and k_{zz} are obtained from Table 2.23. If lateral-torsional buckling is critical, the reduction factor χ_{LT} for lateral-torsional buckling shall be calculated according to clause 2.6.5 and the interaction factors k_{yy}, k_{yz}, k_{zy} and k_{zz} are obtained from Table 2.24.

Table 2.22 Partial safety factors γ_{M1} and values for N_{Rk} , $M_{i.Rk}$ and $\Delta M_{i.Ed}$ in different cross-section Classes

	Class 1, 2 and 3			**Class 4**	
	Square, rectangular and circular hollow sections			Square and rectangular hollow sections	Circular hollow sections
Variable	Class 1	Class 2	Class 3	Class 4	Class 4
N_{Rk}	$f_y A$	$f_y A$	$f_y A$	$f_y A_{eff}$	$\chi_x f_y A$ b)
$M_{y.Rk}$	$f_y W_{pl.y}$	$f_y W_{pl.y}$	$f_y W_{el.y}$	$f_y W_{eff.y}$	$\chi_x f_y W_{el.y}$ b)
$M_{z.Rk}$	$f_y W_{pl.z}$	$f_y W_{pl.z}$	$f_y W_{el.z}$	$f_y W_{eff.z}$	$\chi_x f_y W_{el.z}$ b)
$\Delta M_{y.Ed}$	0	0	0	$e_{Ny} N_{Ed}$ a)	0
$\Delta M_{z.Ed}$	0	0	0	$e_{Nz} N_{Ed}$ a)	0
χ_{M1}	see Table 2.5 (EN 1993-1-1)	see Table 2.5 (EN 1993-1-1)	see Table 2.5 (EN 1993-1-1)	see Table 2.5 (EN 1993-1-1)	see Table 2.5 **(EN 1993-1-6)**

a) Because of double symmetry, on stuctural hollow sections e_{Ny} = 0 and e_{Nz} = 0.
b) The reduction factor χ_x for elastic-plastic local buckling is calculated according to clause 2.5.2.2.

Table 2.23 Interaction factors k_{ij} for members not susceptible to torsional deformations [3,4,5]

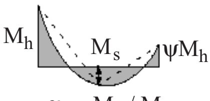
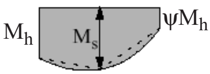
Interaction factors	Cross-section type	Design assumptions	
		Elastic cross-section properties Class 3 and 4	Plastic cross-section properties Class 1 and 2
k_{yy}	Rectangular hollow sections	$C_{my}\left[1+0,6\bar{\lambda}_y \frac{N_{Ed}}{\chi_y N_{Rk}/\gamma_{M1}}\right]$ $\leq C_{my}\left[1+0,6 \frac{N_{Ed}}{\chi_y N_{Rk}/\gamma_{M1}}\right]$	$C_{my}\left[1+(\bar{\lambda}_y-0,2) \frac{N_{Ed}}{\chi_y N_{Rk}/\gamma_{M1}}\right]$ $\leq C_{my}\left[1+0,8 \frac{N_{Ed}}{\chi_y N_{Rk}/\gamma_{M1}}\right]$
k_{yz}	Rectangular hollow sections	k_{zz}	0,6 k_{zz}
k_{zy}	Rectangular hollow sections	0,8 k_{yy}	0,6 k_{yy}
k_{zz}	Rectangular hollow sections	$C_{mz}\left[1+0,6\bar{\lambda}_z \frac{N_{Ed}}{\chi_z N_{Rk}/\gamma_{M1}}\right]$ $\leq C_{mz}\left[1+0,6 \frac{N_{Ed}}{\chi_z N_{Rk}/\gamma_{M1}}\right]$	$C_{mz}\left[1+(\bar{\lambda}_z-0,2) \frac{N_{Ed}}{\chi_z N_{Rk}/\gamma_{M1}}\right]$ $\leq C_{mz}\left[1+0,8 \frac{N_{Ed}}{\chi_z N_{Rk}/\gamma_{M1}}\right]$

- On rectangular hollow sections subject to axial compression and uniaxial bending $M_{y.Ed}$, the value for factor k_{zy} can be taken as $k_{zy} = 0$.
- The limit values presented in the table for the factors k_{ij} do not represent validity conditions, but limit values to be used in the calculations for the relevant factor.

Table 2.24 Interaction factors k_{ij} for members susceptible to torsional deformations [3,4,5]

Interaction factors	Design assumptions	
	Elastic cross-section properties Class 3 and 4	Plastic cross-section properties Class 1 and 2
k_{yy}	k_{yy} according to Table 2.23	k_{yy} according to Table 2.23
k_{yz}	k_{yz} according to Table 2.23	k_{yz} according to Table 2.23
k_{zy}	$\left[1 - \frac{0,05\bar{\lambda}_z}{(C_{mLT} - 0,25)} \cdot \frac{N_{Ed}}{\chi_z N_{Rk}/\gamma_{M1}}\right]$ $\geq \left[1 - \frac{0,05}{(C_{mLT} - 0,25)} \cdot \frac{N_{Ed}}{\chi_z N_{Rk}/\gamma_{M1}}\right]$	$\left[1 - \frac{0,1\bar{\lambda}_z}{(C_{mLT} - 0,25)} \cdot \frac{N_{Ed}}{\chi_z N_{Rk}/\gamma_{M1}}\right]$ $\geq \left[1 - \frac{0,1}{(C_{mLT} - 0,25)} \cdot \frac{N_{Ed}}{\chi_z N_{Rk}/\gamma_{M1}}\right]$ when $\bar{\lambda}_z < 0,4$: $k_{zy} = 0,6 + \bar{\lambda}_z$ $\leq \left[1 - \frac{0,1\bar{\lambda}_z}{(C_{mLT} - 0,25)} \cdot \frac{N_{Ed}}{\chi_z N_{Rk}/\gamma_{M1}}\right]$
k_{zz}	k_{zz} according to Table 2.23	k_{zz} according to Table 2.23
The limit values presented in the table for the factors k_{ij} do not represent validity conditions, but limit values to be used in the calculations for the relevant factor.		

Table 2.25 Equivalent uniform moment factors C_{mi} in Tables 2.23 and 2.24 [3,4,5]

Moment diagram	Range		C_{my} and C_{mz} and C_{mLT}: Uniform load	C_{my} and C_{mz} and C_{mLT}: Concentrated load
	$-1 \leq \psi \leq 1$		$0,6 + 0,4\psi \geq 0,4$	
 $\alpha_s = M_s / M_h$	$0 \leq \alpha_s \leq 1$	$-1 \leq \psi \leq 1$	$0,2 + 0,8\alpha_s \geq 0,4$	$0,2 + 0,8\alpha_s \geq 0,4$
	$-1 \leq \alpha_s < 0$	$0 \leq \psi \leq 1$	$0,1 - 0,8\alpha_s \geq 0,4$	$-0,8\alpha_s \geq 0,4$
		$-1 \leq \psi < 0$	$0,1(1-\psi) - 0,8\alpha_s \geq 0,4$	$0,2(-\psi) - 0,8\alpha_s \geq 0,4$
 $\alpha_h = M_h / M_s$	$0 \leq \alpha_h \leq 1$	$-1 \leq \psi \leq 1$	$0,95 + 0,05\alpha_h$	$0,90 + 0,10\alpha_h$
	$-1 \leq \alpha_h < 0$	$0 \leq \psi \leq 1$	$0,95 + 0,05\alpha_h$	$0,90 + 0,10\alpha_h$
		$-1 \leq \psi < 0$	$0,95 + 0,05\alpha_h(1+2\psi)$	$0,90 + 0,10\alpha_h(1+2\psi)$ a)

For members with sway buckling mode the equivalent uniform moment factor should be taken $C_{my} = 0,9$ or $C_{mz} = 0,9$ respectively.

The factors C_{my}, C_{mz} and C_{mLT} are obtained according to bending moment diagram between the relevant braced points as follows:

Moment factor	Bending axis	Points braced in direction
C_{my}	y-y	z-z
C_{mz}	z-z	y-y
C_{mLT}	y-y	y-y

a) The expression is corrected according to [5].

The limit values presented in the table for the factors k_{ij} do not represent validity conditions, but limit values to be used in the calculations for the relevant factor.

2.10.2 Bending moment, normal force and shear force

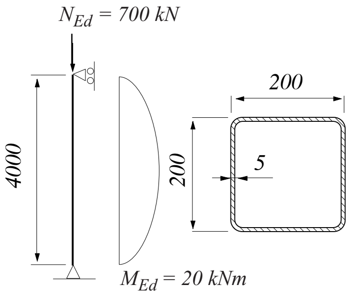
For a combination of simultaneous bending moment, normal force and shear force, Eurocode assigns specific interaction formula only in respect to cross-section resistance. For this combination of forces and moments there are no instructions presented for a member in respect to buckling or lateral-torsional buckling.

However, for a member prone to buckling or lateral-torsional buckling, the combined effect of (M+N+V) combination is included implicitly in the combined effect of (M+N) combination according to clause 2.10.1, where the combined effect of possible transverse loads and shear force will be taken into account indirectly through the equivalent uniform moment factors which depend on the form of the moment diagram (Table 2.25).

Example 2.20

Check the resistance of the hollow section 200×200×5 from Example 2.3, when subjected to the combination of normal force and bending moment caused by uniform transverse load. The transverse load acts only in z-direction. The member is simply supported at both ends about both axes.

The steel grade is SSAB Domex Tube Double Grade, which fulfills the EN 10219 requirements for both steel grades S420MH and S355J2H. Thereby the design calculations may be performed at designer's own choice either according to grade S420 or grade S355. Grade S420 is chosen in this Example as design basis.



A_{eff} $= 3312\ mm^2$ *when subjected to compression only (obtained from Example 2.3)*
W_{eff} $= 220{,}5 \cdot 10^3 mm^3$ *when subjected to bending only (obtained from Example 2.9)*
f_y $= 420\ N/mm^2$
γ_{M1} $= 1{,}0$

This Example is limited to check the combined effect of the loads only in respect to the stability of the member.

It is a task for the Reader to verify the resistance of the cross-section to single loads (N_{Ed}, M_{Ed}, V_{Ed}) and their combined effect at the critical points of the member (i.e. at support the effect of combined normal force and shear force, and at mid-span the effect of combined bending moment and normal force).

Based on Example 2.3, it can be easily concluded that the cross-section shall be classified into Class 4 (in case of bending the compressed flange determines the cross-section Class).

The non-dimensional slenderness, the reduction factor for flexural buckling, and the buckling resistance of the member can be obtained from Example 2.3:

$\bar{\lambda} = 0{,}6675$
$\chi = 0{,}7447$
$N_{b.Rd} = 1036\ kN$

The values given above are the same about both axes, since the cross-section is symmetrical and the buckling length (the restraint conditions of the member) is here the same about both axes.

Since the form of the hollow section is square, it can be concluded that the member is not susceptible to lateral torsional buckling, i.e. lateral-torsional buckling does not reduce the bending resistance. Consequently, the reduction factor for lateral-torsional buckling can be taken as $\chi_{LT} = 1{,}0$ and interaction factors k_{yy}, k_{yz}, k_{zy} and k_{zz} can be obtained from Table 2.23. Since bending moment acts only about y-axis ($M_{z.Ed} = 0$), only the interaction factors k_{yy} and k_{zy} are needed here.

When verifying the combined effect for the member, the characteristic values of the cross-section's compression resistance and bending resistance are needed:

$$N_{Rk} = f_y A_{eff} = 420 \cdot 3312 = 1391\ kN$$

$$M_{y.Rk} = M_{z.Rk} = f_y W_{eff} = 420 \cdot 220,5 \cdot 10^3 = 92,6\ kNm$$

Factor C_{my}, which depends on the form of the moment diagram, can be obtained from Table 2.25 by applying factor α_h given in the same table:

$$\alpha_h = \frac{M_h}{M_s} = 0$$

$$C_{my} = 0,95 + 0,05\alpha_h = 0,95 + 0 = 0,95$$

Next, calculate the interaction factors k_{yy} and k_{zy} by applying Table 2.23:

$$k_{yy} = C_{my}\left[1 + 0,6\bar{\lambda}_y \frac{N_{Ed}}{\chi_y N_{Rk}/\gamma_{M1}}\right]$$

$$= 0,95 \cdot \left[1 + 0,6 \cdot 0,6675 \cdot \frac{700}{0,7447 \cdot 1391/1,0}\right] = 1,207$$

however, the maximum value of k_{yy} is limited to:

$$k_{yy} = 1,207 \le C_{my}\left[1 + 0,6\frac{N_{Ed}}{\chi_y N_{Rk}/\gamma_{M1}}\right] = 0,95 \cdot \left[1 + 0,6 \cdot \frac{700}{0,7447 \cdot 1391/1,0}\right] = 1,335$$

$$k_{zy} = 0,8 \cdot k_{yy} = 0,8 \cdot 1,207 = 0,9656$$

Check the design condition for combined normal force and bending moment:

$$\frac{N_{Ed}}{\chi_y \frac{N_{Rk}}{\gamma_{M1}}} + k_{yy}\frac{M_{y.Ed} + \Delta M_{y.Ed}}{\chi_{LT}\frac{M_{y.Rk}}{\gamma_{M1}}} + k_{yz}\frac{M_{z.Ed} + \Delta M_{z.Ed}}{\frac{M_{z.Rk}}{\gamma_{M1}}} =$$

$$\frac{700}{0,7447 \cdot \frac{1391}{1,0}} + 1,207 \cdot \frac{20 + 0}{1,0 \cdot \frac{92,6}{1,0}} + 0 = 0,9364 \le 1,0 \qquad OK$$

in addition, the following design condition needs to be checked:

$$\frac{N_{Ed}}{\chi_z \frac{N_{Rk}}{\gamma_{M1}}} + k_{zy}\frac{M_{y.Ed} + \Delta M_{y.Ed}}{\chi_{LT}\frac{M_{y.Rk}}{\gamma_{M1}}} + k_{zz}\frac{M_{z.Ed} + \Delta M_{z.Ed}}{\frac{M_{z.Rk}}{\gamma_{M1}}} =$$

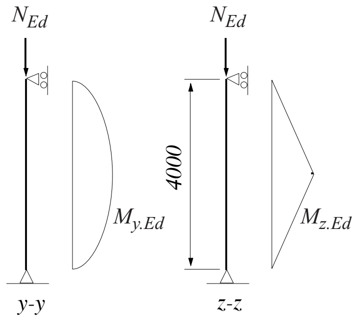
$$\frac{700}{0,7447 \cdot \frac{1391}{1,0}} + 0,9656 \cdot \frac{20 + 0}{1,0 \cdot \frac{92,6}{1,0}} + 0 = 0,8843 \le 1,0 \qquad OK$$

Comparison S420 vs S355:

In case the design calculations would be performed according to grade S355, the result of the (M+N) interaction verification would be (S420: 0,9364 →) S355: 1,037 > 1,0, telling the member is not strong enough. By comparing the 'utilisation ratios' of the interaction formula, we can see that in this Example the increase of the material strength S355 → S420 improves the overall resistance approximately 10 %.

Example 2.21

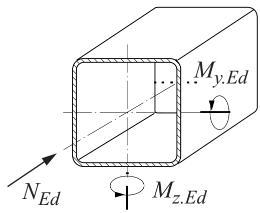
Check the resistance of the hollow section 160×160×5 from Example 2.8, when subjected to the load combination shown in the figure. The buckling length for flexural buckling is 4 m about both axes, and the member is simply supported at both ends about both axes.



The steel grade is SSAB Domex Tube Double Grade, which fulfills the EN 10219 requirements for both steel grades S420MH and S355J2H. Thereby the design calculations may be performed at designer's own choice either according to grade S420 or grade S355. Grade S420 is chosen in this Example as design basis.

Loads:

N_{Ed} = 400 kN (compression)
$M_{y.Ed}$ = 12 kNm
$M_{z.Ed}$ = 12 kNm



A = 3036 mm^2 (Annex 11.1)
I = $1202 \cdot 10^4$ mm^4 (Annex 11.1)
W_{el} = $150{,}3 \cdot 10^3$ mm^3 (Annex 11.1)
f_y = 420 N/mm^2
γ_{M1} = 1,0

This Example is limited to check the combined effect of the loads only in respect to the stability of the member.

It is a task for the Reader to verify the resistance of the cross-section to single loads and their combined effect at the critical points of the member (i.e. in this Example at support and at mid-span).

Based on Example 2.8, it can be easily concluded, that the cross-section shall be classified into Class 3 (in case of bending the compressed flange determines the cross-section Class).

For compressed hollow section, the reduction factor χ for flexural buckling shall be calculated according to buckling curve c (clause 2.5.3.1):

$$N_{cr} = \frac{\pi^2 EI}{L_{cr}^2} = \frac{\pi^2 \cdot 210000 \cdot 1202 \cdot 10^4}{4000^2} = 1557\ kN$$

$$\bar{\lambda} = \sqrt{\frac{Af_y}{N_{cr}}} = \sqrt{\frac{3036 \cdot 420}{1557 \cdot 10^3}} = 0,9050 > 0,2$$

$$\Phi = 0,5 \cdot [1 + \alpha(\bar{\lambda} - 0,2) + \bar{\lambda}^2] = 0,5 \cdot [1 + 0,49 \cdot (0,9050 - 0,2) + 0,9050^2] = 1,082$$

$$\chi = \frac{1}{\Phi + \sqrt{\Phi^2 - \bar{\lambda}^2}} = \frac{1}{1,082 + \sqrt{1,082^2 - 0,9050^2}} = 0,5970 \le 1,0$$

The buckling resistance of the member for only the normal force:

$$N_{b.Rd} = \frac{\chi A f_y}{\gamma_{M1}} = \frac{0,5970 \cdot 3036 \cdot 420}{1,0} = 761,2\ kN \ge N_{Ed} = 400\ kN \quad OK$$

The values given above are the same about both axes, since the cross-section is symmetrical and the buckling length (the restraint conditions of the member) is here the same about both axes.

Since the form of the hollow section is square, it can be concluded that the member is not susceptible to lateral torsional buckling, i.e. lateral-torsional buckling does not reduce the bending resistance. Consequently, the reduction factor for lateral-torsional buckling can be taken as $\chi_{LT} = 1,0$ and interaction factors k_{yy}, k_{yz}, k_{zy} and k_{zz} can be obtained from Table 2.23.

When verifying the combined effects for the member, the characteristic values of the cross-section's compression resistance and bending resistance are needed:

$$N_{Rk} = f_y A = 420 \cdot 3036 = 1275\ kN$$

$$M_{y.Rk} = M_{z.Rk} = f_y W_{el} = 420 \cdot 150,3 \cdot 10^3 = 63,1\ kNm$$

Factors C_{my} and C_{mz}, which depend on the form of the moment diagram, can be obtained from Table 2.25 by applying factor α_h given in the same Table:

bending about y-axis (uniform loading):

$$\alpha_h = \frac{M_h}{M_s} = 0$$

$$C_{my} = 0,95 + 0,05\alpha_h = 0,95 + 0 = 0,95$$

bending about z-axis (concentrated load at mid-span):

$$\alpha_h = \frac{M_h}{M_s} = 0$$

$$C_{mz} = 0,90 + 0,10\alpha_h = 0,90 + 0 = 0,90$$

Interaction factors from Table 2.23:

bending about y-axis:

$$k_{yy} = C_{my}\left[1 + 0,6\bar{\lambda}_y \frac{N_{Ed}}{\chi_y N_{Rk}/\gamma_{M1}}\right]$$

$$= 0,95 \cdot \left[1 + 0,6 \cdot 0,9050 \cdot \frac{400}{0,5970 \cdot 1275/1,0}\right] = 1,221$$

however, the maximum value of k_{yy} *is limited to:*

$$k_{yy} = 1,221 \le C_{my}\left[1 + 0,6\frac{N_{Ed}}{\chi_y N_{Rk}/\gamma_{M1}}\right] = 0,95 \cdot \left[1 + 0,6 \cdot \frac{400}{0,5970 \cdot 1275/1,0}\right] = 1,250$$

$$k_{zy} = 0,8 \cdot k_{yy} = 0,8 \cdot 1,221 = 0,9768$$

bending about z-axis:

$$k_{zz} = C_{mz}\left[1 + 0,6\bar{\lambda}_z \frac{N_{Ed}}{\chi_z N_{Rk}/\gamma_{M1}}\right]$$

$$= 0,90 \cdot \left[1 + 0,6 \cdot 0,9050 \cdot \frac{400}{0,5970 \cdot 1275/1,0}\right] = 1,157$$

however, the maximum value of k_{zz} *is limited to:*

$$k_{zz} = 1,157 \le C_{mz}\left[1 + 0,6\frac{N_{Ed}}{\chi_z N_{Rk}/\gamma_{M1}}\right] = 0,90 \cdot \left[1 + 0,6 \cdot \frac{400}{0,5970 \cdot 1275/1,0}\right] = 1,184$$

$$k_{yz} = k_{zz} = 1,157$$

Check the design condition for combined normal force and bending moments:

$$\frac{N_{Ed}}{\chi_y \frac{N_{Rk}}{\gamma_{M1}}} + k_{yy}\frac{M_{y.Ed}}{\chi_{LT}\frac{M_{y.Rk}}{\gamma_{M1}}} + k_{yz}\frac{M_{z.Ed}}{\frac{M_{z.Rk}}{\gamma_{M1}}} =$$

$$\frac{400}{0,5970 \cdot \frac{1275}{1,0}} + 1,221 \cdot \frac{12}{1,0 \cdot \frac{63,1}{1,0}} + 1,157 \cdot \frac{12}{\frac{63,1}{1,0}} = 0,9777 \le 1,0 \quad OK$$

in addition, the following design condition has to be checked:

$$\frac{N_{Ed}}{\chi_z \frac{N_{Rk}}{\gamma_{M1}}} + k_{zy}\frac{M_{y.Ed}}{\chi_{LT}\frac{M_{y.Rk}}{\gamma_{M1}}} + k_{zz}\frac{M_{z.Ed}}{\frac{M_{z.Rk}}{\gamma_{M1}}} =$$

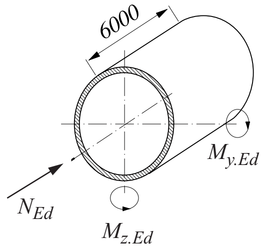
$$\frac{400}{0,5970 \cdot \frac{1275}{1,0}} + 0,9768 \cdot \frac{12}{1,0 \cdot \frac{63,1}{1,0}} + 1,157 \cdot \frac{12}{\frac{63,1}{1,0}} = 0,9313 \le 1,0 \quad OK$$

Comparison S420 vs S355:

In case the design calculations would be performed according to grade S355, the result of the (M+N) interaction verification would be (S420: 0,9777 →) S355: 1,114 > 1,0, telling *the member is not strong enough. By comparing the 'utilisation ratios' of the interaction formula, we can see that in this Example the increase of the material strength S355 → S420 improves the overall resistance almost 14 %.*

Example 2.22

Check the resistance of the Class 4 circular hollow section 323,9×5 from Example 2.17, when subjected to the load combination shown in the adjacent figure. The length of the member is 6 m. The bending moments about both axes are assumed to be constant within the whole length. The member is simply supported at both ends about both axes.



The steel grade is SSAB Domex Tube Double Grade, which fulfills the EN 10219 requirements for both steel grades S420MH and S355J2H. Thereby the design calculations may be performed at designer's own choice either according to grade S420 or grade S355. Grade S420 is chosen in this Example as design basis.

Loads:

N_{Ed} = 700 kN (compression)
$M_{y.Ed}$ = 18 kNm
$M_{z.Ed}$ = 18 kNm

A = 5009 mm^2 (Annex 11.1)
I = 6369 · 10^4 mm^4 (Annex 11.1)
W_{el} = 393,3 · 10^3 mm^4 (Annex 11.1)
f_y = 420 N/mm^2
γ_{M1} = 1,1 (Class 4 circular hollow section, EN 1993-1-6)

This Example is limited to consider the combined effect of the loads only in respect to the stability of the member.

It is a task for the Reader to verify the resistance of <u>the cross-section</u> to single loads and their combined effect.

The compression resistance $N_{c.Rd}$ of the cross-section and reduction factor χ_x for elastic-plastic local buckling are the same as calculated in Example 2.2:

χ_x = 0,8578
$N_{c.Rd}$ = 1641 kN

Next we can check flexural buckling of the member. The elastic critical force for flexural buckling:

$$N_{cr} = \frac{\pi^2 EI}{L_{cr}^2} = \frac{\pi^2 \cdot 210000 \cdot 6369 \cdot 10^4}{6000^2} = 3667\ kN$$

Non-dimensional slenderness of the member:

$$\bar{\lambda} = \sqrt{\frac{\chi_x A f_y}{N_{cr}}} = \sqrt{\frac{0,8578 \cdot 5009 \cdot 420}{3667 \cdot 10^3}} = 0,7015 > 0,2$$

Reduction factor for flexural buckling (buckling curve c: $\alpha = 0{,}49$):

$$\Phi = 0,5\cdot[1+\alpha(\bar{\lambda}-0,2)+\bar{\lambda}^2] = 0,5\cdot[1+0,49\cdot(0,7015-0,2)+0,7015^2] = 0,8689$$

$$\chi = \frac{1}{\Phi+\sqrt{\Phi^2-\bar{\lambda}^2}} = \frac{1}{0,8689+\sqrt{0,8689^2-0,7015^2}} = 0,7238 \le 1,0$$

The buckling resistance of the member is finally:

$$N_{b.Rd} = \chi\cdot N_{c.Rd} = 0,7238\cdot 1641 = 1188\ kN \ge N_{Ed} \quad OK$$

The values given above are the same about both axes, since the cross-section is symmetrical and the buckling length (the restraint conditions of the member) is here the same about both axes.

Since the form of the hollow section is circular, it can be concluded that the member is not susceptible to lateral torsional buckling, i.e. lateral torsional buckling does not reduce the bending resistance. Consequently, the reduction factor for lateral torsional buckling can be taken as $\chi_{LT} = 1{,}0$ and interaction factors k_{yy}, k_{yz}, k_{zy} and k_{zz} can be obtained from Table 2.23.

When verifying the combined effect for the member, the characteristic values of the cross-section's compression resistance and bending resistance are needed (for Class 4 circular hollow section, the characteristic values of the cross-section's compression resistance and bending resistance shall be calculated by using the buckling strength $\chi_x f_y$ of the cross-section):

$$N_{Rk} = \chi_x f_y A = 0,8578\cdot 420\cdot 5009 = 1805\ kN$$

$$M_{y.Rk} = M_{z.Rk} = \chi_x f_y W_{el} = 0,8578\cdot 420\cdot 393,3\cdot 10^3 = 141,7\ kNm$$

Factors C_{my} and C_{mz}, which depend on the form of the moment diagram, can be obtained from Table 2.25 by applying factor ψ, which is the ratio of the end moments of the member:

bending about y-axis (uniform bending moment):

$$\psi = 1$$

$$C_{my} = 0,6+0,4\psi = 0,6+0,4\cdot 1 = 1,0 \quad \text{(however 0,4 as minimum)}$$

for bending about z-axis, the results will be herein the same, i.e. $C_{mz} = C_{my} = 1{,}0$.

Interaction factors from Table 2.23:
bending about y-axis:

$$k_{yy} = C_{my}\left[1+0,6\bar{\lambda}_y\frac{N_{Ed}}{\chi_y N_{Rk}/\gamma_{M1}}\right]$$

$$= 1,0\cdot\left[1+0,6\cdot 0,7015\cdot\frac{700}{0,7238\cdot 1805/1,1}\right] = 1,248$$

however, the maximum value of k_{yy} is limited to:

$$k_{yy} = 1,248 \le C_{my}\left[1+0,6\frac{N_{Ed}}{\chi_y N_{Rk}/\gamma_{M1}}\right] = 1,0\cdot\left[1+0,6\cdot\frac{700}{0,7238\cdot 1805/1,1}\right] = 1,354$$

$$k_{zy} = 0,8\cdot k_{yy} = 0,8\cdot 1,248 = 0,9984$$

bending about z-axis:

$$k_{zz} = C_{mz}\left[1 + 0,6\bar{\lambda}_z \frac{N_{Ed}}{\chi_z N_{Rk}/\gamma_{M1}}\right]$$

$$= 1,0 \cdot \left[1 + 0,6 \cdot 0,7015 \cdot \frac{700}{0,7238 \cdot 1805/1,1}\right] = 1,248$$

however, the maximum value of k_{zz} is limited to:

$$k_{zz} = 1,248 \le C_{mz}\left[1 + 0,6\frac{N_{Ed}}{\chi_z N_{Rk}/\gamma_{M1}}\right] = 1,0 \cdot \left[1 + 0,6 \cdot \frac{700}{0,7238 \cdot 1805/1,1}\right] = 1,354$$

$$k_{yz} = k_{zz} = 1,248$$

Check the design condition for combined normal force and bending moments:

$$\frac{N_{Ed}}{\chi_y \dfrac{N_{Rk}}{\gamma_{M1}}} + k_{yy}\frac{M_{y.Ed}}{\chi_{LT}\dfrac{M_{y.Rk}}{\gamma_{M1}}} + k_{yz}\frac{M_{z.Ed}}{\dfrac{M_{z.Rk}}{\gamma_{M1}}} =$$

$$\frac{700}{0,7238 \cdot \dfrac{1805}{1,1}} + 1,248 \cdot \frac{18}{1,0 \cdot \dfrac{141,7}{1,1}} + 1,248 \cdot \frac{18}{\dfrac{141,7}{1,1}} = 0,9382 \le 1,0 \quad OK$$

in addition, the following design condition has to be checked:

$$\frac{N_{Ed}}{\chi_z \dfrac{N_{Rk}}{\gamma_{M1}}} + k_{zy}\frac{M_{y.Ed}}{\chi_{LT}\dfrac{M_{y.Rk}}{\gamma_{M1}}} + k_{zz}\frac{M_{z.Ed}}{\dfrac{M_{z.Rk}}{\gamma_{M1}}} =$$

$$\frac{700}{0,7238 \cdot \dfrac{1805}{1,1}} + 0,9984 \cdot \frac{18}{1,0 \cdot \dfrac{141,7}{1,1}} + 1,248 \cdot \frac{18}{\dfrac{141,7}{1,1}} = 0,9033 \le 1,0 \quad OK$$

Comparison S420 vs S355:
In case the design calculations would be performed according to grade S355, the result of the (M+N) interaction verification would be (S420: 0,9382 $\rightarrow$) S355: 1,058 > 1,0, telling the member is not strong enough. By comparing the 'utilisation ratios' of the interaction formula, we can see that in this Example the increase of the material strength S355 $\rightarrow$ S420 improves the overall resistance almost 13 %.

2.11 Resistance of a structural hollow section to concentrated loads

2.11.1 Resistance to concentrated loads

The provisions presented here are assigned for determining the resistance of beam elements to concentrated loads. The resistance to concentrated loads in respect to beam-to-column joints or lattice joints; see EN 1993-1-8.

Concentrated loads comprise, in addition to ordinary concentrated loads, also the support reactions of a beam.

Because the walls of the hollow sections are relatively thin, the local resistance of the hollow section's web shall be checked when a concentrated load is present. The wider the region onto which the concentrated load can be distributed, the higher resistance will be achieved.

In respect to hollow sections, Eurocode does not present specific instructions to determine their resistance to concentrated loads.

On circular hollow sections it is advisable to avoid the existence of localized concentrated loads by structural means, e.g. by using welded end-plates to carry the loads as shear force by the whole cross-section.

On square and rectangular hollow sections the resistance to concentrated loads can be determined by using, as a conservative estimation, the provisions given in EN 1993-1-3 for cold-formed sections, but applied for hollow sections as follows:

The design condition for the resistance to concentrated loads is written as:

$$\frac{F_{Ed}}{2F_{1w.Rd}} \leq 1,0 \qquad (2.156)$$

where F_{Ed} is the design value of the concentrated load applied to the hollow section at ultimate limit state

$F_{1w.Rd}$ is the design value of concentrated load resistance per one web

In expression (2.156) it has been assumed that the concentrated load F_{Ed} is applied to both webs of the hollow section symmetrically. Thereby the concentrated load resistance of the hollow section is the sum of the resistances of both webs. In non-symmetric load case the concentrated load resistance of each web shall be checked separately in respect to the portion of the concentrated load applied to it. As a conservative simplification it is always allowed to assume that each web carries the whole concentrated load alone.

Concentrated load resistance of one web is calculated from the following formula:

$$F_{1w.Rd} = C_F \cdot \frac{t^2 f_y}{\gamma_{M1}} \qquad (2.157)$$

where $F_{1w.Rd}$ is the design value of concentrated load resistance per one web

C_F is the factor according to the relevant load type from Table 2.26

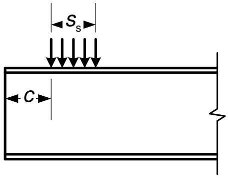
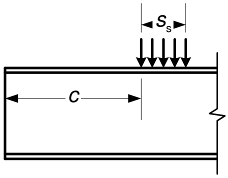
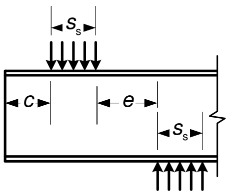
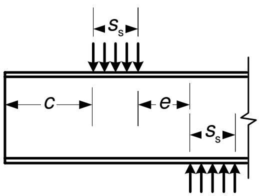
t is the wall thickness

f_y is the nominal yield strength of the material

γ_{M1} is the partial safety factor for resistance (Table 2.5)

In Table 2.26 the concentrated loads are separated into four cases according to whether the concentrated load acts on one side of the beam or on both opposite sides, and whether the concentrated load acts far from or near to the beam end. Concentrated load resistance is always substantially smaller, if the concentrated load acts near to the unstiffened end of the beam.

Table 2.26 Concentrated load resistance of square and rectangular hollow sections. Factor C_F for different load types

Load type		Concentrated load resistance
Type 1a	One concentrated load: $c \le 1{,}5\,(h - t)$ 	when $s_s / t \le 60$: $$C_F = k_1 k_2 k_3 \left[5{,}92 - \frac{(h-t)/t}{132}\right]\left[1 + 0{,}01\frac{s_s}{t}\right]$$ when $s_s / t > 60$: $$C_F = k_1 k_2 k_3 \left[5{,}92 - \frac{(h-t)/t}{132}\right]\left[0{,}71 + 0{,}015\frac{s_s}{t}\right]$$
Type 1b	One concentrated load: $c > 1{,}5\,(h - t)$ 	when $s_s / t \le 60$: $$C_F = k_3 k_4 k_5 \left[14{,}7 - \frac{(h-t)/t}{49{,}5}\right]\left[1 + 0{,}007\frac{s_s}{t}\right]$$ when $s_s / t > 60$: $$C_F = k_3 k_4 k_5 \left[14{,}7 - \frac{(h-t)/t}{49{,}5}\right]\left[0{,}75 + 0{,}011\frac{s_s}{t}\right]$$
Type 2a	Two opposite concentrated loads: $c \le 1{,}5\,(h - t)$ 	when $e \le 1{,}5\,(h - t)$: $$C_F = k_1 k_2 k_3 \left[6{,}66 - \frac{(h-t)/t}{64}\right]\left[1 + 0{,}01\frac{s_s}{t}\right]$$ Note a) and b)
Type 2b	Two opposite concentrated loads: $c > 1{,}5\,(h - t)$ 	when $e \le 1{,}5\,(h - t)$: $$C_F = k_3 k_4 k_5 \left[21{,}0 - \frac{(h-t)/t}{16{,}3}\right]\left[1 + 0{,}013\frac{s_s}{t}\right]$$ Note a) and b)
$k_1 = 1{,}33 - 0{,}33k$, $k_2 = 1{,}15 - 0{,}15(r_i/t)$ but $k_2 \ge 0{,}50$ and $k_2 \le 1{,}0$ $k_3 = 1{,}0$, $k_4 = 1{,}22 - 0{,}22k$, $k_5 = 1{,}06 - 0{,}06(r_i/t)$ but $k_5 \le 1{,}0$ where: r_i = internal corner radius , $k = f_y / 228$ $[f_y] = N/mm^2$		
a) In case of two equal opposite concentrated loads, if the lengths of stiff bearing s_s are unequal, the smaller value is used. b) If $e > 1{,}5\,(h - t)$, the case shall be assessed according to Type 1 for each of the single loads separately.		

In Table 2.26 a so-called length of stiff bearing s_s is needed , i.e. the length on which the concentrated load is assumed to calculatory apply on the hollow section. The length of stiff bearing can be determined on the basis of Figure 2.8. The load is thereby assumed to be distributed at 45º angle [13,14]. The measure s_s is not allowed to be larger than $(h - t)$.

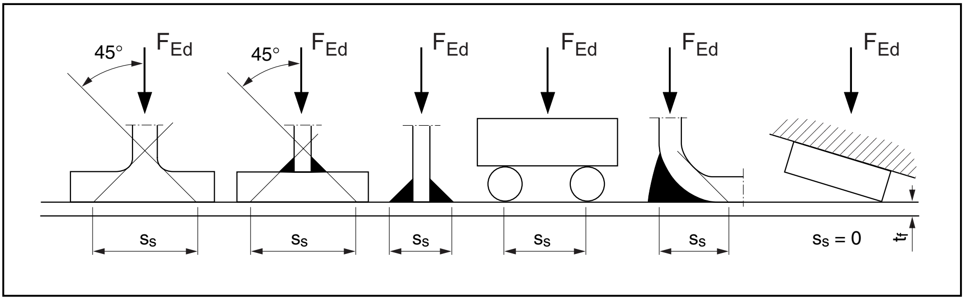


Figure 2.8 Concentrated load and the length of stiff bearing

If the bearing surface which applies the load rests at an angle to the surface of the hollow section (Figure 2.8, the last case), the length of stiff bearing shall be taken as s_s = 0 [13,14].

In the fourth case of Figure 2.8, when the load is applied through two bearing rollers, the verification shall be performed as two different cases: at first the resistance is checked using for the load F_{Ed} the length s_s according to Figure 2.8, and after that the situation is checked for each of the single loads $F_{Ed}/2$ separately using the length s_s = 0 [28].

Generally, if there are several concentrated loads near to each other, the resistance shall be checked for each of the single loads and also for the total load. In the latter case the value of length s_s is taken as the distance of the centres of the outermost concentrated loads [13,14].

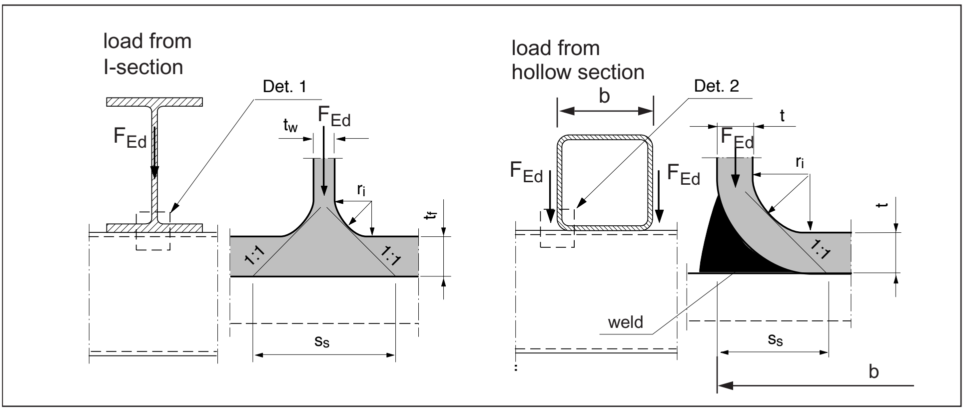


Figure 2.9 Length of stiff bearing when the concentrated load is applied from a hot rolled section or from a hollow section. Details

The first and fifth case of Figure 2.8 have been presented in more details in Figure 2.9. On the basis of the figures the following formulae can be derived for the length of stiff bearing s_s:

$$s_s = t_w + 2t_f + 2(2-\sqrt{2})r_i \quad \textit{(load from rolled section)} \qquad (2.158)$$

$$s_s = t_w + 2t_f + 2\sqrt{2}a \quad \textit{(load from welded section)} \qquad (2.159)$$

$$s_s = 2t + (2-\sqrt{2})r_i \quad \textit{(load from hollow section, when the corner space has filled with a weld, see Fig. 2.9 Det.2)} \qquad (2.160)$$

where t_w is the thickness of the web of I-section
t_f is the thickness of the flange of I-section
r_i is the internal corner radius of hot rolled I-section or structural hollow section
a is the throat thickness of the web-to-flange weld of welded I-section
t is the wall thickness of the hollow section

2.11.2 Combined effect of concentrated load and bending moment

Because the resistance to concentrated loads has above been determined basing on EN 1993-1-3, also the combined effect related to concentrated load shall be checked basing on the same Part of Eurocode. Therein only the combined effect of concentrated load and simultaneous bending moment needs to be checked.

The combined effect is checked according to the following simplified criteria [11]:

$$\frac{M_{Ed}}{f_y W_{eff}/\gamma_{M0}} \le 1,0 \qquad (2.161)$$

$$\frac{F_{Ed}}{2F_{1w.Rd}} \le 1,0 \qquad (2.162)$$

$$\frac{M_{Ed}}{f_y W_{eff}/\gamma_{M0}} + \frac{F_{Ed}}{2F_{1w.Rd}} \le 1,25 \qquad (2.163)$$

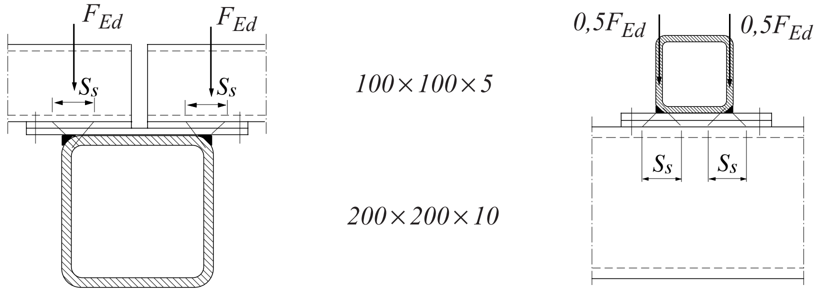
where M_{Ed} is the design value of the bending moment at ultimate limit state
W_{eff} is the section modulus of the effective cross-section (in case of cross-section Classes 1...3 the elastic section modulus W_{el})
f_y is the nominal yield strength of the material
γ_{M0} is the partial safety factor for resistance (Table 2.5)
F_{Ed} is the design value of the concentrated load applied to the hollow section at ultimate limit state
$F_{1w.Rd}$ is the design value of concentrated load resistance per one web according to clause 2.11.1

In expression (2.163) the bending moment M_{Ed} can be calculated at the edge of the support [11].

Example 2.23

Calculate the resistance of hollow sections 200×200×10 and 100×100×5 to transverse concentrated forces, when subjected to load shown in the figure. The thickness of each flange plate is t_p = 10 mm.

The steel grade is SSAB Domex Tube Double Grade, which fulfills the EN 10219 requirements for both steel grades S420MH and S355J2H. Thereby the design calculations may be performed at designer's own choice either according to grade S420 or grade S355. Grade S420 is chosen in this Example as design basis.



r_i/t = 1,0 for hollow section 100×100×5
r_i/t = 1,5 for hollow section 200×200×10

f_y = 420 N/mm^2
γ_{M1} = 1,0

Resistance of hollow section 200×200×10 to transverse concentrated forces:

In regard to the figure on the right, calculate the length of stiff bearing s_s.
First, calculate s_s without the flange plates:

$$s_s = 2t + (2-\sqrt{2})r_i = 2\cdot 5 + (2-\sqrt{2})\cdot 5 = 12{,}9\ mm$$

⇒on the surface of hollow section 200×200×10 the remaining non-loaded length, restricted by the left and right web of hollow section 100×100×5, would be 100 - 2·12,9 = 74,2 mm.

The flange plates (t_p = 10 mm) increase the length of stiff bearing s_s 'outwords and inwards' in proportion to their thickness as follows:

$$\Delta s_{s.in} = \Delta s_{s.out} = t_p + t_p = 10 + 10 = 20\ mm$$

The calculatory length of stiff bearing s_s shall not exceed the existing real length. Therefore it must be checked whether the lengths s_s on the left and on the right do overlap each others at 'inner side':

$$2\cdot(s_s + \Delta s_{s.in}) = 2\cdot(12{,}9 + 20) = 65{,}8 \le b = 100\ mm$$

⇒the lengths s_s do not overlap each others, thus the increase of the length s_s developed by the flange plates can be fully utilised.

The calculatory values for the lengths s_s are thus finally altogether:

$$s_{s.left.tot} = s_{s.right.tot} = s_s + \Delta s_{s.in} + \Delta s_{s.out} = 12,9 + 20 + 20 = 52,9 \ mm \leq h - t = 190 \ mm \quad OK$$

The concentrated loads $0,5F_{Ed}$ are located near to each other, but still separated from each others (because the related lengths s_s do not overlap, as concluded above). Therefore the loads originated from the hollow section $100 \times 100 \times 5$ have to be considered as two different cases: first as two separate loads $0,5F_{Ed}$ each, and then as one combined total load F_{Ed} (see the right-hand figure).

First, consider the loads on the right-hand figure as two separate loads $0,5F_{Ed}$ each:

Load application is herein type 1b (Table 2.26):

$$k = f_y / 228 = 420/228 = 1,842$$

$$k_3 = 1,0$$

$$k_4 = 1,22 - 0,22k = 1,22 - 0,22 \cdot 1,842 = 0,8148$$

$$k_5 = 1,06 - 0,06(r_i/t) = 1,06 - 0,06 \cdot 1,5 = 0,97 \leq 1,0$$

$$s_{s.left.tot}/t = 52,9/10 = 5,29 \leq 60$$

$$C_F = k_3 k_4 k_5 \left[14,7 - \frac{(h-t)/t}{49,5}\right]\left[1 + 0,007\frac{s_{s.left.tot}}{t}\right] =$$

$$1,0 \cdot 0,8148 \cdot 0,97 \cdot \left[14,7 - \frac{(200-10)/10}{49,5}\right] \cdot [1 + 0,007 \cdot 5,29] = 11,73$$

$$F_{1w.Rd} = C_F \cdot \frac{t^2 f_y}{\gamma_{M1}} = 11,73 \cdot \frac{10^2 \cdot 420}{1,0} = 492,7 \ kN$$

$\Rightarrow$ on the right-hand figure, the resistance to concentrated force $0,5F_{Ed}$ is 492,7 kN

$\Rightarrow$ when based on separate concentrated loads, the resistance for the total load F_{Ed} is:

$$F_{Rd} = 2 \cdot 492,7 = 985,4 \ kN$$

Next, consider the concentrated loads on the right-hand figure as one combined load $0,5F_{Ed} + 0,5F_{Ed} = F_{Ed}$:

Herein, the lengths s_s shall be taken as the centre-to-centre distance of the outermost loads (now the flange plates do not give any benefit, because they do not change the centre-to-centre distance of the loads):

$$s_{s.total} = b - 2 \cdot (s_s/2) = 100 - 2 \cdot (12,9/2) = 87,1 \ mm$$

$$s_{s.total}/t = 87,1/10 = 8,71 \leq 60$$

$$C_F = k_3 k_4 k_5 \left[14,7 - \frac{(h-t)/t}{49,5}\right]\left[1 + 0,007\frac{s_{s.total}}{t}\right] =$$

$$1,0 \cdot 0,8148 \cdot 0,97 \cdot \left[14,7 - \frac{(200-10)/10}{49,5}\right] \cdot [1 + 0,007 \cdot 8,71] = 12,00$$

$$F_{1w.Rd} = C_F \cdot \frac{t^2 f_y}{\gamma_{M1}} = 12,00 \cdot \frac{10^2 \cdot 420}{1,0} = 504,0\ kN$$

$\Rightarrow$ *when based on one combined load, the resistance for the total load F_{Ed} is 504,0 kN.*

For hollow section 200×200×10, the critical case is thus the combined load, where the resistance for the total load F_{Ed} is 504,0 kN.

Resistance of hollow section 100×100×5 to transverse concentrated forces:

In regard to the figure on the left, calculate the length of stiff bearing s_s.
First, calculate s_s without the flange plates:

$$s_s = 2t + (2 - \sqrt{2})r_i = 2 \cdot 10 + (2 - \sqrt{2}) \cdot 15 = 28,8\ mm$$

Next, consider also the flange plates. The calculatory length s_s is:

$$s_{s.tot} = s_s + \Delta s_{s.left} + \Delta s_{s.right} = 28,8 + 20 + 20 = 68,8\ mm \le h - t = 95\ mm \quad OK$$

When looking on the left-hand figure and using the same definitions as given on Table 2.26, it can be concluded even without more precise data that:

$$c \le 1,5(h - t) = 1,5 \cdot (100 - 5) = 142,5\ mm$$

$\Rightarrow$ *load application is herein type 1a. Therefore:*

$$k = f_y / 228 = 420/228 = 1,842$$

$$k_1 = 1,33 - 0,33k = 1,33 - 0,33 \cdot 1,842 = 0,7221$$

$$k_2 = 1,15 - 0,15(r_i/t) = 1,15 - 0,15 \cdot 1,0 = 1,0 \qquad (k_2 \ge 0,5 \quad ja \quad k_2 \le 1,0)$$

$$k_3 = 1,0$$

$$s_{s.tot}/t = 68,8/5 = 13,76 \le 60$$

$$C_F = k_1 k_2 k_3 \left[5,92 - \frac{(h-t)/t}{132}\right]\left[1 + 0,01\frac{s_{s.tot}}{t}\right] =$$

$$0,7221 \cdot 1,0 \cdot 1,0 \cdot \left[5,92 - \frac{(100-5)/5}{132}\right] \cdot [1 + 0,01 \cdot 13,76] = 4,745$$

$$F_{1w.Rd} = C_F \cdot \frac{t^2 f_y}{\gamma_{M1}} = 4,745 \cdot \frac{5^2 \cdot 420}{1,0} = 49,8\ kN$$

$\Rightarrow$ *on the left-hand figure, for each hollow section 100×100×5, the resistance for concentrated load F_{Ed} is the sum of the front-side web and back-side web:*

$$F_{Rd} = 2F_{1w.Rd} = 2 \cdot 49,8 = 99,6\ kN$$

For the whole joint, the resistance to transverse concentrated force is thus determined by the hollow section 100×100×5. The maximum allowed value for the concentrated loads acting in this joint is hence F_{Ed} = 99,6 kN.

Comparison S420 vs S355:

In case the design calculations would be performed according to grade S355, the hollow section 200×200×10 would have resistance of 459,0 kN, and the hollow section 100×100×5 would have resistance of 95,2 kN. Increase of the material strength S355 → S420 improves the resistance to transverse concentrated forces in this Example hence by 5...10 % depending on the case.

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3. DESIGN OF HOLLOW SECTION JOINTS

3.1 Design of welded joints in lattice structures

Joints in lattice structures are usually assumed to be nominally pinned, and brace members are designed for axial force only. Depending on the dimensions of the chord versus brace members, the effect of joint stiffness can be accounted for, thereby reducing the buckling length of the brace member (see clause 7.4.3). Transverse loads along the chord span introduce also bending moments, in which case the chord shall be designed for compression and bending. In terms of compression resistance, a hollow section with thin walls is the most favourable solution. However, when considering the resistance of the joint, a thin-walled, wide chord is not as good as a thick-walled narrow chord.

In lattice structures the design conditions shall be checked separately on each member of the joint. The design condition can be presented in general form as follows (written here on purpose in 'backward' order):

$$N_{i.Rd} \geq N_{i.Ed} \tag{3.1}$$

where $N_{i.Rd}$ is the design resistance of brace member i
$N_{i.Ed}$ is the design value of the normal force acting in brace member i at ultimate limit state

In design condition (3.1) it is generally possible to perform the verification by checking the (joint) resistances of brace members alone, because in the joint resistance tables (Annex 11.3) also the resistance of the chord has been determined by the resistance of the brace members (for example the chord face failure by yielding is in the formulae of the table determined as the highest 'allowable' force, i.e. resistance, acting in the brace member).

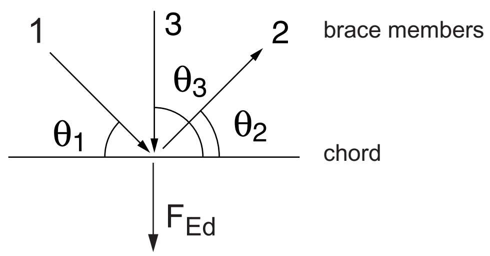


Figure 3.1 Example. KT lattice joint (brace members 1,2,3) which is additionally subject to external concentrated load / support reaction F_{Ed}

The member forces acting in lattice joints are normally calculated assuming the ends of the brace members to be pinned. For example in a case like in Figure 3.1 the design condition (3.1) can be thereby written for each of the brace members in the following form:

Brace member 1: $N_{1.Rd} \geq N_{1.Ed} \quad | \cdot \sin\theta_1$ (3.2)

thus:

$$N_{1.Rd}\sin\theta_1 \geq N_{1.Ed}\sin\theta_1 = N_{2.Ed}\sin\theta_2 \text{ ''}+\text{'' } N_{3.Ed}\sin\theta_3 \text{ ''}+\text{'' } F_{Ed}$$

Brace member 2: $N_{2.Rd} \geq N_{2.Ed} \quad | \cdot \sin\theta_2$ (3.3)

thus:

$$N_{2.Rd}\sin\theta_2 \geq N_{2.Ed}\sin\theta_2 = N_{1.Ed}\sin\theta_1 \text{ ''}+\text{'' } N_{3.Ed}\sin\theta_3 \text{ ''}+\text{'' } F_{Ed}$$

Brace member 3: $N_{3.Rd} \geq N_{3.Ed} \quad | \cdot \sin\theta_3$ (3.4)

thus:

$$N_{3.Rd}\sin\theta_3 \geq N_{3.Ed}\sin\theta_3 = N_{1.Ed}\sin\theta_1 \text{ ''}+\text{'' } N_{2.Ed}\sin\theta_2 \text{ ''}+\text{'' } F_{Ed}$$

where $N_{i.Rd}$ is the design resistance of brace member i

$N_{i.Ed}$ is the design value of the normal force acting in brace member i at ultimate limit state

F_{Ed} is the design value of the possible external concentrated load / support reaction at ultimate limit state (if such load does not exist, set $F_{Ed} = 0$). Also regarding the chord and the structural member transferring the load F_{Ed} to the chord, the corresponding design condition $F_{Rd} \geq F_{Ed}$ shall be checked, where the resistance of the joint F_{Rd} is calculated using the applicable resistance table in Annex 11.3 (usually X joint, presented in Tables 11.3.2 and 11.3.4, or plate-to-chord joint, presented in Table 11.3.15)

"+" means "to be combined with" (i.e. simultaneous action of the forces) (signs of the forces are determined case by case)

When checking the resistances of the lattice joints it shall be kept in mind, that the resistances presented in the resistance tables (Annex 11.3) apply only to the joint resistances of the members. **Regarding the members connecting to the joint (brace members and chords) it shall always be remembered to check in addition also:**

- resistance of the cross-section (Chapter 2) and
- buckling resistance of the compressed members (Chapter 2) and
- resistance of the welds (clause 3.3).

The designations used for the members in lattice joints are presented in Figure 3.2. The subscripts for members are defined as follows:

0 = chord
1 = compression brace member
2 = tension brace member
3 = middle brace member (in case of three brace members in the joint)
i = overlapping brace member (in case of overlap joint)
j = overlapped brace member (in case of overlap joint)

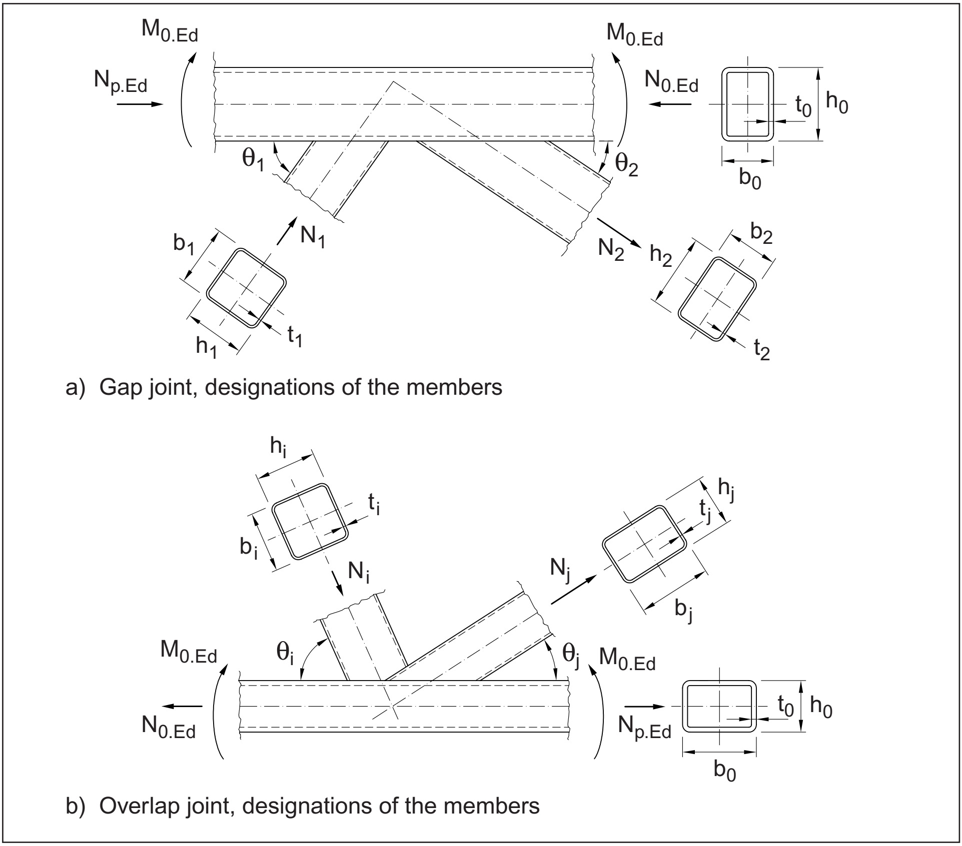


Figure 3.2 Designations of the members in the joint [4,5,6]

Additional essential parameters regarding calculation of the joint are presented in Figures 3.3 and 3.4.

Different joint types of uniplanar trusses are collected in Table 3.2. Joint types of multiplanar trusses are presented in [12] and [13].

Different failure modes appearing in hollow section lattice joints are presented in Table 3.3. The critical failure mode depends on the dimensions of the chord and brace members, and on the joint geometry.

The calculation formulae of lattice joints presented in EN 1993-1-8 are partly based on test results. In this handbook, calculation formulae for different hollow section lattice joints are presented in Annex 11.3. In each of its tables the critical failure modes and resistances for the considered joint type are presented. The resistance of the joint shall be calculated for each failure mode presented in the table, whereafter the governing (lowest) value is adopted as the final resistance of the joint. When using the tables compression is positive. In design of joints, the maximum absolute value of axial force (or correspondingly the stress) acting in the chord member is denoted as $N_{0.Ed}$. When the horizontal components of the forces acting in brace members are excluded, the axial force of the chord member is denoted as $N_{p.Ed}$ (see Figure 3.2).

The resistance of the weld between a brace member and a chord is not taken into account in the joint resistance tables, as it is supposed that the resistance of the welds is designed to be sufficient. The design of welds is presented later on in clause 3.3.

The resistances calculated using the formulae of the tables are valid up to grade S355. When using grades of higher strength, the resistances shall be multiplied by a correction factor according to Table 3.1 [4,5,6].

Table 3.1 Correction factor for resistances of hollow section lattice joints in different steel grades [4,5,6]

Grade	Correction factor for resistance
S235...S355	1,0
S420...S460	0,9

Table 3.2 Joint types in lattice structures

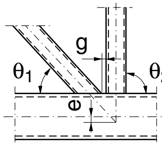
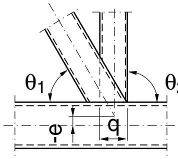
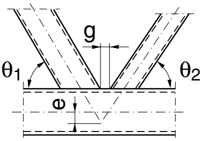
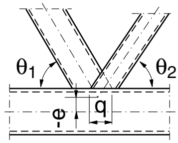
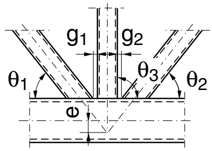
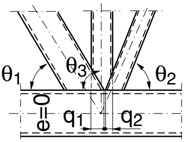
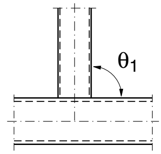
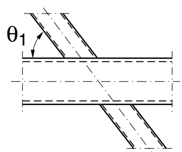
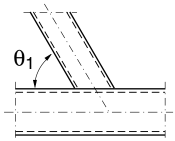
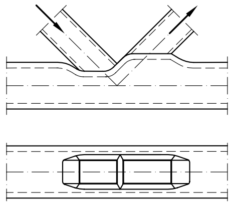
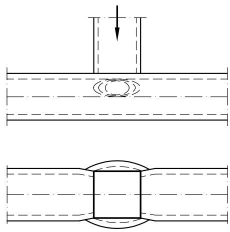
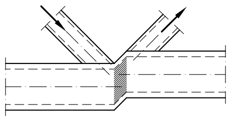
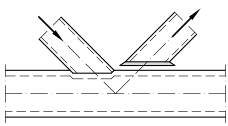
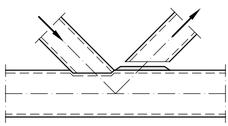
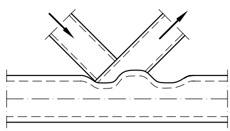
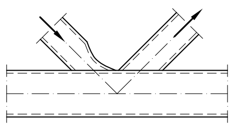
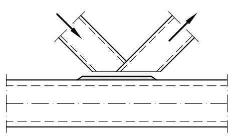
Joint type	Gap joint	Overlap joint
N		
K		
KT		
T		
X		
Y		

Table 3.3 Failure modes of joints in lattice structures [4,5,6]

Failure mode		Structure in which the failure mode is possible
Plastic failure of the chord face or failure of the whole cross-section by plastification		Thin-walled chord and narrow brace member versus chord
Failure of the chord side wall by yielding or by local buckling due to compression of the brace member		High and thin-walled chord of equal width as the brace member
Chord shear failure		Low and thin-walled chord
Punching shear failure of the chord face		Thin-walled and wide chord, brace member slightly narrower than chord
Failure of the brace member or weld		Thin-walled brace member and thick-walled chord
Local buckling of brace member or chord member	 	Chord: Thin-walled and wide Brace member: Thin-walled and wide sides
Shear of the brace members off the chord		Overlap joint, when extent of overlapping is $\lambda_{ov} > \lambda_{ov.lim}$ or when brace members have $h_1 < b_1$ or $h_2 < b_2$

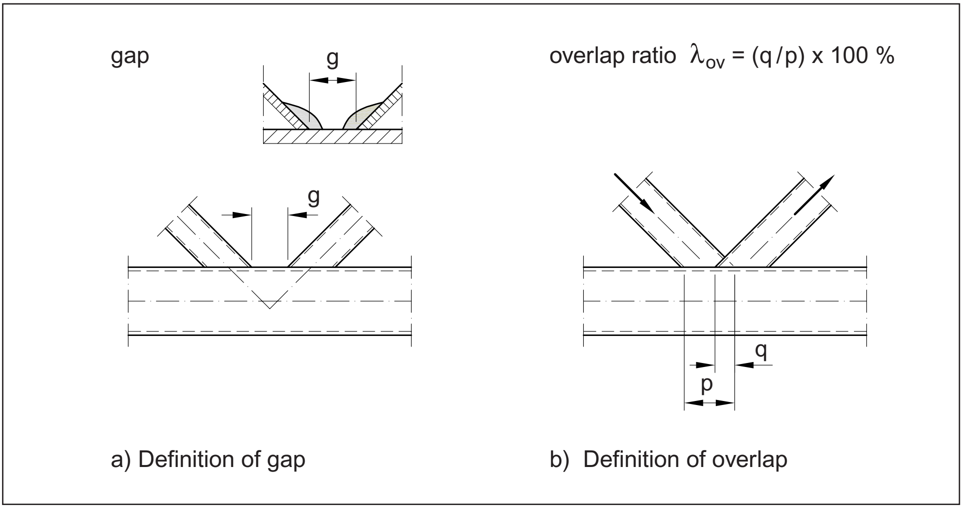


Figure 3.3 Gap and overlap in a joint [4,5,6]

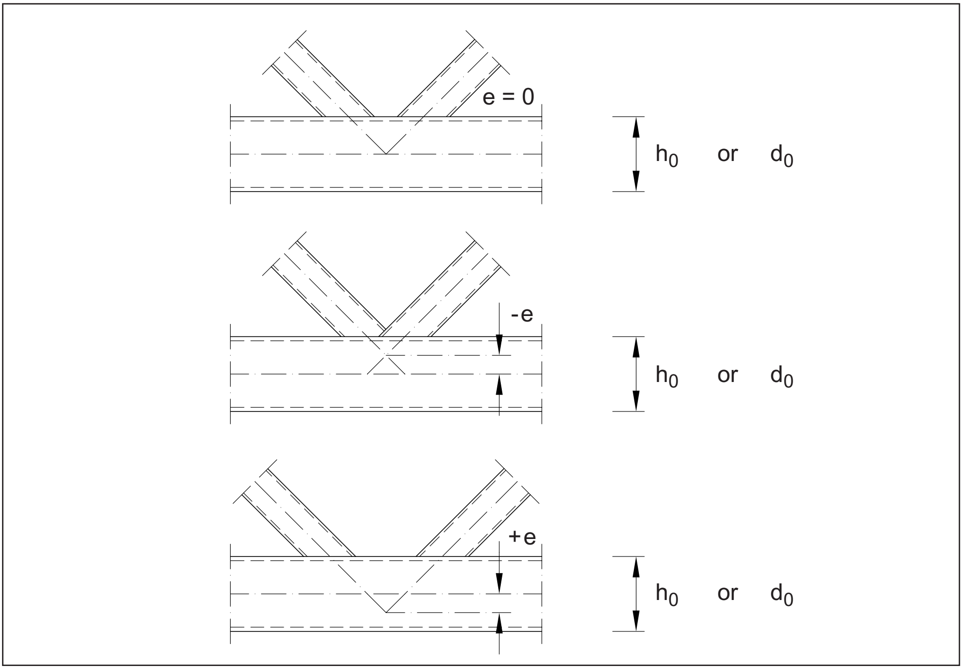


Figure 3.4 Eccentricity of the joint [4,5,6]

According to Figure 3.4 eccentricity obtains a positive value, when the neutral axes of the brace members intersect on the opposite side from the chord's centroidal axis. Correspondingly the eccentricity is negative, when the intersection point is located on the brace member's side from the chord's centroidal axis. The joint's gap refers to the space between the brace members. The joint is overlapped, when the brace members are partially or completely overlaid by each other. The overlap can also be expressed as a gap having a negative value. Eccentricity and gap are interrelated with each other as follows [13]:

$$g = \left(e + \frac{h_0}{2}\right)\frac{\sin(\theta_1 + \theta_2)}{\sin\theta_1 \sin\theta_2} - \frac{h_1}{2\sin\theta_1} - \frac{h_2}{2\sin\theta_2} \tag{3.5}$$

$$e = \left(\frac{h_1}{2\sin\theta_1} + \frac{h_2}{2\sin\theta_2} + g\right)\frac{\sin\theta_1 \sin\theta_2}{\sin(\theta_1 + \theta_2)} - \frac{h_0}{2} \tag{3.6}$$

where θ_i is the smaller angle between the brace member and the chord
h_i is the depth of the brace member
h_0 is the depth of the chord

Moments caused by the eccentricity of the joint need not be taken into account in the calculation of the joint's resistance, if the system length L of the brace member and chord as well as the eccentricity of the joint satisfy the following conditions [4,5,6]:

$$L \geq 6h \tag{3.7}$$

$$-0,55h_0 \leq e \leq 0,25h_0 \tag{3.8}$$

where h is the depth of the cross-section of the member to be considered (brace member or chord) and h_0 is the depth of the cross-section of the chord in the plane of the lattice. In case of circular hollow sections, in the aforementioned expressions the depth h of the cross-section is substituded by the diameter d of the hollow section.

Also in the design of members, the moments caused by joint eccentricity need not be taken into account regarding brace members and tension chord, if both of the aforementioned conditions are satisfied. However, in case of compression chord the moments caused by joint eccentricity shall always be taken into account. In such case the moments caused by eccentricity are distributed between the compression chord members on each side of the joint on the basis of their relative stiffness coefficients I/L, where L is the system length of the member [4,5,6].

If either one of the conditions (3.7) or (3.8) is not met, the moments caused by eccentricity shall be taken into account in the design of both the joint and its members. In such case the moments caused by eccentricity are distributed between all members of the joint on the basis of their relative stiffness coefficients I/L [4,5,6].

The secondary moments caused by the rotational stiffness of the joints can be ignored in the design of members and joints, if the length of the member satisfies condition (3.7) and if the geometry of the joint conforms to the conditions presented in the resistance tables [4,5,6].

Summary of the cases where the moments shall be taken into account is presented in Table 3.4.

Table 3.4 Allowance for bending moments [4,5,6]

<table>
<tr><th rowspan="2">Element or joint to be considered</th><th colspan="3">Source of the bending moment</th></tr>
<tr><th>Secondary effects</th><th>Transverse loading</th><th>Eccentricity</th></tr>
<tr><td>Compression chord</td><td rowspan="4">No, if the condition (3.7) is met, and if geometry of the joint conforms to the conditions presented in the resistance tables</td><td rowspan="4">Yes</td><td>Yes</td></tr>
<tr><td>Tension chord</td><td rowspan="3">No, if the condition (3.7) and (3.8) are met, and if geometry of the joint conforms to the conditions presented in the resistance tables</td></tr>
<tr><td>Brace member</td></tr>
<tr><td>Joint</td></tr>
</table>

Table 3.5 General requirements for hollow section lattice joints [4,5,6]

Members to be joined
• the cold-formed structural hollow sections are circular or square or rectangular structural hollow sections conforming to EN 10219
• the joints may be constructed also by using aforementioned structural hollow sections and open sections together
• wall thickness of the hollow section is at least $t \geq 2{,}5$ mm (Note: SSAB recommends $t \geq 3$ mm in order to ensure fit-up and weldability in assembly of the lattice's components)
• compressed plane elements of the members shall meet the requirements of Class 1 or 2 (determined for uniform compression), if not otherwise specified in the resistance table of that specific joint
Geometry of the joint:
• the angles between chords and brace members, as well as angles between adjacent brace members, shall be at least $\theta_i \geq 30°$
• the ends of the members meeting in the joint are fabricated so that the form of the cross-section of the hollow sections does not change (i.e. the provisions do not cover joints of hollow sections which are flattened at their ends)
Gap joints (see Figure 3.3):
• to ensure sufficient clearance in gap joints to enable proper welds, the gap between brace members shall be at least $g \geq (t_1 + t_2)$
Overlap joints (see Figure 3.3):
• when the wall thicknesses and/or steel grades of the members to be overlapped are different, the member having the smallest value of $t_i \cdot f_{yi}$ shall be set uppermost
• when the width of the overlapping brace members is different, the narrower member shall be set on top of the wider member
• in partially overlapping joints, SSAB recommends that also the hidden seam of the brace member shall always be welded to the chord (according to EN 1993-1-8 this need not be done, provided that the axial forces in the brace members are such that their components perpendicular to the chord do not differ by more than 20 %)
• in overlap joints, the overlapping shall be large enough to transfer the shear force from one brace member to another. The overlap shall be at least $\lambda_{ov} \geq 25$ %
• If $\lambda_{ov} > \lambda_{ov.lim}$ <u>or</u> if on rectangular brace members $h_1 < b_1$ and/or $h_2 < b_2$, also the shear of the brace members off the chord (in the direction of the chord) shall be checked: $\lambda_{ov.lim}$ = 60 % if the hidden seam of the overlapped brace member is not welded to the chord $\lambda_{ov.lim}$ = 80 % if the hidden seam of the overlapped brace member is welded to the chord

When using the resistance formulae of lattice joints, it shall be checked that the joint meets the validity conditions presented in the resistance tables. The validity conditions of the tables are assigned both to the geometry of the joint, and to the cross-section Classes and cross-sectional dimensions of the hollow sections forming the joint. The general requirements given in EN 1993-1-8 for joints are presented in Table 3.5. Additional requirements specific for a certain joint type are presented hereafter in clauses 3.1.1 - 3.1.2, and in the tables of Annex 11.3.

3.1.1 Joint of circular, square and rectangular brace members to a square or rectangular chord

Before determining the resistance of the joint, the members shall be designed for the loads which they are subjected to (Chapter 2). The joints are usually assumed to be pinned, so the brace members are designed for axial force only. The weld between the brace member and chord is normally designed to have equal strength with the brace member (see clause 3.3). Moments caused by eccentricity of the joint need not be taken into account in the calculation of the joint's resistance, if the system length of the members forming the joint and eccentricity of the joint meet the conditions presented in clause 3.1.

The compressed plane elements of the members shall meet the requirements of Class 1 or 2 (determined for uniform compression), if not otherwise specified in the resistance table of that specific joint [4,5,6].

In case of gap joint of type N, K or KT, the gap g (Figure 3.3) shall fulfill the following conditions [4,5,6,13]:

$$g \geq t_1 + t_2 \tag{3.9}$$

$$0,5(1-\beta)b_0 \leq g \leq 1,5(1-\beta)b_0 \tag{3.10}$$

$$\beta = \frac{d_1 + d_2}{2b_0} \quad or \quad \frac{b_1 + b_2 + h_1 + h_2}{4b_0} \qquad \text{for N or K joint} \tag{3.11}$$

$$\beta = \frac{d_1 + d_2 + d_3}{3b_0} \quad or \quad \frac{b_1 + b_2 + b_3 + h_1 + h_2 + h_3}{6b_0} \qquad \text{for KT joint} \tag{3.12}$$

where t_i is the wall thickness of the brace member
d_i is the diameter of the brace member's cross-section
b_i is the width of the brace member's cross-section
h_i is the depth of the brace member's cross-section
b_0 is the width of the chord's cross-section

If the gap is larger than the limit values above, the joint shall be treated as two separate joints [4,5,6].

The chord's normal force and bending moment have an impact on the resistance of the chord face. This interaction is considered by parameter n [4,5,6]:

$$n = \frac{\sigma_{0.Ed}}{f_{y0}/\gamma_{M5}} = \frac{N_{0.Ed}}{A_0 f_{y0}/\gamma_{M5}} + \frac{M_{0.Ed}}{W_{el.0} f_{y0}/\gamma_{M5}} \tag{3.13}$$

where $N_{0.Ed}$ is the maximum absolute value of the normal force acting in the chord (Figure 3.2)

$M_{0.Ed}$ is the bending moment acting in the chord (Figure 3.2)
A_0 is the cross-section area of the chord (Annex 11.1)
$W_{el.0}$ is the elastic bending modulus of the chord (Annex 11.1)
f_{y0} is the nominal yield strength of the chord
γ_{M5} is the partial safety factor for hollow section lattice joints (Table 2.5)

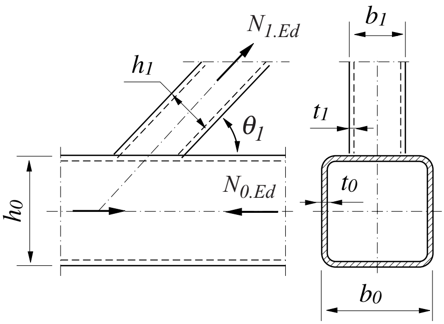
The resistance of joints is calculated in Examples 3.1 - 3.6 according to the tables of Annex 11.3. The resistance of a joint is calculated in the resistance tables for different failure modes, from which the lowest value is adopted as the final resistance of the joint. In the design of hollow section lattice joints, partial safety factor γ_{M5} is applied for the resistance of joints (see Table 2.5), for which the recommended value γ_{M5} = 1,0 has been used as given in EN 1993-1-8.

When using the tables, it shall be checked that the members of the lattice and the geometry of the joint meet the requirements presented in the tables. The validity conditions of the joints are met in Examples 3.1 - 3.6, but the checking of them is not presented in this context.

The steel grade is in all Examples SSAB Domex Tube Double Grade, which fulfills the EN 10219 requirements for both steel grades S420MH and S355J2H. Thereby the design calculations may be performed at designer's own choice either according to grade S420 or grade S355. In the following Examples the design calculations have been carried out according to grade S420, unless otherwise specified.

In the tables of Annex 11.3 and in Examples 3.1 - 3.6, it has been supposed that the value of partial safety factor γ_{M5} for hollow section lattice joints is γ_{M5} = 1,0 according to the recommended value in Eurocode and Finnish National Annex. The values valid in other countries must be checked from the National Annex of the relevant country. The tables in Annex 11.3 apply to steel grades, for which the yield strength is not higher than f_y = 460 N/mm^2. When using steel grades of yield strength higher than f_y = 355 N/mm^2, <u>the joint resistances</u> shall be multiplied by the correction factor according to Table 3.1.

Example 3.1
Y joint, brace member subjected to tension (Table 11.3.1)



The geometry and forces of the joint are as follows:

Chord: 200×200×8
A_0 = 5924 mm^2 *(Annex 11.1)*

Brace member: 100×100×5
A_1 = 1836 mm^2 *(Annex 11.1)*

$\theta_1 = 45°$
$\beta = b_1/b_0 = 100/200 = 0{,}5$
$\eta = h_1/b_0 = 100/200 = 0{,}5$

$N_{0.Ed}$ = 936 kN *(compression)*
$N_{1.Ed}$ = 590 kN *(tension)*

The normal stress $\sigma_{0.Ed}$ in the chord face has an impact on the joint's resistance by the parameter k_n:

$$n = \frac{\sigma_{0.Ed}}{f_{y0}/\gamma_{M5}} = \frac{N_{0.Ed}}{A_0 f_{y0}/\gamma_{M5}} + \frac{M_{0.Ed}}{W_{el.0} f_{y0}/\gamma_{M5}} = \frac{936 \cdot 10^3}{5924 \cdot 420/1,0} + 0 = 0,3762$$

$$k_n = 1,3 - \frac{0,4|n|}{\beta} = 1,3 - \frac{0,4 \cdot 0,3762}{0,5} = 0,9990 \leq 1,0 \qquad \text{(chord in compression)}$$

Chord face failure by yielding:
Since $\beta = 0,5 \leq 0,85$, check the resistance to chord face failure:

$$N_{1.Rd} = 0,9 \cdot \frac{k_n f_{y0} t_0^2}{(1-\beta)\sin\theta_1}\left(\frac{2\eta}{\sin\theta_1} + 4\sqrt{1-\beta}\right)/\gamma_{M5} \qquad \text{(S420: resistance factor = 0,9)}$$

$$= 0,9 \cdot \frac{0,9990 \cdot 420 \cdot 8^2}{(1-0,5)\sin 45}\left(\frac{2 \cdot 0,5}{\sin 45} + 4\sqrt{1-0,5}\right)/1,0 = 290,0 \text{ kN} \leq N_{1.Ed} = 590 \text{ kN}$$

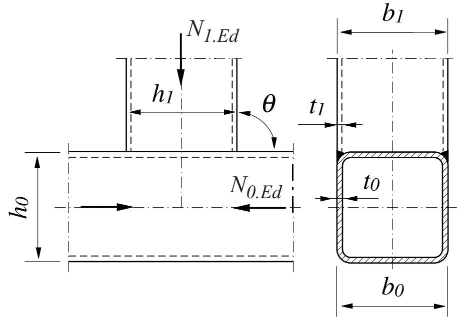
Joint's resistance:
The joint's resistance is hereby the joint resistance of the brace member 1 (i.e. i = 1) being $N_{1.Rd}$ = 290,0 kN, which is substantially lower than the brace member's normal force $N_{1.Ed}$ = 590 kN. Hence, in order to achieve sufficient resistance for the joint, a bigger hollow section has to be chosen for the brace member, or the chord face needs to be reinforced.

In addition to the preceding verification of the joint's resistance, also the following needs to be specifically checked:
- *the welds of the joint, and*
- *the cross-section resistance of the brace member and the chord, and*
- *the buckling resistance of the compressed member (chord).*

Comparison S420 vs S355:
In case the design calculations would be performed according to grade S355, the joint's resistance would be 257,3 kN. Hence, increase of the material strength S355 → S420 improves the joint's resistance in this Example by approx. 13 %.

Example 3.2
T joint, brace member subjected to compression (Table 11.3.1).

The geometry and forces of the joint are as follows:



Chord: 100×100×6
A_0 = 2163 mm^2 (Annex 11.1)

Brace member: 100×100×5
A_1 = 1836 mm^2 (Annex 11.1)

θ_1 = 90°
$\beta = b_1/b_0$ = 100/100 = 1,0

$N_{0.Ed}$ = 400 kN (compression)
$N_{1.Ed}$ = 350 kN (compression)

$$n = \frac{\sigma_{0.Ed}}{f_{y0}/\gamma_{M5}} = \frac{N_{0.Ed}}{A_0 f_{y0}/\gamma_{M5}} + \frac{M_{0.Ed}}{W_{el.0} f_{y0}/\gamma_{M5}} = \frac{400 \cdot 10^3}{2163 \cdot 420/1,0} + 0 = 0,4403$$

$$k_n = 1,3 - \frac{0,4|n|}{\beta} = 1,3 - \frac{0,4 \cdot 0,4403}{1,0} = 1,124 > 1,0 \qquad \Rightarrow k_n = 1,0$$

Chord side wall buckling or yielding:
Because $\beta = 1,0$ chord side wall buckling / yielding may be the critical failure mode. First, calculate buckling stress by using buckling curve c:

$$\bar{\lambda} = 3,46 \cdot \frac{\left(\frac{h_0}{t_0} - 2\right) \cdot \sqrt{\frac{1}{\sin\theta_1}}}{\pi \cdot \sqrt{\frac{E}{f_{y0}}}} = 3,46 \cdot \frac{\left(\frac{100}{6} - 2\right) \cdot \sqrt{\frac{1}{\sin 90}}}{\pi \cdot \sqrt{\frac{2,1 \cdot 10^5}{420}}} = 0,7224$$

$$\Phi = 0,5 \cdot [1 + \alpha(\bar{\lambda} - 0,2) + \bar{\lambda}^2] = 0,5 \cdot [1 + 0,49 \cdot (0,7224 - 0,2) + 0,7224^2] = 0,8889$$

$$\chi = \frac{1}{\Phi + \sqrt{\Phi^2 - \bar{\lambda}^2}} = \frac{1}{0,8889 + \sqrt{0,8889^2 - 0,7224^2}} = 0,7108 \leq 1,0$$

For T joint:

$$f_b = \chi \cdot f_{y0} = 0,7108 \cdot 420 = 298,5 \text{ N/mm}^2$$

Now we can calculate the resistance in respect to the chord side wall:

$$N_{1.Rd} = 0,9 \cdot \frac{k_n f_b t_0}{\sin\theta_1}\left(\frac{2h_1}{\sin\theta_1} + 10t_0\right)/\gamma_{M5} \qquad \text{(S420: resistance factor = 0,9)}$$

$$= 0,9 \cdot \frac{1,0 \cdot 298,5 \cdot 6}{\sin 90}\left(\frac{2 \cdot 100}{\sin 90} + 10 \cdot 6\right)/1,0 = 419,1 \text{ kN}$$

Brace member failure by yielding:
The effective width of the brace member:

$$b_{eff} = \frac{10}{b_0/t_0} \cdot \frac{f_{y0} \cdot t_0}{f_{y1} \cdot t_1} \cdot b_1 = \frac{10}{100/6} \cdot \frac{420 \cdot 6}{420 \cdot 5} \cdot 100 = 72 \text{ mm} \leq b_1 = 100 \text{ mm}$$

$$N_{1.Rd} = 0,9 \cdot f_{y1} t_1 (2h_1 - 4t_1 + 2b_{eff})/\gamma_{M5} \qquad \text{(S420: resistance factor = 0,9)}$$

$$= 0,9 \cdot 420 \cdot 5 \cdot (2 \cdot 100 - 4 \cdot 5 + 2 \cdot 72)/1,0 = 612,4 \text{ kN}$$

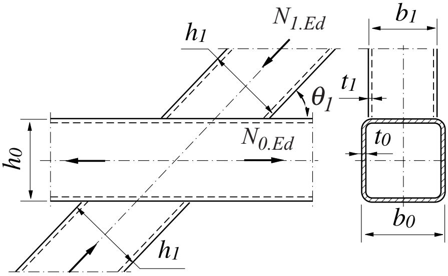
Joint's resistance:
The joint's resistance is hereby governed by the chord's side wall resistance, which is determined by the resistance of the brace member 1 (i.e. i = 1) being
$N_{1.Rd} = 419,1 \text{ kN} \geq N_{1.Ed}$ *OK*

In addition to the preceding verification of the joint's resistance, also the following needs to be specifically checked:
- *the welds of the joint, and*
- *the cross-section resistance of the brace member and the chord, and*
- *the buckling resistance of the compressed members (brace member and chord).*

Comparison S420 vs S355:
In case the design calculations would be performed according to grade S355, the joint's resistance would be 413,6 kN. Hence, increase of the material strength S355 → S420 does not improve the joint's resistance very much in this Example.

Example 3.3
X joint, brace members subjected to compression (Table 11.3.1)

The geometry and forces of the joint are as follows:



Chord: 200×200×8
$A_0 = 5924\ mm^2$ *(Annex 11.1)*

Brace members: 180×180×6
$A_1 = 4083\ mm^2$ *(Annex 11.1)*

$\theta_1 = 30°$
$\beta = b_1/b_0 = 180/200 = 0{,}90$
$\eta = h_1/b_0 = 180/200 = 0{,}90$
$\gamma = 0{,}5b_0/t_0 = 0{,}5 \cdot 200/8 = 12{,}5$

$N_{0.Ed} = 620\ kN$ *(tension)* $\Rightarrow k_n = 1{,}0$
$N_{1.Ed} = 1000\ kN$ *(compression)*

Chord face punching shear:
Since $0{,}85 \le \beta \le 1\text{-}(1/\gamma) = 0{,}92$, *chord face punching shear must be checked:*

$$b_{e.p} = \frac{10}{b_0/t_0} \cdot b_1 = \frac{10}{200/8} \cdot 180 = 72\ mm \le b_1 = 180\ mm$$

$$N_{1.Rd} = 0{,}9 \cdot \frac{f_{y0} \cdot t_0}{\sqrt{3}\sin\theta_1} \cdot \left(\frac{2h_1}{\sin\theta_1} + 2b_{e.p}\right)/\gamma_{M5} \qquad \text{(S420: resistance factor = 0,9)}$$

$$= 0{,}9 \cdot \frac{420 \cdot 8}{\sqrt{3}\sin 30} \cdot \left(\frac{2 \cdot 180}{\sin 30} + 2 \cdot 72\right)/1{,}0 = 3017\ kN$$

Chord face yielding and chord side wall buckling / yielding:
Since $0{,}85 < \beta < 1{,}0$, *chord's resistance must be checked in respect to chord face and chord side wall. First, calculate joint's resistance in respect to chord face when* $\beta = 0{,}85$, *and in respect to chord side wall when* $\beta = 1{,}0$. *After that, apply linear interpolation with the actual value* $\beta = 0{,}9$:

a) $\beta = 0{,}85$:

$$N_{1.Rd} = 0{,}9 \cdot \frac{k_n f_{y0} t_0^2}{(1-\beta)\sin\theta_1} \cdot \left(\frac{2\eta}{\sin\theta_1} + 4\sqrt{1-\beta}\right) / \gamma_{M5} \qquad \text{(S420: resistance factor = 0,9)}$$

$$= 0{,}9 \cdot \frac{1{,}0 \cdot 420 \cdot 8^2}{(1-0{,}85)\sin 30} \cdot \left(\frac{2 \cdot 0{,}9}{\sin 30} + 4\sqrt{1-0{,}85}\right) / 1{,}0 = 1661\ kN$$

b) $\beta = 1{,}0$:

$$\bar{\lambda} = 3{,}46 \cdot \frac{\left(\frac{h_0}{t_0} - 2\right) \cdot \sqrt{\frac{1}{\sin\theta_1}}}{\pi \cdot \sqrt{\frac{E}{f_{y0}}}} = 3{,}46 \cdot \frac{\left(\frac{200}{8} - 2\right) \cdot \sqrt{\frac{1}{\sin 30}}}{\pi \cdot \sqrt{\frac{2{,}1 \cdot 10^5}{420}}} = 1{,}602$$

$$\Phi = 0{,}5 \cdot [1 + \alpha(\bar{\lambda} - 0{,}2) + \bar{\lambda}^2] = 0{,}5 \cdot [1 + 0{,}49 \cdot (1{,}602 - 0{,}2) + 1{,}602^2] = 2{,}127$$

$$\chi = \frac{1}{\Phi + \sqrt{\Phi^2 - \bar{\lambda}^2}} = \frac{1}{2{,}127 + \sqrt{2{,}127^2 - 1{,}602^2}} = 0{,}2836 \le 1{,}0$$

For X joint:

$$f_b = 0{,}8\chi \cdot f_{y0} \cdot \sin\theta_1 = 0{,}8 \cdot 0{,}2836 \cdot 420 \cdot \sin 30 = 47{,}6\ N/mm^2$$

$$N_{1.Rd} = 0{,}9 \cdot \frac{k_n f_b t_0}{\sin\theta_1} \cdot \left(\frac{2h_1}{\sin\theta_1} + 10t_0\right) / \gamma_{M5} \qquad \text{(S420: resistance factor = 0,9)}$$

$$= 0{,}9 \cdot \frac{1{,}0 \cdot 47{,}6 \cdot 8}{\sin 30} \cdot \left(\frac{2 \cdot 180}{\sin 30} + 10 \cdot 8\right) / 1{,}0 = 548{,}4\ kN$$

When $\beta = 1{,}0$*, check whether the above calculated resistance 548,4 kN can be directly adopted, or must the chord shear be checked, too:*

$$\cos\theta_1 = \cos 30 = 0{,}8660 \le h_1/h_0 = 180/200 = 0{,}9$$

$\Rightarrow$ *chord shear does not need to be checked*

Next, interpolate the chord's resistance from the above calculated results:

$$N_{1.Rd} = 1661 + \frac{548{,}4 - 1661}{1{,}0 - 0{,}85} \cdot (0{,}9 - 0{,}85) = 1290\ kN$$

Brace member failure by yielding:

The effective width of the brace member is:

$$b_{eff} = \frac{10}{b_0/t_0} \cdot \frac{f_{y0} \cdot t_0}{f_{y1} \cdot t_1} \cdot b_1 = \frac{10}{200/8} \cdot \frac{420 \cdot 8}{420 \cdot 6} \cdot 180 = 96{,}0\ mm \le b_1 = 180\ mm$$

$$N_{1.Rd} = 0{,}9 \cdot f_{y0} t_1 (2h_1 - 4t_1 + 2b_{eff}) / \gamma_{M5} \qquad \text{(S420: resistance factor = 0,9)}$$

$$= 0{,}9 \cdot 420 \cdot 6 \cdot (2 \cdot 180 - 4 \cdot 6 + 2 \cdot 96{,}0) / 1{,}0 = 1198\ kN$$

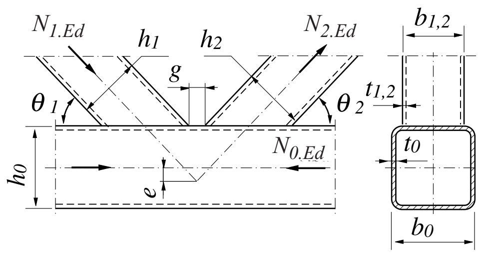
Joint's resistance:
The joint's resistance is the smallest of the above obtained results, i.e.
$N_{1.Rd} = 1198\ kN \geq N_{1.Ed}$ *OK*

In addition to the preceding verification of the joint's resistance, also the following needs to be specifically checked:
- *the welds of the joint, and*
- *the cross-section resistance of the brace members and the chord, and*
- *the buckling resistance of the compressed members (brace members and chord).*

Comparison S420 vs S355:
In case the design calculations would be performed according to grade S355, the joint's resistance would be 1125 kN. Hence, increase of the material strength S355 → S420 improves the joint's resistance in this Example by approx. 6,5 %.

Example 3.4
Gap K joint (Table 11.3.2)

The geometry and forces of the joint are as follows:



Chord: 200×200×8
$A_0 = 5924\ mm^2$ *(Annex 11.1)*

Brace members: 150×150×6
$A_1 = A_2 = 3363\ mm^2$ *(Annex 11.1)*

$\theta_1 = \theta_2 = 45°$
$g = 28\ mm$

$N_{0.Ed} = 1364\ kN$ *(compression)*
$N_{1.Ed} = 600\ kN$ *(compression)*
$N_{2.Rd} = 600\ kN$ *(tension)*

$$\beta = \frac{b_1 + b_2 + h_1 + h_2}{4b_0} = \frac{150 + 150 + 150 + 150}{4 \cdot 200} = 0,75$$

$$\gamma = \frac{b_0}{2t_0} = \frac{200}{2 \cdot 8} = 12,5$$

$$n = \frac{\sigma_{0.Ed}}{f_{y0}/\gamma_{M5}} = \frac{N_{0.Ed}}{A_0 f_{y0}/\gamma_{M5}} + \frac{M_{0.Ed}}{W_{el.0} f_{y0}/\gamma_{M5}} = \frac{1364 \cdot 10^3}{5924 \cdot 420/1,0} + 0 = 0,5482$$

$$k_n = 1,3 - \frac{0,4|n|}{\beta} = 1,3 - \frac{0,4 \cdot 0,5482}{0,75} = 1,008 > 1,0 \quad \Rightarrow k_n = 1,0$$

Check the validity conditions of the joint's gap:

$$g = 28\ mm \geq t_1 + t_2 = 6 + 6 = 12\ mm \qquad OK$$

$$g/b_0 = 28/200 = 0,14 \geq 0,5(1-\beta) = 0,5 \cdot (1-0,75) = 0,125 \qquad OK$$

$$g/b_0 = 0,14 \leq 1,5(1-\beta) = 1,5 \cdot (1-0,75) = 0,375 \qquad OK$$

$\Rightarrow$ *the joint's gap meets the validity conditions*

Check the validity conditions of the joint's eccentricity:

$$e = \left(\frac{h_1}{2\sin\theta_1} + \frac{h_2}{2\sin\theta_2} + g\right)\frac{\sin\theta_1 \sin\theta_2}{\sin(\theta_1+\theta_2)} - \frac{h_0}{2}$$

$$= \left(\frac{150}{2\sin 45} + \frac{150}{2\sin 45} + 28\right)\frac{\sin 45 \sin 45}{\sin(45+45)} - \frac{200}{2} = 20,1\ mm$$

$$-0,55h_0 = -110\ mm \leq e = 20,1\ mm \leq 0,25h_0 = 50\ mm \quad OK$$

$\Rightarrow$ *the joint's eccentricity meets the validity conditions*

When calculating the resistance of the joint, only the brace member 1 needs to be checked, because both brace members are equal in respect to their size, their angle and their loading.

Chord face failure by yielding:

$$N_{1.Rd} = 0,9 \cdot \frac{8,9 \cdot k_n \cdot f_{y0} \cdot t_0^2 \cdot \sqrt{\gamma}}{\sin\theta_1} \cdot \left(\frac{b_1 + b_2 + h_1 + h_2}{4b_0}\right)/\gamma_{M5} \qquad \text{(S420: factor = 0,9)}$$

$$= 0,9 \cdot \frac{8,9 \cdot 1,0 \cdot 420 \cdot 8^2 \cdot \sqrt{12,5}}{\sin 45} \cdot \left(\frac{150+150+150+150}{4 \cdot 200}\right) = 807,4\ kN$$

Chord face punching shear:

Since $\beta \leq 1 - (1/\gamma) = 0,92$, chord face punching shear must be checked:

$$b_{e.p} = \frac{10}{b_0/t_0} \cdot b_1 = \frac{10}{200/8} \cdot 150 = 60\ mm \leq b_1 = 150\ mm$$

$$N_{1.Rd} = 0,9 \cdot \frac{f_{y0}\, t_0}{\sqrt{3}\sin\theta_1}\left(\frac{2h_1}{\sin\theta_1} + b_1 + b_{ep}\right)/\gamma_{M5} \qquad \text{(S420: resistance factor = 0,9)}$$

$$= 0,9 \cdot \frac{420 \cdot 8}{\sqrt{3}\sin 45}\left(\frac{2 \cdot 150}{\sin 45} + 150 + 60\right)/1,0 = 1566\ kN$$

Chord shear:
The shear resistance of the chord at the location of the joint is:

$$\alpha = \sqrt{\frac{1}{1+\frac{4g^2}{3t_0^2}}} = \sqrt{\frac{1}{1+\frac{4\cdot 28^2}{3\cdot 8^2}}} = 0,2402$$

$$A_{v0} = (2h_0 + \alpha b_0)t_0 = (2\cdot 200 + 0,2402\cdot 200)\cdot 8 = 3584\ mm^2$$

$$N_{1.Rd} = 0,9\cdot \frac{f_{y0}\cdot A_{v0}}{\sqrt{3}\sin\theta_1}/\gamma_{M5} \qquad \text{(S420: resistance factor = 0,9)}$$

$$= 0,9\cdot \frac{420\cdot 3584}{\sqrt{3}\sin 45}/1,0 = 1106\ kN$$

Chord's resistance to normal force at the location of the joint's gap:

$$N_{0.gap.Ed} = N_{0.Ed} - N_{2.Ed}\cos\theta_2 = 1364 - 600\cdot\cos 45 = 939,7\ kN \qquad \text{(compression)}$$

$$V_{0.gap.Ed} = N_{1.Ed}\sin\theta_1 = 600\cdot\sin 45 = 424,3\ kN$$

$$A_{V0} = A_0\cdot\frac{h_0}{b_0+h_0} = 5924\cdot\frac{200}{200+200} = 2962\ mm^2$$

$$V_{pl.Rd} = A_{V0}\cdot\frac{f_{y0}/\sqrt{3}}{\gamma_{M0}} = 2962\cdot\frac{420/\sqrt{3}}{1,0} = 718,2\ kN$$

$$N_{0.gap.Rd} = 0,9\cdot[(A_0 - A_{v0})f_{y0} + A_{v0}f_{y0}\sqrt{1-(V_{0.gap.Ed}/V_{pl.Rd})^2}]/\gamma_{M5} \qquad \text{(S420 = 0,9)}$$

$$= 0,9\cdot[(5924-3584)\cdot 420 + 3584\cdot 420\cdot\sqrt{1-(424,3/718,2)^2}]/1,0 = 1978\ kN$$

$$N_{0.gap.Rd} = 1978\ kN \geq N_{0.gap.Ed} = 939,7\ kN \qquad OK$$

Brace member failure by yielding:
The effective width of the brace member is:

$$b_{eff} = \frac{10}{b_0/t_0}\cdot\frac{f_{y0}\cdot t_0}{f_{y1}\cdot t_1}\cdot b_1 = \frac{10}{200/8}\cdot\frac{420\cdot 8}{420\cdot 6}\cdot 150 = 80\ mm \leq b_1 = 150\ mm$$

$$N_{1.Rd} = 0,9\cdot f_{y1}t_1(2h_1 - 4t_1 + b_1 + b_{eff})/\gamma_{M5} \qquad \text{(S420: resistance factor = 0,9)}$$

$$= 0,9\cdot 420\cdot 6\cdot(2\cdot 150 - 4\cdot 6 + 150 + 80)/1,0 = 1148\ kN$$

Joint's resistance:
In respect to the forces acting in the chord, the joint's resistance is governed by the chord's resistance at the gap $N_{0.gap.Rd} = 1978\ kN \geq N_{0.gap.Ed} = 939,7\ kN$ *OK*

In respect to the forces acting in the brace members, the joint's resistance is governed by chord face failure by yielding, where the resistances of the brace members are:
$N_{1.Rd} = 807,4\ kN \geq N_{1.Ed}$ *OK*
$N_{2.Rd} = 807,4\ kN \geq N_{2.Ed}$ *OK*

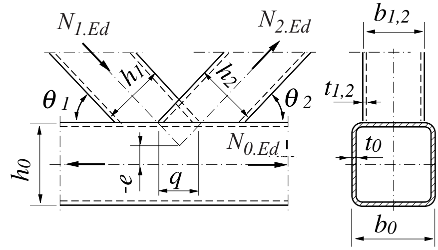
In addition to the preceding verification of the joint's resistance, also the following needs to be specifically checked:

- *the welds of the joint, and*
- *the cross-section resistance of the brace members and the chord, and*
- *the buckling resistance of the compressed members (chord and brace member 1).*

Comparison S420 vs S355:
In case the design calculations would be performed according to grade S355, the joint's resistance would be 1741 kN. Hence, increase of the material strength S355 → S420 improves the joint's resistance in this Example by almost 6,5 % in respect to the brace members, and almost 14 % in respect to the chord.

Example 3.5
Overlap K joint (Table 11.3.3)

The geometry and forces of the joint are as follows:



Chord: 180×180×8
$A_0 = 5284\ mm^2$ *(Annex 11.1)*

Brace members: 140×140×6
$A_1 = A_2 = 3123\ mm^2$ *(Annex 11.1)*

$\theta_1 = \theta_2 = 60°$
$q = -g = 90\ mm$ *(overlap)*

$N_{0.Ed} = 1500\ kN$ *(tension)*
$N_{1.Ed} = 800\ kN$ *(compression)*
$N_{2.Rd} = 800\ kN$ *(tension)*

Check the validity conditions of the joint's eccentricity:

$$e = \left(\frac{h_1}{2\sin\theta_1} + \frac{h_2}{2\sin\theta_2} + g\right)\frac{\sin\theta_1 \sin\theta_2}{\sin(\theta_1+\theta_2)} - \frac{h_0}{2}$$

$$= \left(\frac{140}{2\sin 60} + \frac{140}{2\sin 60} + (-90)\right)\frac{\sin 60 \sin 60}{\sin(60+60)} - \frac{180}{2} = -27,9\ mm$$

$$-0,55h_0 = -99\ mm \le e = -27,9\ mm \le 0,25h_0 = 45\ mm \quad OK$$

⇒ *the joint's eccentricity meets the validity conditions*

The overlap ratio λ_{ov} *is (Figure 3.3):*

$$p = h_1/\sin\theta_1 = 140/\sin 60 = 161,7\ mm$$

$$\lambda_{ov} = (q/p)\cdot 100\ \% = (90/161,7)\cdot 100\ \% = 55,7\ \% \ge 25\ \% \quad OK$$

Shear of the brace members off the chord:
This failure mode must be checked if one of the following conditions takes place:

$$if\ \lambda_{ov} > \lambda_{ov.lim} = 80\ \%\ \ (when\ the\ hidden\ seam\ is\ welded\ to\ the\ chord)$$

$$if\ h_1 < b_1\ \ or\ \ h_2 < b_2$$

$\Rightarrow$ *now neither one of the above conditions takes place, thus shear of the brace members off the chord will not become critical.*

When the chord is a square or rectangular hollow section, in case of overlap joint, the resistance of the overlapping member (i.e. the member located on top) is always calculated first (see Table 11.3.3):

Brace member 1 (= overlapping brace member):

Brace member failure by yielding:
Now $50\ \% \le \lambda_{ov} < 80\ \%$, *thus the effective width is:*

$$b_{eff} = \frac{10}{b_0/t_0} \cdot \frac{f_{y0} \cdot t_0}{f_{y1} \cdot t_1} \cdot b_1 = \frac{10}{180/8} \cdot \frac{420 \cdot 8}{420 \cdot 6} \cdot 140 = 83\ mm \le b_1 = 140\ mm$$

$$b_{e.ov} = \frac{10}{b_2/t_2} \cdot \frac{f_{y2} \cdot t_2}{f_{y1} \cdot t_1} \cdot b_1 = \frac{10}{140/6} \cdot \frac{420 \cdot 6}{420 \cdot 6} \cdot 140 = 60\ mm \le b_1 = 140\ mm$$

$$N_{1.Rd} = 0,9 \cdot f_{y1} t_1 \cdot (b_{eff} + b_{e.ov} + 2h_1 - 4t_1)/\gamma_{M5} \qquad (S420:\ resistance\ factor = 0,9)$$

$$= 0,9 \cdot 420 \cdot 6 \cdot (83 + 60 + 2 \cdot 140 - 4 \cdot 6)/1,0 = 904,9\ kN \ge N_{1.Ed} \qquad OK$$

Brace member 2 (= overlapped brace member):

The resistance of the overlapped brace member is obtained by setting its utilisation ratio to be the same as the overlapping brace member's:

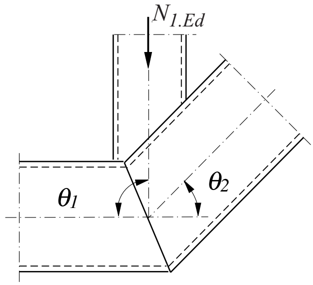
$$N_{2.Rd} = \frac{A_2 \cdot f_{y2}}{A_1 \cdot f_{y1}} \cdot N_{1.Rd} = \frac{3123 \cdot 420}{3123 \cdot 420} \cdot 904,9 = 904,9\ kN \ge N_{2.Ed} \qquad OK$$

In addition to the preceding verification of the joint's resistance, also the following needs to be specifically checked:
- *the welds of the joint, and*
- *the cross-section resistance of the brace members and the chord, and*
- *the buckling resistance of the compressed member (overlapping brace member 1).*

Comparison S420 vs S355:
In case the design calculations would be performed according to grade S355, the joint's resistance would be 849,9 kN. Hence, increase of the material strength S355 $\rightarrow$ *S420 improves the joint's resistance in this Example by approx. 6,5 %.*

Example 3.6
Cranked-chord joint at bottom corner of a lattice structure (Table 11.3.14 => 11.3.3).

The resistance of a cranked-chord joint at bottom corner of a lattice structure can be calculated with formulae of an overlap joint by imagining the bottom chord to continue straight forward without any angle (see lower figure).



Cranked-chord joint at bottom corner of a lattice structure

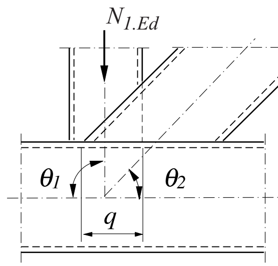
The geometry and forces of the joint are as follows:

Chord: 120×120×6
$A_0 = 2643\ mm^2$ *(Annex 11.1)*

Brace member: 100×100×5
$A_1 = 1836\ mm^2$ *(Annex 11.1)*

$\theta_1 = 90°$
$\theta_2 = 45°$

$N_{1.Ed} = 500\ kN$ *(compression)*
$N_{0.l.Ed} = 500\ kN$ *(tension, left chord)*
$N_{0.r.Ed} = 707\ kN$ *(tension, right chord)*



The calculation model

The joint will be designed to have the joint eccentricity e = 0.

In order to achieve eccentricity e = 0 as wanted, the needed amount of overlap (= negative gap) can be derived from the joint gap formula by setting therein the eccentricity to be e = 0.

Thereby the overlap will be (expression 3.5):

$$q = -\left[\left(e + \frac{h_0}{2}\right)\frac{\sin(\theta_1 + \theta_2)}{\sin\theta_1 \sin\theta_2} - \frac{h_1}{2\sin\theta_1} - \frac{h_2}{2\sin\theta_2}\right]$$

$$= -\left[\left(0 + \frac{120}{2}\right)\frac{\sin(90 + 45)}{\sin 90 \sin 45} - \frac{100}{2\sin 90} - \frac{120}{2\sin 45}\right] = (-)74,9\ mm$$

The overlap ratio λ_{ov} *is (Figure 3.3):*

$$p = h_1 / \sin\theta_1 = 100 / \sin 90 = 100\ mm$$

$$\lambda_{ov} = (q/p) \cdot 100\ \% = (74,9/100) \cdot 100\ \% = 74,9\ \% \geq 25\ \% \quad OK$$

Shear of the brace members off the chord:
This failure mode must be checked if one of the following conditions takes place:

if $\lambda_{ov} > \lambda_{ov.lim} = 80\ \%$ *(when the hidden seam is welded to the chord)*

if $h_1 < b_1$ *or* $h_2 < b_2$

⇒ *now neither one of the above conditions takes place, thus shear of the brace members off the chord will not become critical.*

When the chord is a square or rectangular hollow section, in case of overlap joint, the resistance of the overlapping member (i.e. the member on the top) is always calculated first (see Table 11.3.3):

Brace member 1 (= overlapping brace member):

Brace member failure by yielding:
Now $50\,\% \leq \lambda_{ov} < 80\,\%$, thus the effective width is:

$$b_{eff} = \frac{10}{b_0/t_0} \cdot \frac{f_{y0} \cdot t_0}{f_{y1} \cdot t_1} \cdot b_1 = \frac{10}{120/6} \cdot \frac{420 \cdot 6}{420 \cdot 5} \cdot 100 = 60\ mm \leq b_1 = 100\ mm$$

$$b_{e.ov} = \frac{10}{b_2/t_2} \cdot \frac{f_{y2} \cdot t_2}{f_{y1} \cdot t_1} \cdot b_1 = \frac{10}{120/6} \cdot \frac{420 \cdot 6}{420 \cdot 5} \cdot 100 = 60\ mm \leq b_1 = 100\ mm$$

$$N_{1.Rd} = 0,9 \cdot f_{y1} t_1 \cdot (b_{eff} + b_{e.ov} + 2h_1 - 4t_1)/\gamma_{M5} \qquad \text{(S420: resistance factor = 0,9)}$$

$$= 0,9 \cdot 420 \cdot 5 \cdot (60 + 60 + 2 \cdot 100 - 4 \cdot 5)/1,0 = 567,0\ kN \geq N_{1.Ed} = 500\ kN \quad OK$$

Brace member 2 (= overlapped brace member):

The resistance of the overlapped brace member is obtained by setting its utilisation ratio to be the same as the overlapping brace member's:

$$N_{2.Rd} = \frac{A_2 \cdot f_{y2}}{A_1 \cdot f_{y1}} \cdot N_{1.Rd} = \frac{2643 \cdot 420}{1836 \cdot 420} \cdot 567,0 = 816,2\ kN \geq N_{0.r.Ed} = 700\ kN \quad OK$$

Joint's resistance:
The resistance of the real joint is governed hence by the brace member's resistance:
$N_{1.Rd} = 567{,}0\ kN \geq N_{1.Ed}$ *OK*

In addition to the preceding verification of the joint's resistance, also the following needs to be specifically checked:
- *the welds of the joint, and*
- *the cross-section resistance of the brace member and the chord, and*
- *the buckling resistance of the compressed member (brace member).*

Comparison S420 vs S355:
In case the design calculations would be performed according to grade S355, the joint's resistance would be 532,5 kN. Hence, increase of the material strength S355 → S420 improves the joint's resistance in this Example by approx. 6,5 %.

3.1.2 Joint of circular brace members to a circular chord

The joint of a circular brace member to a circular chord is calculated according to the tables of Annex 11.3. With the lattices fabricated from circular hollow sections, otherwise same calculation principles will be followed as in clause 3.1.1 on square and rectangular hollow sections.

A compressed brace member and chord shall meet the requirements of Class 1 or 2 [4,5,6].

In case of gap N, K or KT joint, the gap g (Figure 3.3) shall satisfy the following condition [4,5,6,13]:

$$g \geq t_1 + t_2 \tag{3.14}$$

where t_i is the wall thickness of the brace member

The chord's normal force and bending moment have an impact on the resistance of the chord face. In case of chords made of circular hollow sections, this interaction is considered by parameter n_p (cf. parameter n in clause 3.1.1):

$$n_p = \frac{\sigma_{p.Ed}}{f_{y0}/\gamma_{M5}} = \frac{N_{p.Ed}}{A_0 f_{y0}/\gamma_{M5}} + \frac{M_{0.Ed}}{W_{el.0} f_{y0}/\gamma_{M5}} \tag{3.15}$$

where $N_{p.Ed} = N_{0.Ed} - \Sigma N_{i.Ed} \cos\theta_i$

$N_{0.Ed}$ is the maximum absolute value of the normal force acting in the chord (Figure 3.2)

$N_{i.Ed}$ is the normal force acting in the brace member (see Figure 3.2)

θ_i is the angle between the brace member and chord

A_0 is the cross-section area of the chord (Annex 11.1)

$M_{0.Ed}$ is the bending moment acting in the chord (see Figure 3.2)

$W_{el.0}$ is the elastic bending modulus of the chord (Annex 11.1)

f_{y0} is the nominal yield strength of the chord

γ_{M5} is the partial safety factor for hollow section lattice joints (Table 2.5)

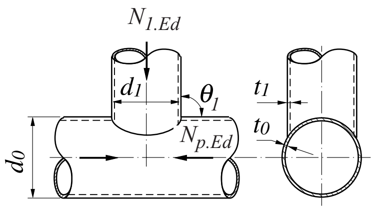
The resistance of joints is calculated in Examples 3.7 - 3.10 according to the tables of Annex 11.3. The resistance of a joint is calculated in the resistance tables for different failure modes, from which the lowest value is adopted as the final resistance of the joint. In the design of hollow section lattice joints, partial safety factor γ_{M5} is applied for the resistance of joints (see Table 2.5), for which the recommended value γ_{M5} = 1,0 has been used as given in EN 1993-1-8.

When using the tables it shall be checked, that the members of the lattice and geometry of the joint meet the requirements presented in the tables. The validity conditions of the joints are met in Examples 3.7 - 3.10, but the checking of them is not presented in this context.

The steel grade is in all Examples SSAB Domex Tube Double Grade, which fulfills the EN 10219 requirements for both steel grades S420MH and S355J2H. Thereby the design calculations may be performed at designer's own choice either according to grade S420 or grade S355. In the following Examples the design calculations have been carried out according to grade S420, unless otherwise specified.

In the tables of Annex 11.3 and in Examples 3.7 - 3.10, it has been supposed that the value of partial safety factor γ_{M5} for hollow section lattice joints is γ_{M5} = 1,0 according to the recommended value in Eurocode and Finnish National Annex. The values valid in other countries must be checked from the National Annex of the relevant country. The tables in Annex 11.3 apply to steel grades, for which the yield strength is not higher than f_y = 460 N/mm^2. When using steel grades of yield strength higher than f_y = 355 N/mm^2, the joint resistances shall be multiplied by the correction factor according to Table 3.1.

Example 3.7
T joint (Table 11.3.4)



The geometry and forces of the joint are as follows:

Chord: 219,1×10
$A_0 = 6569\ mm^2$ *(Annex 11.1)*

Brace member: 168,3×5
$A_1 = 2565\ mm^2$ *(Annex 11.1)*

$\theta_1 = 90°$

$N_{1.Ed} = \ \ 550\ kN$ *(compression)*
$N_{p.Ed} = 1020\ kN$ *(compression)*

The normal stress $\sigma_{0.Ed}$ *in the chord face has an impact on the joint's resistance by the parameter* k_p*:*

$$n_p = \frac{\sigma_{p.Ed}}{f_{y0}/\gamma_{M5}} = \frac{N_{p.Ed}}{A_0 f_{y0}/\gamma_{M5}} + \frac{M_{0.Ed}}{W_{el.0} f_{y0}/\gamma_{M5}} = \frac{1020 \cdot 10^3}{6569 \cdot 420/1,0} + 0 = 0,3697$$

$$k_p = 1,0 - 0,3|n_p| - 0,3n_p^2 = 1,0 - 0,3 \cdot 0,3697 - 0,3 \cdot 0,3697^2 = 0,8481 \le 1,0$$

$$\beta = \frac{d_1}{d_0} = \frac{168,3}{219,1} = 0,7681$$

$$\gamma = \frac{d_0}{2t_0} = \frac{219,1}{2 \cdot 10} = 10,96$$

Chord face failure by yielding:
In respect to chord face failure by yielding, the joint's resistance is:

$$N_{1.Rd} = 0,9 \cdot \frac{\gamma^{0,2} \cdot k_p \cdot f_{y0} \cdot t_0^2}{\sin\theta_1} \cdot (2,8 + 14,2\beta^2)/\gamma_{M5} \qquad \text{(S420: resistance factor = 0,9)}$$

$$= 0,9 \cdot \frac{10,96^{0,2} \cdot 0,8481 \cdot 420 \cdot 10^2}{\sin 90} \cdot (2,8 + 14,2 \cdot 0,7681^2)/1,0 = 578,4\ kN$$

Chord face punching shear:
Chord face punching shear must be checked because
$d_1 = 168,3\ mm \le d_0 - 2t_0 = 199,1\ mm$*:*

$$N_{1.Rd} = 0,9 \cdot \frac{f_{y0}}{\sqrt{3}} t_0 \pi d_1 \left(\frac{1 + \sin\theta_1}{2\sin^2\theta_1}\right)/\gamma_{M5} \qquad \text{(S420: resistance factor = 0,9)}$$

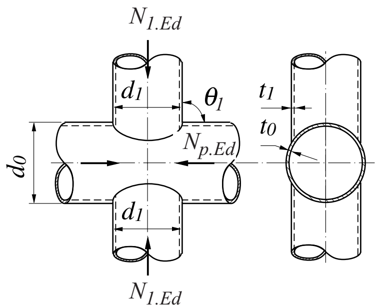
$$= 0,9 \cdot \frac{420}{\sqrt{3}} \cdot 10 \cdot \pi \cdot 168,3 \cdot \left(\frac{1 + \sin 90}{2\sin^2 90}\right)/1,0 = 1154\ kN$$

Joint's resistance:
The joint's resistance is governed hence by the brace member's failure by yielding, wherein:
$N_{1.Rd} = 578{,}4\ kN \geq N_{1.Ed}$ *OK*

In addition to the preceding verification of the joint's resistance, also the following needs to be specifically checked:
- *the welds of the joint, and*
- *the cross-section resistance of the brace member and the chord, and*
- *the buckling resistance of the compressed members (brace member and chord).*

Comparison S420 vs S355:
In case the design calculations would be performed according to grade S355, the joint's resistance would be 519,7 kN < $N_{1.Ed}$*, telling the joint's resistance is not sufficient. Increase of the material strength S355 → S420 improves the joint's resistance in this Example by approx. 11 %.*

Example 3.8
X joint (Table 11.3.4)



The geometry and forces of the joint are as follows:

Chord: 219,1×10
$A_0 = 6569\ mm^2$ *(Annex 11.1)*

Brace members: 193,7×6
$A_1 = 3538\ mm^2$ *(Annex 11.1)*

$\theta_1 = 90°$

$N_{1.Ed} = 550\ kN$ *(compression)*
$N_{p.Ed} = 1020\ kN$ *(compression)*

$$n_p = \frac{\sigma_{p.Ed}}{f_{y0}/\gamma_{M5}} = \frac{N_{p.Ed}}{A_0 f_{y0}/\gamma_{M5}} + \frac{M_{0.Ed}}{W_{el.0} f_{y0}/\gamma_{M5}} = \frac{1020 \cdot 10^3}{6569 \cdot 420/1,0} + 0 = 0,3697$$

$$k_p = 1,0 - 0,3|n_p| - 0,3 n_p^2 = 1,0 - 0,3 \cdot 0,3697 - 0,3 \cdot 0,3697^2 = 0,8481 \leq 1,0$$

$$\beta = \frac{d_1}{d_0} = \frac{193,7}{219,1} = 0,8841$$

Chord face failure by yielding:
In respect to chord face failure by yielding, the joint's resistance is:

$$N_{1.Rd} = 0,9 \cdot \frac{k_p f_{y0} t_0^2}{\sin\theta_1}\left(\frac{5,2}{1-0,81\beta}\right)/\gamma_{M5}$$ *(S420: resistance factor = 0,9)*

$$= 0,9 \cdot \frac{0,8481 \cdot 420 \cdot 10^2}{\sin 90}\left(\frac{5,2}{1-0,81 \cdot 0,8841}\right)/1,0 = 587,2\ kN$$

Chord face punching shear:
Chord face punching shear must be checked because
$d_1 = 193{,}7\ mm \le d_0 - 2t_0 = 199{,}1\ mm$:

$$N_{1.Rd} = 0{,}9 \cdot \frac{f_{y0}}{\sqrt{3}} t_0 \pi d_1 \left(\frac{1 + \sin\theta_1}{2\sin^2\theta_1} \right) / \gamma_{M5} \qquad \text{(S420: resistance factor = 0,9)}$$

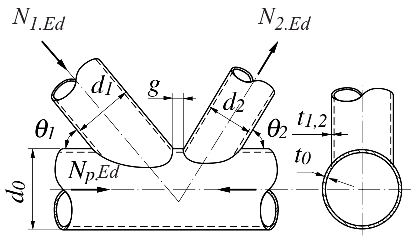
$$= 0{,}9 \cdot \frac{420}{\sqrt{3}} \cdot 10 \cdot \pi \cdot 193{,}7 \cdot \left(\frac{1 + \sin 90}{2\sin^2 90} \right) / 1{,}0 = 1328\ kN$$

Joint's resistance:
The joint's resistance is governed hence by the brace member's failure by yielding, wherein:
$N_{1.Rd} = 587{,}2\ kN \ge N_{1.Ed}$ *OK*

In addition to the preceding verification of the joint's resistance, also the following needs to be specifically checked:
- *the welds of the joint, and*
- *the cross-section resistance of the brace members and the chord, and*
- *the buckling resistance of the compressed members (brace members and chord).*

Comparison S420 vs S355:
In case the design calculations would be performed according to grade S355, the joint's resistance would be 527,6 kN < $N_{1.Ed}$*, telling the joint's resistance is not sufficient. Increase of the material strength S355 → S420 improves the joint's resistance in this Example by approx. 11 %.*

Example 3.9
Gap K joint (Table 11.3.5)



The geometry and forces of the joint are as follows:

Chord: 219,1×10
$A_0 = 6569\ mm^2$ *(Annex 11.1)*

Brace members:
168,3×5 (compression brace)
$A_1 = 2565\ mm^2$ *(Annex 11.1)*

139,7×5 (tension brace)
$A_2 = 2116\ mm^2$ *(Annex 11.1)*

$\theta_1 = 50°$
$\theta_2 = 60°$
$g = 25\ mm$

$N_{1.Ed} = 600\ kN$ *(compression)*
$N_{2.Ed} = 530{,}7\ kN$ *(tension)*
$N_{p.Ed} = 636{,}4\ kN$ *(compression)*

$$n_p = \frac{\sigma_{p.Ed}}{f_{y0}/\gamma_{M5}} = \frac{N_{p.Ed}}{A_0 f_{y0}/\gamma_{M5}} + \frac{M_{0.Ed}}{W_{el.0} f_{y0}/\gamma_{M5}} = \frac{636,4 \cdot 10^3}{6569 \cdot 420/1,0} + 0 = 0,2307$$

$$k_p = 1,0 - 0,3|n_p| - 0,3n_p^2 = 1,0 - 0,3 \cdot 0,2307 - 0,3 \cdot 0,2307^2 = 0,9148 \le 1,0$$

$$\gamma = \frac{d_0}{2t_0} = \frac{219,1}{2 \cdot 10} = 10,96$$

Check the validity conditions of the joint's gap:
$g = 25\ mm \ge t_1 + t_2 = 5 + 5 = 10\ mm$ *OK*
$\Rightarrow$ *the joint's gap meets the validity conditions*

Check the joint's eccentricity (expression (3.6)) and the permitted limit values for it (expression (3.8)):

$$e = \left(\frac{d_1}{2\sin\theta_1} + \frac{d_2}{2\sin\theta_2} + g\right)\frac{\sin\theta_1 \sin\theta_2}{\sin(\theta_1 + \theta_2)} - \frac{d_0}{2}$$

$$= \left(\frac{168,3}{2\sin 50} + \frac{139,7}{2\sin 60} + 25\right)\frac{\sin 50 \sin 60}{\sin(50 + 60)} - \frac{219,1}{2} = 42,6\ mm$$

$-0,55d_0 = -120,5\ mm \le e = 42,6\ mm \le 0,25d_0 = 54,8\ mm$ *OK*
$\Rightarrow$ *the joint's eccentricity meets the validity conditions*

Brace member 1 (= compression brace member):

Chord face failure by yielding:

$$k_g = \gamma^{0,2} \cdot \left(1 + \frac{0,024 \cdot \gamma^{1,2}}{1 + e^{\left(\frac{g}{2t_0} - 1,33\right)}}\right) = 10,96^{0,2} \cdot \left(1 + \frac{0,024 \cdot 10,96^{1,2}}{1 + e^{\left(\frac{25}{2 \cdot 10} - 1,33\right)}}\right) = 1,971$$

$$N_{1.Rd} = 0,9 \cdot \frac{k_g \cdot k_p \cdot f_{y0} \cdot t_0^2}{\sin\theta_1}\left(1,8 + 10,2 \cdot \frac{d_1}{d_0}\right)/\gamma_{M5} \qquad \text{(S420: resistance factor = 0,9)}$$

$$= 0,9 \cdot \frac{1,971 \cdot 0,9148 \cdot 420 \cdot 10^2}{\sin 50}\left(1,8 + 10,2 \cdot \frac{168,3}{219,1}\right)/1,0 = 857,2\ kN$$

Chord face punching shear:
Chord face punching shear must be checked because
$d_1 = 168,3\ mm \le d_0 - 2t_0 = 199,1\ mm$:

$$N_{1.Rd} = 0,9 \cdot \frac{f_{y0}}{\sqrt{3}} t_0 \pi d_1 \left(\frac{1 + \sin\theta_1}{2\sin^2\theta_1}\right)/\gamma_{M5} \qquad \text{(S420: resistance factor = 0,9)}$$

$$= 0,9 \cdot \frac{420}{\sqrt{3}} \cdot 10 \cdot \pi \cdot 168,3 \cdot \left(\frac{1 + \sin 50}{2\sin^2 50}\right)/1,0 = 1736\ kN$$

The resistance of the compression brace member is hence governed by the chord face failure by yielding, wherein:
$N_{1.Rd} = 857{,}2\ kN \geq N_{1.Ed}$ *OK*

Brace member 2 (= tension brace member):

Chord face failure by yielding:
Resistance can be calculated from that of the compression brace member:

$$N_{2.Rd} = \frac{\sin\theta_1}{\sin\theta_2} \cdot N_{1.Rd} = \frac{\sin 50}{\sin 60} \cdot 857{,}2 = 758{,}2\ kN$$

Chord face punching shear:
Chord face punching shear must be checked because
$d_2 = 139{,}7\ mm \leq d_0 - 2t_0 = 199{,}1\ mm$:

$$N_{2.Rd} = 0{,}9 \cdot \frac{f_{y0}}{\sqrt{3}} t_0 \pi d_2 \left(\frac{1+\sin\theta_2}{2\sin^2\theta_2}\right) / \gamma_{M5} \qquad \text{(S420: resistance factor = 0,9)}$$

$$= 0{,}9 \cdot \frac{420}{\sqrt{3}} \cdot 10 \cdot \pi \cdot 139{,}7 \cdot \left(\frac{1+\sin 60}{2\sin^2 60}\right) / 1{,}0 = 1192\ kN$$

The resistance of the tension brace member is hence also here governed by the chord face failure by yielding, wherein:
$N_{2.Rd} = 758{,}2\ kN \geq N_{2.Ed}$ *OK*

In addition to the preceding verification of the joint's resistance, also the following needs to be specifically checked:
- *the welds of the joint, and*
- *the cross-section resistance of the brace members and the chord, and*
- *the buckling resistance of the compressed members (chord and brace member 1).*

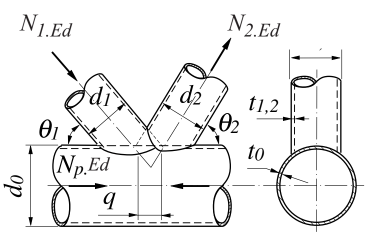
Comparison S420 vs S355:
In case the design calculations would be performed according to grade S355, the joint's resistance would be still governed by the chord face failure by yielding, wherein the compression brace member's resistance would be 788,4 kN and the tension brace member's resistance would be 697,4 kN. Increase of the material strength S355 → S420 improves the joint's resistance for both brace members in this Example by approx. 9 %.

Example 3.10
Overlap K joint (Table 11.3.6)

The geometry and forces of the joint are as follows:

Chord: $219{,}1 \times 10$
$A_0 = 6569\ mm^2$ *(Annex 11.1)*

Brace members: $139{,}7 \times 5$



$A_1 = A_2 = 2116\ mm^2$ *(Annex 11.1)*

$\theta_1 = 40°$
$\theta_2 = 50°$
$q = -g = 85\ mm$ *(overlap)*

$N_{1.Ed} = 600\ kN$ *(compression)*
$N_{2.Ed} = 503{,}5\ kN$ *(tension)*
$N_{p.Ed} = 636{,}4\ kN$ *(compression)*

$$n_p = \frac{\sigma_{p.Ed}}{f_{y0}/\gamma_{M5}} = \frac{N_{p.Ed}}{A_0 f_{y0}/\gamma_{M5}} + \frac{M_{0.Ed}}{W_{el.0} f_{y0}/\gamma_{M5}} = \frac{636,4 \cdot 10^3}{6569 \cdot 420/1,0} + 0 = 0,2307$$

$$k_p = 1,0 - 0,3|n_p| - 0,3 n_p^2 = 1,0 - 0,3 \cdot 0,2307 - 0,3 \cdot 0,2307^2 = 0,9148 \le 1,0$$

$$\gamma = \frac{d_0}{2t_0} = \frac{219,1}{2 \cdot 10} = 10,96$$

Check the validity conditions of the joint's eccentricity:

$$e = \left(\frac{d_1}{2\sin\theta_1} + \frac{d_2}{2\sin\theta_2} + g\right)\frac{\sin\theta_1 \sin\theta_2}{\sin(\theta_1+\theta_2)} - \frac{d_0}{2}$$

$$= \left(\frac{139,7}{2\sin 40} + \frac{139,7}{2\sin 50} + (-85)\right)\frac{\sin 40 \sin 50}{\sin(40+50)} - \frac{219,1}{2} = -53,0\ mm$$

$$-0,55d_0 = -120,5\ mm \le e = -53,0\ mm \le 0,25d_0 = 54,8\ mm \quad OK$$

$\Rightarrow$ *the joint's eccentricity meets the validity conditions*

The overlap ratio λ_{ov} is (Figure 3.3):

$$p = d_2/\sin\theta_2 = 139,7/\sin 50 = 182,4\ mm$$

$$\lambda_{ov} = (q/p) \cdot 100\ \% = (85/182,4) \cdot 100\ \% = 46,6\ \% \ge 25\ \% \quad OK$$

Shear of the brace members off the chord:
This failure mode must be checked if one of the following conditions takes place:

if $\lambda_{ov} > \lambda_{ov.lim} = 80\ \%$ *(when the hidden seam is welded to the chord)*

if $h_1 < b_1$ *or* $h_2 < b_2$ *(this concerns rectangular sections only)*

$\Rightarrow$ *now neither one of the above conditions takes place, thus shear of the brace members off the chord will not become critical.*

<u>When the chord is a circular hollow section</u>, in case of overlap joint, the resistance of the overlapped member (i.e. the member located underneath) is always calculated first (see Table 11.3.6):

Brace member 1 (= overlapped brace member):

Chord face failure by yielding:

$$k_g = \gamma^{0,2} \cdot \left(1 + \frac{0,024 \cdot \gamma^{1,2}}{1 + e^{\left(\frac{-q}{2t_0} - 1,33\right)}} \right) = 10,96^{0,2} \cdot \left(1 + \frac{0,024 \cdot 10,96^{1,2}}{1 + e^{\left(\frac{-85}{2 \cdot 10} - 1,33\right)}} \right) = 2,297$$

$$N_{1.Rd} = 0,9 \cdot \frac{k_g \cdot k_p \cdot f_{y0} \cdot t_0^2}{\sin\theta_1} \left(1,8 + 10,2 \cdot \frac{d_1}{d_0} \right) / \gamma_{M5} \qquad \text{(S420: resistance factor = 0,9)}$$

$$= 0,9 \cdot \frac{2,297 \cdot 0,9148 \cdot 420 \cdot 10^2}{\sin 40} \left(1,8 + 10,2 \cdot \frac{139,7}{219,1} \right) / 1,0 = 1026 \ kN \geq N_{1.Ed} \quad OK$$

Brace member 2 (= overlapping brace member):

Resistance can be calculated from the resistance of the overlapped brace member:

$$N_{2.Rd} = \frac{\sin\theta_1}{\sin\theta_2} \cdot N_{1.Rd} = \frac{\sin 40}{\sin 50} \cdot 1026 = 860,9 \ kN \geq N_{2.Ed} \qquad OK$$

In addition to the preceding verification of the joint's resistance, also the following needs to be specifically checked:

- *the welds of the joint, and*
- *the cross-section resistance of the brace members and the chord, and*
- *the buckling resistance of the compressed members (chord and brace member 1).*

Comparison S420 vs S355:
In case the design calculations would be performed according to grade S355, the joint's resistance would be still governed by the chord face failure by yielding, wherein the compression brace member's resistance would be 943,6 kN and the tension brace member's resistance would be 791,8 kN. Hence, increase of the material strength S355 → S420 improves the joint's resistance for both brace members in this Example by approx. 9 %.

3.1.3 Joint of circular, square and rectangular brace members to an I-section chord

The joints are designed according to the same principles as in clause 3.1.1, but now the Tables 11.3.7 and 11.3.8 of Annex 11.3 shall be used.

3.2 Welded joints in frame structures

There is both bending moment and axial forces present in the joints of frame structures. The stiffness of the joint influences the magnitude of joint moment, and the joint moment influences the stiffness of the joint. To determine the final distribution of action effects, the calculation should be done by iteration. Usually a conservative method is to model all joints of the structure first as fully pinned, whereby maximum span moments are obtained, and then as fully rigid, whereby the maximum actions for the joints and through them those to the adjacent elements are obtained. Annex 11.4 gives instructions for estimating the stiffness of the joint, when square or rectangular hollow sections are applied.

A moment-rotation curve of a frame structure joint is presented in Figure 3.5. The slope of the curve represents the stiffness of the joint, which depends on the beam width to column width ratio, and the wall thicknesses of the joint elements. The bigger the b_1/b_0 ratio and wall thickness is, the stiffer the joint is.

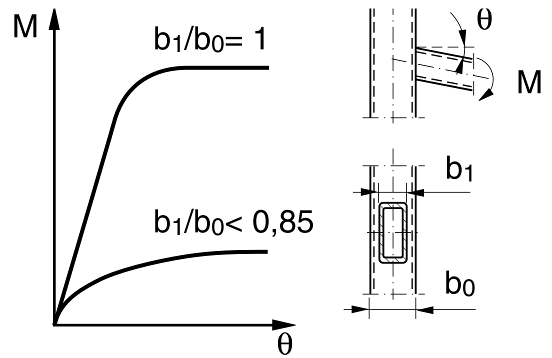


Figure 3.5 Stiffness of a frame structure joint

3.2.1 Joints of square and rectangular hollow sections subject to bending moment

Table 11.3.11 of Annex 11.3 presents calculation formulae for joints of square and rectangular hollow sections subject to bending moment in the plane of the lattice (in-plane, M_{ip}) or perpendicular to the plane of the lattice (out-of-plane, M_{op}).

The joints of brace members to the chord, when the brace members are subject to simultaneous bending and normal force, are designed using the following interaction formula [4,5,6]:

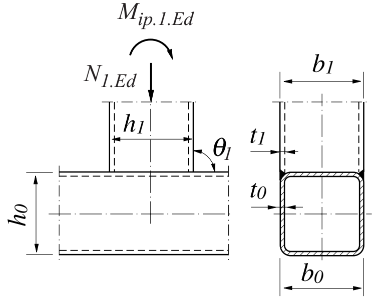
$$\frac{N_{1.Ed}}{N_{1.Rd}} + \frac{M_{ip.1.Ed}}{M_{ip.1.Rd}} + \frac{M_{op.1.Ed}}{M_{op.1.Rd}} \leq 1,0 \qquad (3.16)$$

where $N_{1.Ed}$ is the normal force acting in the brace member
$N_{1.Rd}$ is the joint resistance of the brace member to normal force according to clause 3.1.1
$M_{ip.1.Ed}$ is the bending moment acting in the brace member in the plane of the lattice
$M_{ip.1.Rd}$ is the bending resistance of the brace member joint in the plane of the lattice (Annex 11.3)
$M_{op.1.Ed}$ is the bending moment acting in the brace member in the plane perpendicular to the lattice
$M_{op.1.Rd}$ is the bending resistance of the brace member joint in the plane perpendicular to the lattice (Annex 11.3)

The design value of the internal moment $M_{1.Ed}$ may be determined at a point where the centreline of the brace member intersects the surface of the chord [4,5,6].

In addition to the interaction verification, the resistance of the joint shall be checked separately also for the normal force alone (clause 3.1.1) and for the bending moment alone (Table 11.3.11 of Annex 11.3).

Example 3.11
T joint subjected to compression and bending
(Tables 11.3.1 and 11.3.11)



The members and forces of the joint are as follows:

Chord: $150 \times 150 \times 6$
$A_0 = 3363\ mm^2$ *(Annex 11.1)*

Brace member: $150 \times 150 \times 6$
$A_1 = 3363\ mm^2$ *(Annex 11.1)*
$W_{pl.1} = 179{,}9 \cdot 10^3\ mm^3$ *(Annex 11.1)*

Loads:
$M_{ip.1.Ed} = 15\ kNm$
$N_{1.Ed} = 150\ kN$ *(compression)*

$\beta = b_1 / b_0 = 150/150 = 1{,}0$

Bending resistance of the joint (Table 11.3.11):

Chord side wall failure by yielding:
Since $\beta = 1{,}0$, *check the chord side wall failure by yielding :*
T joint: $f_{yk} = f_{y0}$

$$M_{ip.1.Rd} = 0{,}9 \cdot 0{,}5 f_{yk} t_0 (h_1 + 5t_0)^2 / \gamma_{M5} \qquad \text{(S420: resistance factor = 0,9)}$$
$$= 0{,}9 \cdot 0{,}5 \cdot 420 \cdot 6 \cdot (150 + 5 \cdot 6)^2 / 1{,}0 = 36{,}7\ kNm$$

Brace member failure by yielding:
Since $\beta = 1{,}0$, *also brace member failure by yielding must be checked:*

$$b_{eff} = \frac{10}{b_0/t_0} \cdot \frac{f_{y0} \cdot t_0}{f_{y1} \cdot t_1} \cdot b_1 = \frac{10}{150/6} \cdot \frac{420 \cdot 6}{420 \cdot 6} \cdot 150 = 60\ mm \le b_1 = 150\ mm$$

$$M_{ip.1.Rd} = 0{,}9 \cdot f_{y1} \cdot \left[W_{pl.1} - \left(1 - \frac{b_{eff}}{b_1}\right) b_1 (h_1 - t_1)\, t_1 \right] / \gamma_{M5} \quad \text{(S420: resistance factor = 0,9)}$$
$$= 0{,}9 \cdot 420 \cdot \left[179{,}9 \cdot 10^3 - \left(1 - \frac{60}{150}\right) \cdot 150 \cdot (150 - 6) \cdot 6 \right] / 1{,}0 = 38{,}6\ kNm$$

The joint's bending resistance is the smallest of the above obtained results, i.e.
$M_{ip.1.Rd} = 36{,}7\ kNm \ge M_{ip.1.Ed}$ *OK*

Compression resistance of the joint (Table 11.3.1):

Brace member's compression resistance at the joint shall be calculated according to clause 3.1.1. Since the brace member and the chord have equal width (i.e. $\beta = 1,0$), brace member failure by yielding and also chord side wall buckling / yielding must be checked:

Chord side wall buckling or yielding:
First, calculate buckling stress by using buckling curve c:

$$\bar{\lambda} = 3,46 \cdot \frac{\left(\frac{h_0}{t_0} - 2\right) \cdot \sqrt{\frac{1}{\sin\theta_1}}}{\pi \cdot \sqrt{\frac{E}{f_{y0}}}} = 3,46 \cdot \frac{\left(\frac{150}{6} - 2\right) \cdot \sqrt{\frac{1}{\sin 90}}}{\pi \cdot \sqrt{\frac{2,1 \cdot 10^5}{420}}} = 1,133$$

$$\Phi = 0,5 \cdot [1 + \alpha(\bar{\lambda} - 0,2) + \bar{\lambda}^2] = 0,5 \cdot [1 + 0,49 \cdot (1,133 - 0,2) + 1,133^2] = 1,370$$

$$\chi = \frac{1}{\Phi + \sqrt{\Phi^2 - \bar{\lambda}^2}} = \frac{1}{1,370 + \sqrt{1,370^2 - 1,133^2}} = 0,4672 \le 1,0$$

T joint:

$$f_b = \chi \cdot f_{y0} = 0,4672 \cdot 420 = 196,2 \text{ N/mm}^2$$

Because the chord has no other loads except the brace member loads, we will get:

$$n = \frac{\sigma_{0.Ed}}{f_{y0}/\gamma_{M5}} = \frac{N_{0.Ed}}{A_0 f_{y0}/\gamma_{M5}} + \frac{M_{0.Ed}}{W_{el.0} f_{y0}/\gamma_{M5}} = 0 + 0 = 0$$

$$k_n = 1,3 - \frac{0,4|n|}{\beta} = 1,3 - 0 = 1,3 \quad \text{but } k_n \le 1,0 \quad \Rightarrow k_n = 1,0$$

$$N_{1.Rd} = 0,9 \cdot \frac{k_n f_b t_0}{\sin\theta_1}\left(\frac{2h_1}{\sin\theta_1} + 10t_0\right)/\gamma_{M5} \qquad \text{(S420: resistance factor = 0,9)}$$

$$= 0,9 \cdot \frac{1,0 \cdot 196,2 \cdot 6}{\sin 90}\left(\frac{2 \cdot 150}{\sin 90} + 10 \cdot 6\right)/1,0 = 381,4 \text{ kN}$$

Brace member failure by yielding:

$$b_{eff} = \frac{10}{b_0/t_0} \cdot \frac{f_{y0} t_0}{f_{y1} t_1} \cdot b_1 = \frac{10}{150/6} \cdot \frac{420 \cdot 6}{420 \cdot 6} \cdot 150 = 60 \text{ mm} \le b_1 = 150 \text{ mm}$$

$$N_{1.Rd} = 0,9 \cdot f_{y1} t_1 (2h_1 - 4t_1 + 2b_{eff})/\gamma_{M5} \qquad \text{(S420: resistance factor = 0,9)}$$

$$= 0,9 \cdot 420 \cdot 6 \cdot (2 \cdot 150 - 4 \cdot 6 + 2 \cdot 60)/1,0 = 898,1 \text{ kN}$$

The joint's compression resistance is the smallest of the above obtained results
$N_{1.Rd} = 381,4$ kN $\ge N_{1.Ed}$ OK

Interaction formula for the joint:

$$\frac{N_{1.Ed}}{N_{1.Rd}} + \frac{M_{ip.1.Ed}}{M_{ip.1.Rd}} + \frac{M_{op.1.Ed}}{M_{op.1.Rd}} = \frac{150}{381,4} + \frac{15}{36,7} + 0 = 0,8020 \leq 1,0 \qquad OK$$

In addition to the preceding verification of the joint's resistance, also the following needs to be specifically checked:
- *the welds of the joint, and*
- *the cross-section resistance of the brace member and the chord, and*
- *the buckling resistance of the compressed members (brace member).*

Comparison S420 vs S355:
In case the design calculations would be performed according to grade S355, the result of the (M+N) interaction verification would be (S420: 0,8020 →) S355: 0,8137. By comparing the 'utilisation ratios' of the interaction formula, we can see that in this Example the increase of the material strength S355 → S420 does not improve the joint's overall resistance very much.

3.2.2 Joints of circular hollow sections subject to bending moment

Table 11.3.12 of Annex 11.3 presents calculation formulae for joints of circular hollow sections subject to bending moment in the plane of the lattice or perpendicular to the plane of the lattice.

The joints of brace members to the chord, when the brace members are subject to simultaneous bending and normal force, are designed using the following interaction formula (cf. square and rectangular hollow sections earlier) [4,5,6]:

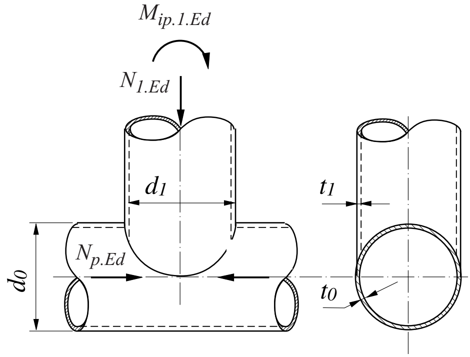
$$\frac{N_{1.Ed}}{N_{1.Rd}} + \left[\frac{M_{ip.1.Ed}}{M_{ip.1.Rd}}\right]^2 + \frac{M_{op.1.Ed}}{M_{op.1.Rd}} \leq 1,0 \tag{3.17}$$

where $N_{1.Ed}$ is the normal force acting in the brace member
$N_{1.Rd}$ is the joint resistance of the brace member to normal force according to clause 3.1.2
$M_{ip.1.Ed}$ is the bending moment acting in the brace member in the plane of the lattice
$M_{ip.1.Rd}$ is the bending resistance of the brace member joint in the plane of the lattice (Annex 11.3)
$M_{op.1.Ed}$ is the bending moment acting in the brace member in the plane perpendicular to the lattice
$M_{op.1.Rd}$ is the bending resistance of the brace member joint in the plane perpendicular to the lattice (Annex 11.3)

The design value of the internal moment $M_{1.Ed}$ may be determined at a point where the centreline of the brace member intersects the surface of the chord [4,5,6].

In addition to the interaction verification, the resistance of the joint shall be checked separately also for the normal force alone (clause 3.1.2) and for the bending moment alone (Table 11.3.12 of Annex 11.3).

Example 3.12
T joint subjected to compression and bending (Tables 11.3.4 and 11.3.12)



The members and forces of the joint are as follows:

Chord: 219,1×5
$A_0 = 3363\ mm^2$ *(Annex 11.1)*

Brace member: 219,1×5
$A_1 = 3363\ mm^2$ *(Annex 11.1)*

Loads:
$M_{ip.1.Ed} = 33\ kNm$
$N_{1.Ed} = 100\ kN$ *(compression)*
$N_{p.Ed} = 300\ kN$ *(compression)*

$\theta_1 = 90°$
$\beta = d_1 / d_0 = 219{,}1 / 219{,}1 = 1{,}0$
$\gamma = 0{,}5 d_0 / t_0 = 0{,}5 \cdot 219{,}1 / 5 = 21{,}91$

First, calculate the parameters related to the normal stress $\sigma_{0.Ed}$ *acting in the chord face:*

$$n_p = \frac{\sigma_{p.Ed}}{f_{y0}/\gamma_{M5}} = \frac{N_{p.Ed}}{A_0 f_{y0}/\gamma_{M5}} + \frac{M_{0.Ed}}{W_{el.0} f_{y0}/\gamma_{M5}} = \frac{300 \cdot 10^3}{3363 \cdot 420/1,0} + 0 = 0,2124$$

$$k_p = 1,0 - 0,3|n_p| - 0,3 n_p^2 = 1,0 - 0,3 \cdot 0,2124 - 0,3 \cdot 0,2124^2 = 0,9227 \le 1,0$$

Bending resistance of the joint (Table 11.3.12):

Chord face failure by yielding:

$$M_{ip.1.Rd} = 0,9 \cdot \frac{4,85 \cdot \sqrt{\gamma} \cdot k_p \cdot f_{y0} \cdot t_0^2 \cdot d_1}{\sin\theta_1} \cdot \beta/\gamma_{M5} \qquad \text{(S420: resistance factor = 0,9)}$$

$$= 0,9 \cdot \frac{4,85 \cdot \sqrt{21,91} \cdot 0,9227 \cdot 420 \cdot 5^2 \cdot 219,1}{\sin 90} \cdot 1,0/1,0 = 43,4\ kNm$$

Chord face punching shear:
Chord face punching shear does not need to be checked because
$d_1 = 219{,}1\ mm > d_0 - 2t_0 = 219{,}1 - 2 \cdot 5 = 209{,}1\ mm.$

Hence, the joint's bending resistance is
$M_{ip.1.Rd} = 43{,}4\ kNm \ge M_{ip.1.Ed}$ *OK*

Compression resistance of the joint (Table 11.3.4):

Brace member's compression resistance at the joint shall be calculated according to clause 3.1.2.
Chord face failure by yielding:

$$N_{1.Rd} = 0,9 \cdot \frac{\gamma^{0,2} \cdot k_p \cdot f_{y0} \cdot t_0^2}{\sin\theta_1} \cdot (2,8 + 14,2\,\beta^2)/\gamma_{M5} \qquad \text{(S420: resistance factor = 0,9)}$$

$$= 0,9 \cdot \frac{21,91^{0,2} \cdot 0,9227 \cdot 420 \cdot 5^2}{\sin 90}(2,8 + 14,2 \cdot 1,0^2)/1,0 = 274,8\ kN$$

Chord face punching shear:
Chord face punching shear does not need to be checked because
$d_1 = 219,1\ mm > d_0 - 2t_0 = 219,1 - 2 \cdot 5 = 209,1\ mm.$

Thus, the compression resistance of the joint, governed by the chord face, is
$N_{1.Rd} = 274,8\ kN \geq N_{1.Ed}$ *OK*

Interaction formula for the joint:

$$\frac{N_{1.Ed}}{N_{1.Rd}} + \left[\frac{M_{ip.1.Ed}}{M_{ip.1.Rd}}\right]^2 + \frac{M_{op.1.Ed}}{M_{op.1.Rd}} = \frac{100}{274,8} + \left[\frac{33}{43,4}\right]^2 + 0 = 0,9421 \leq 1,0 \qquad OK$$

In addition to the preceding verification of the joint's resistance, also the following needs to be specifically checked:
- *the welds of the joint, and*
- *the cross-section resistance of the brace member and the chord, and*
- *the buckling resistance of the compressed members (brace member and chord).*

Comparison S420 vs S355:
In case the design calculations would be performed according to grade S355, the result of the (M+N) interaction verification would be (S420: 0,9421 →) S355: 1,075 > 1,0, telling the joint's resistance is not sufficient. By comparing the 'utilisation ratios' of the interaction formula, we can see that the increase of the material strength S355 → S420 improves the joint's overall resistance in this Example by approx. 14 %.

3.3 Design of welds

3.3.1 General

Welds to be done on site should be avoided if possible, because welding is more troublesome on construction site than in the workshop. Moreover, quality assurance of welding can be performed easier in the workshop.

The provisions presented herein are based on Part EN 1993-1-8 of Eurocode, the rules of which cover common arc welding methods with welding consumables (e.g. metal-arc welding with covered electrode, MIG/MAG welding, submerged arc welding) and when the thickness of the plates to be joined is at least 4 mm and the wall thickness of the structural hollow sections is at least 2,5 mm [4,5,6].

It should be noted, that the provisions presented in Eurocode do not cover laser welding.

The welding consumables to be used shall conform to the reference standards presented in EN 1993-1-8. The welding consumables shall be chosen to have their yield strength, ultimate tensile strength, elongation at failure and Charpy-V impact energy at least equivalent minimum values as those of the parent metal to be welded [4,5,6].

EN ISO 5817 defines the weld quality levels for the welded joints [15]. There are three quality levels which are designated by symbols D (moderate), C (intermediate) and B (stringent). The quality levels are based on permitted weld imperfections, for which the limits are given in the standard itself (see more details in clause 8.5.1 of this handbook). The required quality level for the weld is determined according to the execution class chosen in EN 1090-2 for the structure or the member, as presented in Table 3.6 [14]. The execution classes (EXC) which are needed when using the table, and the procedure to select between them, is presented in Chapter 8. Usually the quality level B or C will be chosen for the weld. The choice of weld quality level has, however, no influence to the calculation of the resistance of the welds.

Table 3.6 Determination of weld quality levels according to EN 1090-2 [14]

	Execution class (EN 1090-2)			
	EXC1	EXC2	EXC3	EXC4
Weld quality level a)	Quality level D	Quality level C b)	Quality level B	Quality level B+ c)

a) Weld quality level according to EN ISO 5817 except imperfection types:
- "Incorrect toe" (505)
- "Micro lack of fusion" (401)

that are not taken into account.

b) Generally quality level C, except quality level D for the following imperfection types:
- "Undercut" (5011,5012)
- "Overlap" (506)
- "Stray arc" (601)
- "End crater pipe" (2025)

c) Quality level B, and additional requirements according to Table 17 of EN 1090-2.

Lamellar tearing of welds shall be avoided. Lamellar tearing is discussed in Chapter 5.

Welded joints shall be designed to have sufficient deformation capacity. In joints where plastic hinge may develop, the welds shall be designed to have at least the same design resistance as the weakest of the parts to be joined. In other joints where deformation capacity is required due to the possibility of excessive straining, the welds require sufficient strength not to rupture before general yielding in the adjacent parent material [4,5,6].

On hollow sections, the welds between brace members and chord shall be designed such that they have sufficient resistance to withstand uneven stress distributions and also enough deformation capability for redistribution of bending moments. The design resistance of the weld, per unit length of perimeter of a brace member, should normally not be less than the design resistance of the cross-section of that member per unit length of perimeter. This requirement can be neglected, if smaller weld size can be justified both in regard to resistance, and deformation capacity and rotation capacity, taking at the same time into account that only part of the weld length is effective [4,5,6].

3.3.2 Welding a structural hollow section and cold-formed corner

In welded joints of structural hollow sections, usually the weld goes around the whole perimeter of the hollow section, being either a butt weld or a fillet weld or combination of both of them. In lattice joints, the choice of the weld type is made on the basis of the angle between the brace member and the chord (see Chapter 8, Figures 8.15 - 8.18). According to EN 1993-1-8, on square and rectangular hollow sections also flare groove welds can be used, Figure 3.6.

Preparation of the parts to be welded and execution of welding is presented in more details in Chapter 8.

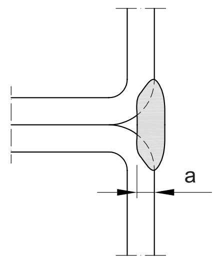
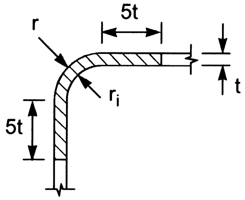


Figure 3.6 Flare groove weld in a square or rectangular hollow section, and its design throat thickness [4,5,6]

According to EN 1993-1-8, the cold-formed corner of a hollow section and the region $5t$ on both sides of it is allowed to be welded provided the conditions presented in Table 3.7 are fulfilled. If the conditions of the table are not met, the cold-formed regions shall be normalized after cold-forming but before welding [4,5,6].

The cold-formed SSAB Domex Tube structural hollow sections fulfill the requirements of EN 1993-1-8 presented in Table 3.7, thus they can be welded without normalizing before welding.

Table 3.7 Conditions for welding in the cold-formed region or adjacent to it [4,5,6]



r/t	r_i/t	Strain in the surface due to cold-forming (%)	Maximum permitted thickness a) (mm)
≥26	≥25	≤2	all
≥11	≥10	≤5	all
≥4,0	≥3,0	≤14	24
≥3,0	≥2,0	≤20	12
≥2,5	≥1,5	≤25	10
≥2,0	≥1,0	≤33	6

a) When using fully killed steel with Al ≥ 0,02 %

Cold-formed hollow sections according to EN 10219:2006 which do not fulfill the limits presented in this table, are deemed to fulfill these limits provided that all of the following conditions are fulfilled:
- the wall thickness is not more than 12,5 mm
- the steel is aluminium-killed according to the condition (a)
- the steel grade is one of the following: J2H, K2H, MH, MLH, NH or NLH
- the chemical analysis of the steel fulfills the following conditions: C ≤ 0,18 %
P ≤ 0,020 %
S ≤ 0,012 %

3.3.3 Fillet welds

Fillet welds may be used to join parts, when the angle between fusion faces is 60 - 120°. Also angles smaller than 60° are permitted. In such case the weld is, however, considered to be a partial penetration butt weld (clause 3.3.4), where the weld thickness is chosen to be not more than the thickness which can consistently be achieved (to be proved with welding tests). When the angle is more than 120° the resistance of the fillet weld shall be determined by testing according to the provisions presented in Part EN 1990 of Eurocode for "Design assisted by testing" [4,5,6].

The effective throat thickness of a fillet weld is the height of the largest triangle (with equal or unequal legs) that can be inscribed within the fusion faces and weld surface, measured perpendicular to the outer side of this triangle, see Figure 3.8 [4,5,6]. Normally the aim is to use fillet welds with equal legs.

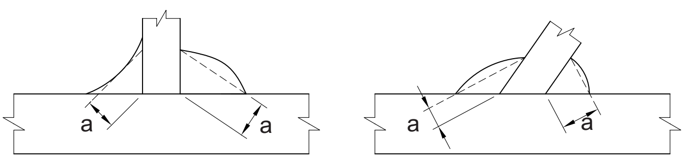


Figure 3.7 Effective throat thickness of a fillet weld [4,5,6]

For a load carrying fillet weld, the throat thickness shall be at least 3 mm [4,5,6]:

$$a \geq 3\ mm \tag{3.18}$$

Welding technology related aspects affect the weld size, when it is not determined on the basis of requirements of structural engineering. In such case, due to the cooling rate of the weld, the throat thickness is advised in [16] to be determined for grades S235-S420 as follows:

$$a \geq \sqrt{t}\ mm - 0,5\ mm \tag{3.19}$$

where t is the material thickness (thicker of the plates to be joined). If the throat thickness of the weld is smaller than the value according to expression (3.19), the part to be joined shall be preheated.

The designer always marks in the workshop drawing the effective throat thickness obtained from calculations without the effect of penetration. When using mechanized welding, in the workshop it is possible to consider exploitation of penetration according to Figure 3.8, provided that it is proven by welding tests that the required penetration can consistently be achieved with the applied welding procedure [4,5,6].

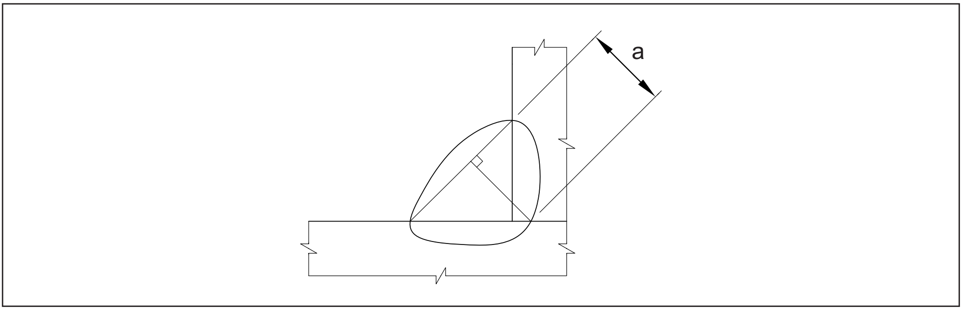


Figure 3.8 Effective throat thickness of a fillet weld, when penetration is exploited [4,5,6]

The effective length of a fillet weld l_{eff} should be taken as the length over which the fillet is full-size. This may be taken as the overall length of the weld reduced by twice the effective throat thickness. Provided that the weld is full-size throughout its length including starts and terminations, no reduction in effective length need to be made for either the start or the termination of the weld.

Fillet welds having the effective length less than 30 mm or 6 times the throat thickness, whichever is larger, shall not be considered as load carrying [4,5,6]:

$$l_{eff} \geq max[6a\ ;\ 30\ mm] \tag{3.20}$$

Fillet welds are not permitted to be finished at the ends or sides of parts, but they shall be welded to be continuous and full-sized around the corner for a distance of twice the leg length of the weld, if the configuration of the joint makes it possible. In case of intermittent fillet welds this rule is applied only to the last weld at corners [4,5,6].

The resistance of a fillet weld is calculated by using either the Directional method or the Simplified method as presented in the following.

3.3.3.1 Resistance of a fillet weld by using the Directional method

In the Directional method the forces transmitted by a unit length of weld are divided into components parallel and transverse to the longitudinal axis of the weld and normal and transverse to the plane of its throat [4,5,6].

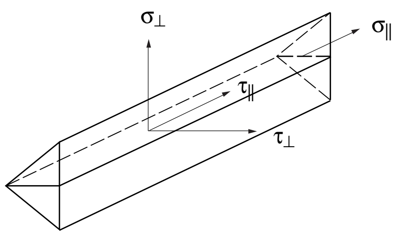
The design throat area of the weld is [4,5,6]:

$$A_w = \sum a l_{eff} \tag{3.21}$$

where a is the effective throat thickness of a fillet weld
l_{eff} is the effective length of a fillet weld

The location of the design throat area of the weld is supposed to be concentrated in the root of the weld [4,5,6].

The stresses are divided on the throat section of the weld to the components according to Figure 3.9. Each component is supposed to be distributed uniformly on the throat section of the weld [4,5,6].



- $\sigma_\perp$ is the normal stress perpendicular to the throat
- $\sigma_{||}$ is the normal stress parallel to the axis of the weld
- $\tau_\perp$ is the shear stress perpendicular to the axis of the weld (in the plane of the throat)
- $\tau_{||}$ is the shear stress parallel to the axis of the weld (in the plane of the throat).

Figure 3.9 Stresses on the throat section of a fillet weld

The normal stress $\sigma_{||}$ parallel to the axis of the weld is not considered when verifying the resistance of the weld [4,5,6]. The normal stress parallel to the axis develops because the weld is obliged to stretch in the longitudinal direction the same amount as the joined parts. In respect to the weld, this stress is secondary because it is not caused by a primary force loading the weld. That is why it is not considered when calculating the static resistance of the weld.

The design conditions of a fillet weld are [4,5,6]:

$$\sqrt{\sigma_{\perp}^2 + 3(\tau_{\perp}^2 + \tau_{\parallel}^2)} \leq \frac{f_u}{\beta_w \gamma_{M2}} \qquad (3.22)$$

$$\sigma_{\perp} \leq \frac{0,9 f_u}{\gamma_{M2}} \qquad (3.23)$$

where f_u is the nominal ultimate tensile strength of the weakest part to be joined
β_w is the appropriate strength factor (Table 3.8)
γ_{M2} is the partial safety factor for resistance (Table 2.5)

On a fillet weld with equal legs, condition (3.22) will always be the governing one.

Table 3.8 Strength factor of the weld for different steel grades [4,5,6]

	Correlation factor of the weld β_w
S235	0,8
S275	0,85
S355	**0,9**
S420	**1,0**
S460	1,0

In welded joints of hollow section lattices local stress concentrations are developed in the region of the joint. Plastic yielding, however, evens out the stress concentrations. That is why the welds of joints are normally designed to have equal strength with the members.

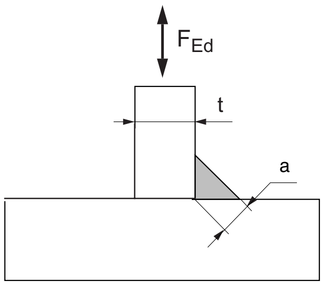
By applying condition (3.22) (on a fillet weld with equal legs condition (3.22) is always governing) and by setting the weld resistance per unit length of the perimeter of the hollow section to be at least equal to the plastic tension resistance of the wall of the hollow section $t \cdot f_y / \gamma_{M0}$, the throat thickness required for a fillet weld (single-sided and with equal legs) is derived so that the weld will have equal strength in respect to axial tension, compression and/or bending moment applied to the hollow section [17]:

$$a \geq 2 \cdot \frac{\beta_w}{\sqrt{2}} \cdot \frac{\gamma_{M2}}{\gamma_{M0}} \cdot \frac{f_y}{f_u} \cdot t \qquad (3.24)$$

where t is the wall thickness
f_y is the nominal yield strength of the material
f_u is the nominal ultimate tensile strength of the material
β_w is the appropriate strength factor (Table 3.8)
γ_{M2} and γ_{M0} are partial safety factors for resistance (Table 2.5)

Table 3.9 gives the required throat thickness for an equal strength fillet weld (having equal legs and welded around the whole perimeter of the hollow section) determined according to expression (3.24), when the hollow section is subject to axial tension, compression and/or bending moment. The weld has hereby also sufficient deformation capability. **For example, the throat thickness for the welds of the brace members in lattice structures can be chosen according to Table 3.9.**

Table 3.9 The required throat thickness for an equal strength fillet weld made around the perimeter of the hollow section which is subject to axial tension, compression and/or bending



Steel grade	Yield strength a) f_y (N/mm^2)	Ultimate tensile strength a) f_u (N/mm^2)	Throat thickness of the weld b)
S235H	235	360	0,92·t
S275H	275	430	0,96·t
S355H	**355**	**510**	**1,11·t**
S275NH	275	370	1,12·t
S355NH	355	470	1,20·t
S460NH	460	550	1,48·t
S275MH	275	360	1,15·t
S355MH	355	470	1,20·t
S420MH	**420**	**500**	**1,48·t**
S460MH	460	530	1,53·t

a) Nominal strength values according to Table 1.7.
b) However at least a ≥ 3 mm.

- The values in this table have been calculated using the recommended values $\gamma_{M0} = 1{,}0$ and $\gamma_{M2} = 1{,}25$ as given in Parts EN 1993-1-1 and EN 1993-1-8 of Eurocode. **National requirements must be checked from the National Annex of the relevant country.**
- The throat thickness values of this table are valid:
  - when the hollow section to be joined is subject to axial tension or compression and /or bending
  - when the weld is made around the perimeter of the hollow section to be joined
  - when the hollow section to be joined is of the same grade or lower grade than the adjacent member
  - when using a welding consumable having its yield strength and ultimate tensile strength at least equivalent minimum values as those of the hollow section to be joined.

3.3.3.2 Resistance of a fillet weld by using the Simplified method

The resistance of a fillet weld can be alternatively calculated using so-called Simplified method. In this case the resultant of the forces applied to the weld are supposed always to cause only shear in the throat section of the weld, no matter what is the actual direction of the force resultant and weld (a conservative simplification). From the design condition (3.22) given on the Directional method, the following formula can be derived for the design (shear) strength of the weld [4,5,6]:

$$f_{vw.d} = \frac{f_u/\sqrt{3}}{\beta_w \gamma_{M2}} \tag{3.25}$$

where f_u is the nominal ultimate tensile strength of the weakest part to be joined
β_w is the appropriate strength factor (Table 3.8)
γ_{M2} is the partial safety factor for resistance (Table 2.5)

The benefit of this method is, that the direction of the force resultant need not be known, because calculating this way it has no effect to the design resistance of the weld. Only the magnitude of the force resultant applied to the weld is needed. On the other hand the method leads slightly to oversizing, depending on the direction of the force.

The design condition for a weld is [4,5,6]:

$$F_{w.Ed} \leq F_{w.Rd} \tag{3.26}$$

where $F_{w.Ed}$ is the design value of the force acting per unit length of the weld
$F_{w.Rd}$ is the design weld resistance per unit length

The design weld resistance per unit length is calculated from the formula [4,5,6]:

$$F_{w.Rd} = f_{vw.d}\, a \tag{3.27}$$

where $f_{vw.d}$ is the design shear strength of the weld according to expression (3.25)
a is the throat thickness of the fillet weld

By applying expression (3.27) and by supposing that the force, applied to unit length of the perimeter of the wall of the hollow section to be joined, is equal to its plastic tension resistance $t \cdot f_y / \gamma_{M0}$, it is possible to derive the required throat thickness (as a conservative simplification) for an equal strength single-sided fillet weld with equal legs. Thereby the following expression for the required throat thickness will be obtained:

$$a \geq \sqrt{3}\,\beta_w \cdot \frac{\gamma_{M2}}{\gamma_{M0}} \cdot \frac{f_y}{f_u} \cdot t \tag{3.28}$$

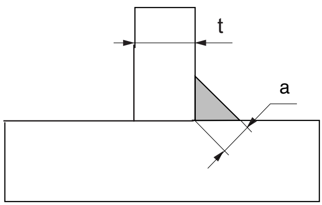
where t is the wall thickness
f_y is the nominal yield strength of the material
f_u is the nominal ultimate tensile strength of the material
β_w is the appropriate strength factor (Table 3.8)
γ_{M2} and γ_{M0} are partial safety factors for resistance (Table 2.5)

Table 3.10 gives the required throat thickness for an equal strength fillet weld (having equal legs and welded around the whole perimeter of the hollow section) determined according to expression (3.28), when the hollow section may be subject to loads causing normal stress and/or shear stress. The values of the table can be applied regardless of the direction of the resultant of the forces applied to the hollow section. The weld has hereby also sufficient deformation capability.

The throat thicknesses presented in Table 3.10 are about 22 % bigger than those in Table 3.9, which is limited to the case where the hollow section is subjected to loads causing normal stresses only. If the wall thickness of the hollow section is more than 8 mm, the required throat thickness begins to be so big that it is beneficial to consider execution of the weld according to clause 3.3.4 as a partial or full penetration butt weld, reinforced when needed with a fillet weld [17].

If only shear stress acts in the wall of the hollow section, the throat thicknesses in Table 3.10 are clearly oversized, and it is more efficient to design correspondingly also the weld for only the shear acting in the wall of the hollow section. Thereby it is possible to achieve over 40 % smaller throat thickness.

Table 3.10 The required throat thickness for an at least equal strength fillet weld made around the perimeter of the hollow section which is subject to axial tension, compression, bending and/or shear



Steel grade	Yield strength a) f_y (N/mm^2)	Ultimate tensile strength a) f_u (N/mm^2)	Throat thickness of the weld b)
S235H	235	360	$1,13 \cdot t$
S275H	275	430	$1,18 \cdot t$
S355H	**355**	**510**	**$1,36 \cdot t$**
S275NH	275	370	$1,37 \cdot t$
S355NH	355	470	$1,47 \cdot t$
S460NH	460	550	$1,81 \cdot t$
S275MH	275	360	$1,41 \cdot t$
S355MH	355	470	$1,47 \cdot t$
S420MH	**420**	**500**	**$1,82 \cdot t$**
S460MH	460	530	$1,88 \cdot t$

a) Nominal strength values according to Table 1.7.
b) However at least $a \geq 3$ mm.

- The values in this table have been calculated using the recommended values $\gamma_{M0} = 1,0$ and $\gamma_{M2} = 1,25$ as given in Parts EN 1993-1-1 and EN 1993-1-8 of Eurocode. **National requirements must be checked from the National Annex of the relevant country.**
- The throat thickness values of this table are valid:
  - regardless of the direction of the resultant of the stresses acting in the wall of the hollow section
  - when the weld is made around the perimeter of the hollow section to be joined
  - when the hollow section to be joined is of the same grade or lower grade than the adjacent member
  - when using a welding consumable having its yield strength and ultimate tensile strength at least equivalent minimum values as those of the hollow section to be joined.

3.3.4 Butt welds

Welded end-to-end splice joints of hollow sections and some of the brace member joints in hollow section lattices (see Chapter 8, Figures 8.15 - 8.18) are executed as full penetration or partial penetration butt welds.

According to EN 1993-1-8, the resistance of a full penetration butt weld may be assumed equal to the resistance of the weakest part to be joined, provided that the welding consumables to be used have their yield strength and ultimate strength at least equivalent minimum values as those of the parent metal to be welded [4,5,6].

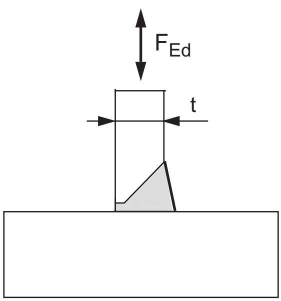
The resistance of a partial penetration butt weld shall be calculated according to clause 3.3.3 like the resistance of a penetration fillet weld. The throat thickness of a partial penetration butt weld shall be chosen to be not more than the depth of penetration that can be consistently achieved (to be proven by welding tests) [4,5,6].

a) Full penetration butt weld b) Partial penetration butt weld

Figure 3.10 Butt welds in a half V-groove

So, as stated above, in case of a full penetration butt weld Eurocode allows to assume that the resistance of the joint is equal to the resistance of the weakest part to be joined. However, if the resistance of a full penetration butt weld would be calculated using the Directional method and applying consistently the provisions given for resistance of a partial penetration weld, the outcome would be that the results obtained from the calculation model are not fully compatible with the assumption of equal strength, see Table 3.11.

Table 3.11 Theoretical resistance of a full penetration butt weld if applying calculation provisions assigned for partial penetration butt weld using the Directional method



Steel grade	Yield strength a) f_y (N/mm^2)	Ultimate tensile strength a) f_u (N/mm^2)	Resistance of the weld b)
S235H	235	360	$1{,}00 \cdot N_{pl.Rd}$
S275H	275	430	$1{,}00 \cdot N_{pl.Rd}$
S355H	**355**	**510**	$\mathbf{1{,}00 \cdot N_{pl.Rd}}$
S275NH	275	370	$0{,}97 \cdot N_{pl.Rd}$
S355NH	355	470	$0{,}95 \cdot N_{pl.Rd}$
S460NH	460	550	$0{,}86 \cdot N_{pl.Rd}$
S275MH	275	360	$0{,}94 \cdot N_{pl.Rd}$
S355MH	355	470	$0{,}95 \cdot N_{pl.Rd}$
S420MH	**420**	**500**	$\mathbf{0{,}86 \cdot N_{pl.Rd}}$
S460MH	460	530	$0{,}83 \cdot N_{pl.Rd}$

a) Nominal strength values according to Table 1.7.
b) $N_{pl.Rd} = t \cdot f_y / \gamma_{M0}$ is the plastic resistance of the hollow section wall per unit length of the perimeter of the hollow section. The design resistance is limited as maximum to the resistance $N_{pl.Rd}$ of the hollow section wall. According to EN 1993-1-8, the resistance of a full penetration butt weld can be assumed equal to the resistance of the weakest part to be joined.

- The values in this table have been calculated using the recommended values $\gamma_{M0} = 1{,}0$ and $\gamma_{M2} = 1{,}25$ as given in Parts EN 1993-1-1 and EN 1993-1-8 of Eurocode. **National requirements must be checked from the National Annex of the relevant country.**
- The values of this table are valid:
  - when the weld is made around the perimeter of the hollow section to be joined
  - when the hollow section to be joined is of the same grade or lower grade than the adjacent member
  - when using a welding consumable having its yield strength and ultimate tensile strength at least equivalent minimum values as those of the hollow section to be joined.

3.3.5 Welded joints between different steel grades

3.3.5.1 Choice of welding consumable

The welding consumable should be slightly more alloyed than the parent metal, in order to achieve in the final weld the strength and impact toughness properties comparable with those of the parent metal. In practice the welding consumable is chosen on the basis of both the steel grade and the impact strength quality class. When welding hot-rolled steels, it is recommended to use a welding consumable with similar or a little higher (5...10 %) strength than the parent metal [18].

In Eurocode nor in EN 1090-2, there are no instructions for the choice of welding consumable in a case, where steels of different grades are to be joined. In such case the welding consumable is, however, normally chosen according to the lower grade, unless there is a specific reason to determine otherwise [16].

In case of a full penetration butt weld, when materials with the same thickness but different grades are joined, it is obvious that the resistance of the lower grade material governs the resistance of the whole joint (provided the welding consumable is chosen to have at least the same strength as the concerned parent metal). The resistance can not be increased, even though the welding consumable were chosen according to the stronger parent metal.

The same holds true also in regard to a fillet weld: in the design according to Eurocode the theoretical resistance of the joint can not be increased, even though the welding consumable were chosen according to the stronger parent metal.

On the other hand, if using unnecessarily strong (= alloyed) welding consumable, high residual stress state and increased risk to distortion and cracking is produced as a consequence.

3.3.5.2 Design of welds between different steel grades

3.3.5.2.1 Fillet welds

The resistance of a fillet weld between different grades shall be calculated according to the provisions presented in clause 3.3.3. When determining the resistance of the weld, the ultimate tensile strength f_u applied in expressions (3.22) - (3.28) shall always be chosen according to the <u>weaker parent metal</u>, and the strength factor β_w of the corresponding steel grade in Table 3.8 shall be applied [4,5,6].

Table 3.12 gives the required throat thickness (determined by the Directional method according to expression (3.24) presented in clause 3.3.3.1) for a tube-to-plate joint between different steel grades conforming to an equal strength fillet weld (having equal legs and welded around the whole perimeter of the hollow section) in respect to the concerned hollow section, when the hollow section is subject to axial tension, compression and / or bending moment. For example, the throat thickness for an equal strength fillet weld between a hollow section column and base plate can be chosen according to Table 3.12.

Table 3.13 gives the required throat thickness (determined by the Simplified method according to expression (3.28) presented in clause 3.3.3.2) for a tube-to-plate joint between different steel grades conforming to an equal strength fillet weld (having equal legs and welded around the whole perimeter of the hollow section) in respect to the concerned hollow section, when the hollow section may be subject to loads causing normal stress and / or shear stress. The values of the table can be used regardless of the direction of the resultant of the forces applied to the plate.

It can be noticed, that when looking at Table 3.12 and Table 3.13 as well, there may be some illogicality in the throat thickness values between different steel grades. This is due to the discontinuity caused by the grade specific strength factor β_w defined in Eurocode.

Table 3.12 Welds between different steel grades. The required throat thickness for an equal strength fillet weld made around the perimeter of the hollow section which is subject to axial tension, compression and / or bending

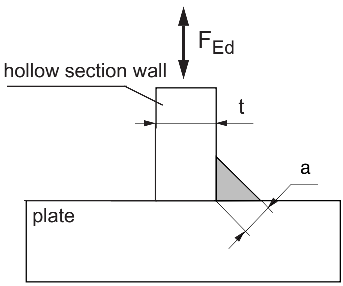


Plate		Throat thickness of the weld [b)]	
Steel grade	Ultimate tensile strength [a)] f_u (N/mm^2)	Structural hollow section S355H	Structural hollow section S420MH
S235	360	1,39·t	1,65·t
S275	430	1,24·t	1,48·t
S355	**490**	**1,15·t**	**1,36·t**
S275N	390	1,37·t	1,62·t
S355N	490	1,15·t	1,36·t
S420N	520	1,11·t	1,48·t
S460N	540	1,11·t	1,48·t
S275M	370	1,44·t	1,71·t
S355M	470	1,20·t	1,42·t
S420M	520	1,11·t	1,48·t
S460M	540	1,11·t	1,48·t

a) Nominal values of the ultimate tensile strength presented in EN 1993-1-1 for flat steels in grades S235 - S460 conforming to EN 10025, when $t \leq 40$ mm.
b) However at least $a \geq 3$ mm.

- The values in this table have been calculated using the recommended values $\gamma_{M0} = 1{,}0$ and $\gamma_{M2} = 1{,}25$ as given in Parts EN 1993-1-1 and EN 1993-1-8 of Eurocode. **National requirements must be checked from the National Annex of the relevant country.**
- The throat thickness values of this table are valid:
  - when the hollow section to be joined is subject to axial tension or compression and / or bending
  - when the weld is made around the perimeter of the hollow section to be joined
  - when using a welding consumable having its yield strength and ultimate tensile strength at least equivalent minimum values as those of the weaker part to be joined.

Table 3.13 Welds between different steel grades. The required throat thickness for an at least equal strength fillet weld made around the perimeter of the hollow section which is subject to axial tension, compression, bending and / or shear

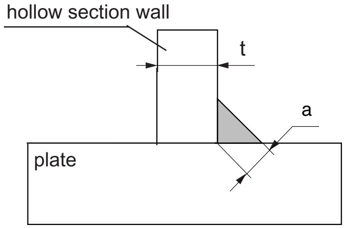


Plate		Throat thickness of the weld [b)]	
Steel grade	Ultimate tensile strength [a)] f_u (N/mm^2)	Structural hollow section S355H	Structural hollow section S420MH
S235	360	1,71·t	2,02·t
S275	430	1,52·t	1,80·t
S355	**490**	**1,41·t**	**1,67·t**
S275N	390	1,68·t	1,98·t
S355N	490	1,41·t	1,67·t
S420N	520	1,36·t	1,82·t
S460N	540	1,36·t	1,82·t
S275M	370	1,77·t	2,09·t
S355M	470	1,47·t	1,74·t
S420M	520	1,36·t	1,82·t
S460M	540	1,36·t	1,82·t

a) Nominal values of the ultimate tensile strength presented in EN 1993-1-1 for flat steels in grades S235 - S460 conforming to EN 10025, when t ≤ 40 mm.
b) However at least a ≥ 3 mm.

- The values in this table have been calculated using the recommended values $\gamma_{M0} = 1{,}0$ and $\gamma_{M2} = 1{,}25$ as given in Parts EN 1993-1-1 and EN 1993-1-8 of Eurocode. **National requirements must be checked from the National Annex of the relevant country.**
- The throat thickness values of this table are valid:
  - regardless of the direction of the resultant of the stresses acting in the wall of the hollow section
  - when the weld is made around the perimeter of the hollow section to be joined
  - when using a welding consumable having its yield strength and ultimate tensile strength at least equivalent minimum values as those of the weaker part to be joined.

3.3.5.2.2 Butt welds

The resistance of a partial penetration butt weld between different grades shall be calculated according to the provisions presented in clause 3.3.4. When determining the resistance of the weld, the ultimate tensile strength f_u applied in expressions (3.22) - (3.28) shall always be chosen according to the weaker parent metal, and the strength factor β_w of the corresponding steel grade in Table 3.8 shall be applied [4,5,6].

According to EN 1993-1-8, the resistance of a full penetration butt weld may be assumed to be equal to the resistance of the weakest part to be joined, provided that the welding consumables to be used have their yield strength and ultimate strength at least equivalent minimum values as those of the parent metal to be welded [4,5,6].

Table 3.14 gives the resistance of a full penetration butt weld for a tube-to-plate joint between different steel grades in respect to tube's own plastic tension resistance $t \cdot f_y / \gamma_{M0}$, if determined by the Directional method according to provisions presented for partial penetration butt weld (see clauses 3.3.4 and 3.3.3.1).

Table 3.14 Welds between different steel grades. Theoretical resistance of a full penetration butt weld if applying calculation provisions assigned for partial penetration butt weld using the Directional method

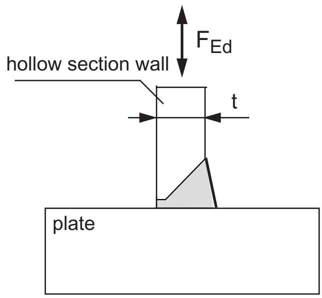


Plate		Resistance of the weld b)	
Steel grade	Ultimate tensile strength a) f_u (N/mm^2)	Structural hollow section S355H	Structural hollow section S420MH
S235	360	$0{,}66 \cdot N_{pl.Rd}$	$0{,}56 \cdot N_{pl.Rd}$
S275	430	$0{,}77 \cdot N_{pl.Rd}$	$0{,}65 \cdot N_{pl.Rd}$
S355	**490**	$\mathbf{0{,}99 \cdot N_{pl.Rd}}$	$\mathbf{0{,}84 \cdot N_{pl.Rd}}$
S275N	390	$0{,}77 \cdot N_{pl.Rd}$	$0{,}65 \cdot N_{pl.Rd}$
S355N	490	$0{,}99 \cdot N_{pl.Rd}$	$0{,}84 \cdot N_{pl.Rd}$
S420N	520	$1{,}00 \cdot N_{pl.Rd}$	$0{,}86 \cdot N_{pl.Rd}$
S460N	540	$1{,}00 \cdot N_{pl.Rd}$	$0{,}86 \cdot N_{pl.Rd}$
S275M	370	$0{,}75 \cdot N_{pl.Rd}$	$0{,}63 \cdot N_{pl.Rd}$
S355M	470	$0{,}95 \cdot N_{pl.Rd}$	$0{,}81 \cdot N_{pl.Rd}$
S420M	**520**	$\mathbf{1{,}00 \cdot N_{pl.Rd}}$	$\mathbf{0{,}86 \cdot N_{pl.Rd}}$
S460M	540	$1{,}00 \cdot N_{pl.Rd}$	$0{,}86 \cdot N_{pl.Rd}$

a) Nominal values of the ultimate tensile strength presented in EN 1993-1-1 for flat steels in grades S235-S460 conforming to EN 10025, when $t \leq 40$ mm.
b) $N_{pl.Rd} = t \cdot f_y / \gamma_{M0}$ is the plastic resistance of the hollow section wall per unit length of the perimeter of the hollow section. The design resistance is limited as maximum to the resistance $N_{pl.Rd}$ of the hollow section wall. According to EN 1993-1-8, the resistance of a full penetration butt weld can be assumed equal to the resistance of the weakest part to be joined.

- The values in this table have been calculated using the recommended values $\gamma_{M0} = 1{,}0$ and $\gamma_{M2} = 1{,}25$ as given in Parts EN 1993-1-1 and EN 1993-1-8 of Eurocode. **National requirements must be checked from the National Annex of the relevant country.**
- The values of this table are valid:
  - when the weld in made around the perimeter of the hollow section to be joined
  - when using a welding consumable having its yield strength and ultimate tensile strength at least equivalent minimum values as those of the weaker part to be joined.

3.4 Bolted hollow section joints

3.4.1 End-to-end bolted joints

It is usually favourable to manufacture the steel components in the workshop by welding, and then connect these components on site with bolted joints. Bolted joints are quicker and easier to prepare at on-site conditions than welded joints. Various versions for hollow section bolted joints are presented in Figure 3.11.

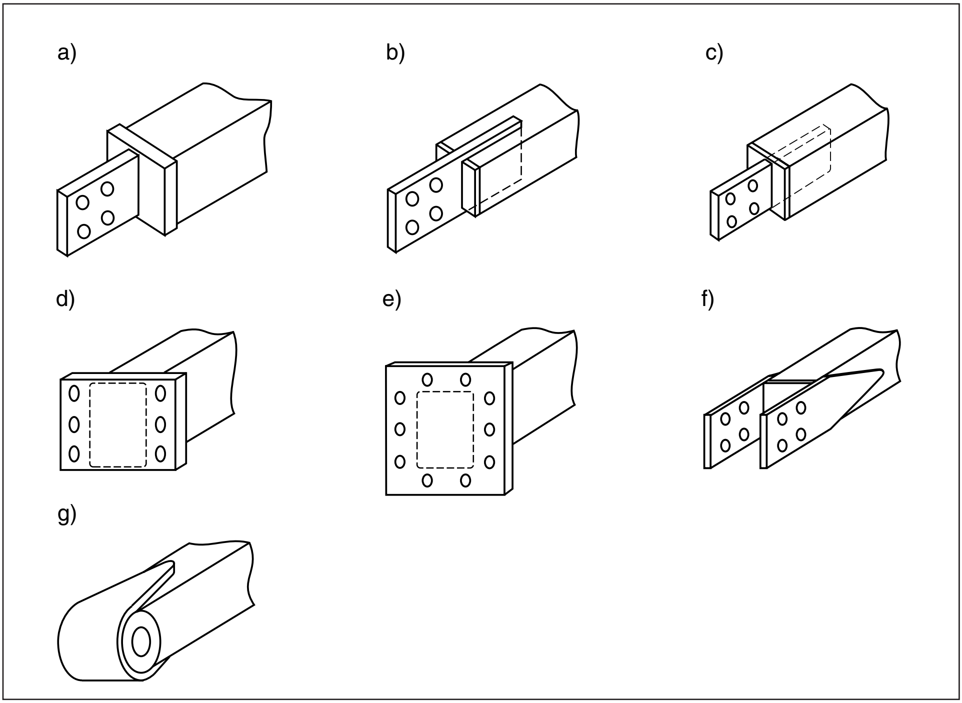


Figure 3.11 Alternatives for end-to-end joints of hollow sections

When designing a joint, it is essential to ensure that the load is as concentric as possible in relation to the cross-section and that the rigidity of the joint components is uniform. In this respect, a tension joint is best constructed using the versions b, c, f or g. In these joint types the tension load is transmitted through a more direct path to the hollow section, than in joints a, d or e, which also include the risk of lamellar tearing. In flange-plate joints d and e, the thickness of the flange-plate should be chosen big enough to keep small the bolt prying forces due to flange elasticity.

3.4.1.1 End-to-end bolted joints using flange-plates

In Eurocode there are no instructions for the flange-plate joints for hollow sections. The Eurocode-compatible instructions presented herein are based on [12,13,19].

3.4.1.1.1 Flange-plate tension joint of square and rectangular hollow sections

A flange-plate joint can be designed by modelling it as a two-dimensional joint of T elements, where the bolts are placed at opposite sides of the hollow section (Figure 3.12). Due to tension load, a plastic hinge is developed in the flange-plate at hollow section walls (Figure 3.13). The tension resistance of the flange can be determined using the plastic moment.

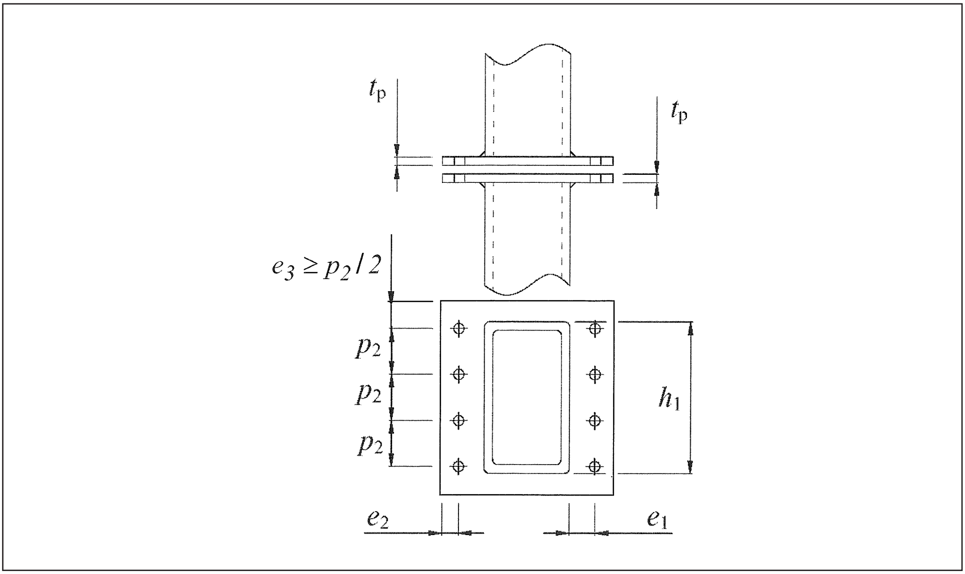


Figure 3.12 End-to-end joint with flange-plates, when having square or rectangular hollow sections

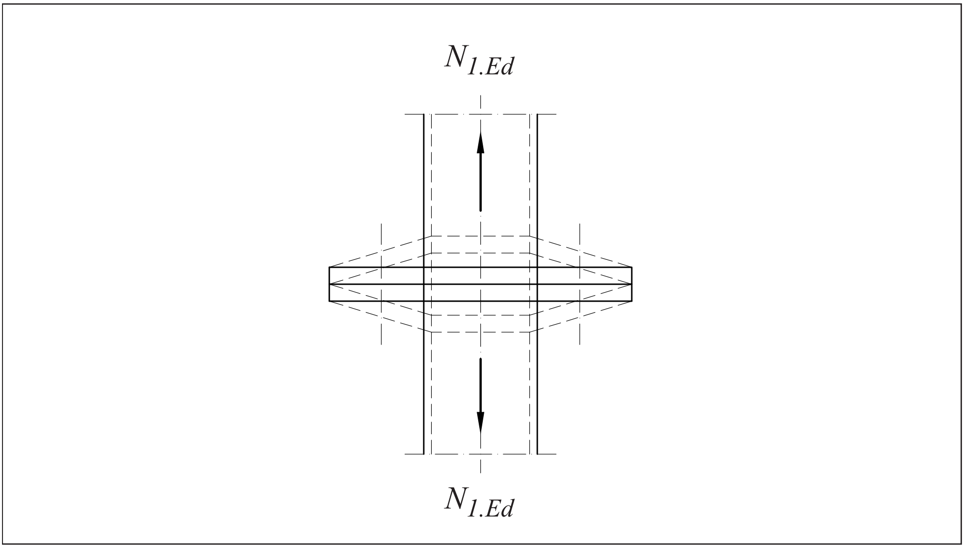


Figure 3.13 Structural model of a flange-plate joint

The calculation model for a flange-plate joint (Figure 3.13) is in principal similar with the so-called *equivalent T-stub* model applied for flange-plate joints of I-sections, where the failure mode is the bolt failure with simultaneous yielding of the flange-plate. The T-stub calculation model for I-sections does not, however, fully apply for hollow sections where e.g. the plastic hinges are located in slightly different places (as shown by the tests) than in the flange-plate joints of I-sections.

When calculating the forces and resistances of the bolts, the additional loads due to the prying forces need not be separately taken into account, because they are already implicitly included in the calculating model in the same way, as in the equivalent T-stub calculation model for the flange-plate joints of I-sections in EN 1993-1-8. However if wanted, the effect of prying forces can be separately calculated as presented later on.

The applied calculation method is semi-empirical, and it has been tested for the flange-plate thicknesses 12...26 mm. The method can be used, when the joint fulfills the following conditions [19]:

- the bolt rows are positioned at uniform spacing on two opposite sides of the flange-plate joint according to Figure 3.12

- the number of bolts is: $4 \leq n \leq 2(h_1 / p_2) + 2$

- the nominal clearance of bolt holes is as specified in EN 1090-2 for normal round holes

- positioning of the holes (Figure 3.12):
  $e_1 \geq 1{,}2 d_0$, where d_0 = diameter of the bolt hole
  $e_2 \geq 1{,}2 d_0$ and $e_2 \leq 1{,}25 e_1$
  $e_3 \geq p_2 / 2$, the distance exceeding the lower limit shall not be, however, exploited in the resistance of the flange-plate
  $2{,}4 d_0 \leq p_2 \leq \min[14 t_p \text{ ; } 200 \text{ mm}]$, recommendation: $p_2 = (3...5) \times d_0$

To minimize the prying forces and to maximize the resistance of the joint in return, it is advisable to keep the distance e_1 as small as possible so that $e_1 = (1{,}5 \ ... \ 2) \times d_0$ (however, at least 5 mm shall be left between the bolt head and the weld in the flange-plate), and for the distance e_2 the value $e_2 \approx 1{,}25 e_1$ should be chosen [19].

The calculation of the resistance of a flange-plate joint begins with determination of the factor δ which represents the relative net area of the bolt row [19]:

$$\delta = 1 - \frac{d_0}{p_2} \qquad (3.29)$$

where d_0 is the diameter of the bolt hole
p_2 is the spacing between centres of the holes

An auxiliary variable K related to the plastic moment of the flange-plate is calculated as follows [19]:

$$K = \frac{4b'}{0,9 \cdot (f_{yp}/\gamma_{M0}) \cdot p_2} \tag{3.30}$$

$$b' = e_1 - 0,5d + t_1 \qquad \textit{the lever arm of the bolt row to plastic hinge} \tag{3.31}$$

where e_1 is the distance of the hole centre from the edge of the hollow section (Figure 3.12)
d is the nominal diameter of the bolt
t_1 is the wall thickness of the hollow section
p_2 is the spacing between centres of the holes (Figure 3.12)
f_{yp} is the nominal yield strength of the flange-plate
γ_{M0} is the partial safety factor for resistance (Table 2.5)

The thickness of the flange-plate t_p shall be chosen between the following minimum and maximum values [19]:

$$\sqrt{\frac{K \cdot F_{t.Ed}}{1+\delta}} \le t_p \le \sqrt{K \cdot F_{t.Ed}} \tag{3.32}$$

where $F_{t.Ed}$ is the normal force per one bolt (= $N_{1.Ed}/n$)

When calculating the resistance of the joint at ultimate limit state, the tension force acting in the bolts $F_{t.Ed}$ in expression (3.32), is supposed to be the tension resistance of the bolts $F_{t.Rd}$ (to be presented later on).

A parameter α_{Rd} is calculated taking into account the influence of the holes to the plastic moment of the flange-plate at bolt-line in relation to the plastic moment at the wall of the hollow section, when the tension force in bolts is supposed to be equal to the tension resistance of the bolts [19]:

$$\alpha_{Rd} = \left(\frac{K \cdot F_{t.Rd}}{t_p^2} - 1\right) \cdot \left[\frac{e_2 + 0,5d}{\delta \cdot (e_2 + e_1 + t_1)}\right] \ge 0 \tag{3.33}$$

The resistance of the joint is derived by setting equal the work in plastic hinges and the work done by external load [19]:

$$N_{1.Rd} = \frac{t_p^2(1 + \delta\alpha_{Rd})n}{K} \tag{3.34}$$

where n is the number of bolts

Partial safety factor γ_{M2}, which has been incorrectly written in the corresponding expression in [19], has been left away from expression (3.34) (the preceding expression for factor K includes already the needed safety margin both in implicit and explicit way).

Finally the following design conditions shall be checked [19]:

$$N_{1.Ed} \leq N_{1.Rd} \quad \textit{resistance of the joint} \tag{3.35}$$

$$N_{1.Ed} \leq n \cdot F_{t.Rd} \quad \textit{tension resistance of the bolts} \tag{3.36}$$

$$N_{1.Ed} \leq n \cdot B_{p.Rd} \quad \textit{punching shear resistance of the bolts} \tag{3.37}$$

Additionally the resistance of the weld between the hollow section and the flange-plate shall be checked according to clause 3.3.

The tension resistance of the bolt $F_{t.Rd}$ and the punching shear resistance $B_{p.Rd}$ in expressions (3.32) - (3.37) are calculated from the following expressions [4,5,6]:

$$F_{t.Rd} = \frac{k_2 f_{ub} A_s}{\gamma_{M2}} \tag{3.38}$$

$$B_{p.Rd} = 0,6 \pi d_m t_p f_u / \gamma_{M2} \tag{3.39}$$

where k_2 = 0,63 for countersunk bolts
k_2 = 0,90 for other bolts
f_{ub} is the nominal ultimate tensile strength of the bolt
A_s is the tensile stress area of the bolt (area in the threaded portion)
d_m the punching shear diameter, which is the smaller of the following values:
- the mean of the across points and across flats dimensions of the bolt head, or
- the mean of the across points and across flats dimensions of the nut

t_p is the thickness of the flange-plate
f_u is the nominal ultimate tensile strength of the plate material (flange-plate)
γ_{M2} is the partial safety factor for resistance (Table 2.5)

The expression for punching shear resistance (3.39) is not valid for countersunk bolts.

If wanted, the total force in the bolts including also the effect due to prying forces can be calculated as follows (as stated earlier, the prying forces need not be separately taken into account because their influence is already implicitly included in the calculation of the joint resistance presented above) [13]:

$$F_{b.Ed} = F_{t.Ed} \cdot \left[1 + \frac{b'}{a'} \cdot \frac{\delta \alpha_{Ed}}{1 + \delta \alpha_{Ed}} \right] \tag{3.40}$$

$$a' = e_2 + 0,5d \tag{3.41}$$

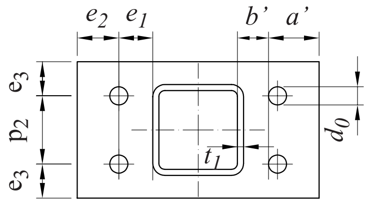
$$\alpha_{Ed} = \left(\frac{K \cdot F_{t.Ed}}{t_p^2} - 1 \right) \cdot \frac{1}{\delta} \tag{3.42}$$

where $F_{b.Ed}$ is the total bolt force per one bolt including the prying force

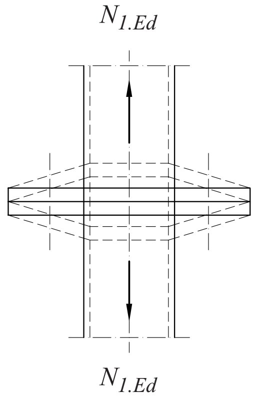
The total force acting in the bolt $F_{b.Ed}$ shall be smaller than the tension resistance of the bolt $F_{t.Rd}$ and the punching shear resistance $B_{p.Rd}$.

Example 3.13

Calculate the tension resistance of the flange-plate joint in the figure. The thickness of the flange-plate is 35 mm and the steel grade is S355J2. The bolts are M24 in class 10.9.



The hollow section is 120×120×8. The steel grade is SSAB Domex Tube Double Grade, which fulfills the EN 10219 requirements for both steel grades S420MH and S355J2H. Thereby the design calculations may be performed at designer's own choice either according to grade S420 or grade S355. Grade S420 is chosen in this Example as design basis.



A_1 $= 3364\ mm^2$ *(Annex 11.1)*
f_y $= 420\ N/mm^2$ *yield strength (hollow section)*
f_{yp} $= 355\ N/mm^2$ *yield strength (plate)*
f_{up} $= 490\ N/mm^2$ *ultimate tensile strength (plate)*
f_{ub} $= 1000\ N/mm^2$ *ultimate tensile strength (bolts)*
$\gamma_{M0} = 1{,}0$
$\gamma_{M2} = 1{,}25$

The parameters defining the positioning of the holes in the flange-plate:
$d = 24\ mm$
$d_0 = 26\ mm$
$p_2 = 90\ mm$
$e_1 = 45\ mm$
$e_2 = 55\ mm$
$e_3 = 45\ mm$

Check the conditions for the positioning of the holes:
$e_1 = 45\ mm \geq 1{,}2d_0 = 31{,}2\ mm$ *OK*
$e_2 = 55\ mm \geq 1{,}2d_0 = 31{,}2\ mm$ *OK*
$e_2 = 55\ mm \leq 1{,}25e_1 = 56{,}3\ mm$ *OK*
$e_3 = 45\ mm \geq p_2 / 2 = 45\ mm$ *OK*
$p_2 = 90\ mm \geq 2{,}4d_0 = 62{,}4\ mm$ *OK*
$p_2 = 90\ mm \leq min[14t_p\ ;\ 200\ mm] = min[490\ mm\ ;\ 200\ mm] = 200\ mm$ *OK*

Check the condition for the number of bolts:

$4 \leq n = 4 \leq 2(h_1/p_2) + 2 = 2 \cdot (120/90) + 2 = 4,7$ *OK*

Tension resistance of the cross-section of the hollow section:

$$N_{t.Rd} = \frac{Af_y}{\gamma_{M0}} = \frac{3364 \cdot 420}{1,0} = 1413\ kN$$

Resistance of the welds:
Design the throat thickness of the weld to have at least equal strength with the plastic tension resistance of the hollow section $N_{t.Rd}$, when the hollow section is welded to flange-plate of grade S355J2 (Table 3.12):

$$a \geq 1,36 \cdot t_1 = 1,36 \cdot 8 = 10,9\ mm$$

$\Rightarrow$ *choose* $a = 11\ mm$

Resistance of the bolts:
The tension resistance per bolt:

$A_s = 353\ mm^2$ *tensile stress area of a M24 bolt (area in the threaded portion)*

$$F_{t.Rd} = \frac{k_2 f_{ub} A_s}{\gamma_{M2}} = \frac{0,9 \cdot 1000 \cdot 353}{1,25} = 254,2\ kN$$

The punching shear resistance per bolt:

$d_m = 37,8\ mm$ *M24 bolt, punching shear diameter of the nut or the bolt head*

$$B_{p.Rd} = 0,6\pi d_m t_p f_u / \gamma_{M2} = 0,6 \cdot \pi \cdot 37,8 \cdot 35 \cdot 490 / 1,25 = 977,6\ kN$$

$\Rightarrow$ *the resistance of the bolts is governed by the tension resistance 254,2 kN*

Resistance of the flange-plate joint:
Calculate the auxiliary parameters needed:

$$\delta = 1 - \frac{d_0}{p_2} = 1 - \frac{26}{90} = 0,7111$$

$$b' = e_1 - 0,5d + t_1 = 45 - 0,5 \cdot 24 + 8 = 41,0\ mm$$

$$K = \frac{4b'}{0,9 \cdot (f_{yp}/\gamma_{M0}) \cdot p_2} = \frac{4 \cdot 41,0}{0,9 \cdot (355/1,0) \cdot 90} = 5,703 \cdot 10^{-3}\ mm^2/N$$

Check the conditions regarding the thickness of the flange-plate:
The minimum and maximum values for the thickness of the flange-plate are calculated by the formula:

$$\sqrt{\frac{K \cdot F_{t.Ed}}{1 + \delta}} \leq t_p \leq \sqrt{K \cdot F_{t.Ed}}$$

Since the force applied to a bolt is here unknown, and the task is to determine the joint's resistance at ultimate limit state, the force acting in a bolt $F_{t.Ed}$ shall be assumed to be equal to the tension resistance of the bolt $F_{t.Rd}$ = 254,2 kN.

Thereby the minimum and maximum values for the thickness of the flange-plate will be:

$$\sqrt{\frac{5,703 \cdot 10^{-3} \cdot 254,2 \cdot 10^3}{1 + 0,7111}} \leq t_p \leq \sqrt{5,703 \cdot 10^{-3} \cdot 254,2 \cdot 10^3}$$

$$29,1\ mm \leq t_p\ mm \leq 38,1\ mm$$

$\Rightarrow t_p = 35\ mm = OK$

The resistance of the joint is finally:

$$\alpha_{Rd} = \left(\frac{K \cdot F_{t.Rd}}{t_p^2} - 1\right) \cdot \left[\frac{e_2 + 0,5d}{\delta \cdot (e_2 + e_1 + t_1)}\right]$$

$$= \left(\frac{5,703 \cdot 10^{-3} \cdot 254,2 \cdot 10^3}{35^2} - 1\right) \cdot \left[\frac{55 + 0,5 \cdot 24}{0,7111 \cdot (55 + 45 + 8)}\right] = 0,1600 \geq 0$$

$$N_{1.Rd} = \frac{t_p^2(1 + \delta\alpha_{Rd})n}{K} = \frac{35^2 \cdot (1 + 0,7111 \cdot 0,1600) \cdot 4}{5,703 \cdot 10^{-3}} = 957,0\ kN$$

Resistance of the whole joint:
The resistance of the whole joint is governed by the smallest of the above obtained results, which is the resistance of the flange-plate joint 957 kN.

Effect due to prying forces:
It is not necessary to explicitly calculate the prying forces, because the above calculated resistance of the flange-plate joint $N_{1.Rd}$ *already implicitly takes into account the prying forces, too. Nevertheless, the total force acting in a bolt* $F_{b.Ed}$ *due to the prying forces can be calculated when assuming that the flange-plate joint is subjected to a force equal to its resistance* $N_{1.Ed} = N_{1.Rd} = 957{,}0$ *kN, whereby the normal force per one bolt is 957,0 kN / 4 = 239,3 kN:*

$$\alpha_{Ed} = \left(\frac{K \cdot F_{t.Ed}}{t_p^2} - 1\right) \cdot \frac{1}{\delta} = \left(\frac{5,703 \cdot 10^{-3} \cdot 239,3 \cdot 10^3}{35^2} - 1\right) \cdot \frac{1}{0,7111} = 0,1604$$

$$a' = e_2 + 0,5d = 55 + 0,5 \cdot 24 = 67,0\ mm$$

$$F_{b.Ed} = F_{t.Ed} \cdot \left[1 + \frac{b'}{a'} \cdot \frac{\delta\alpha_{Ed}}{1 + \delta\alpha_{Ed}}\right] = 239,3 \cdot \left[1 + \frac{41}{67} \cdot \frac{0,7111 \cdot 0,1604}{1 + 0,7111 \cdot 0,1604}\right] = 254,3\ kN$$

$$F_{b.Ed} = 254,3\ kN \leq F_{t.Rd} = 254,2\ kN \quad OK$$

$$F_{b.Ed} = 254,3\ kN \leq B_{p.Rd} = 977,6\ kN \quad OK$$

It can be seen, that when the flange-plate joint is subjected to a normal force which is equal to its resistance $N_{1.Rd} = 957{,}0$ *kN, the total bolt force, including also the prying forces, is in this Example* $F_{b.Ed} = 1{,}06 \times F_{t.Ed} = 254{,}3$ *kN, being in practice equal to the bolt's tension resistance* $F_{t.Rd}$ *(if having different initial joint data, the portion of the prying forces could be significantly higher).*

In other words, when the thickness of the flange-plate has been chosen within the minimum and maximum values calculated from expression (3.32), the resistance of the flange-plate joint is reached by the same normal force applied to the hollow section $N_{1.Ed}$*, with which the total bolt force* $F_{b.Ed}$ *reaches the bolt's tension resistance.*

Thus, by this Example we can see that the total force in the bolt will not be exceeded even if the effect due to prying forces was not separately checked. In other words, the effect due to prying forces is implicitly included, as stated in the preceding design guidance.

Comparison S420 vs S355:
In case the design calculations would be performed according to grade S355 in respect to the hollow section, the resistance of the flange-plate joint would still remain the same 957 kN. But as it comes to the weld, a slightly smaller throat thickness would be sufficient: $a \geq 1,15 \cdot t = 9,2$ *mm* $\Rightarrow$ $a = 10$ *mm (Table 3.12). However, increase of the hollow section's material strength S355 → S420 does not improve the resistance of the flange-plate joint in this Example.*

3.4.1.1.2 Flange-plate tension joint of circular hollow sections

A flange-plate joint between circular hollow sections and the symbols for its dimensions is presented in Figure 3.14. The calculation method is similar to that of rectangular hollow sections in the sense that additional forces due to prying forces need not either here be separately taken into account when calculating the loading and resistance of the bolts, since prying forces are implicitly included in the calculation model.

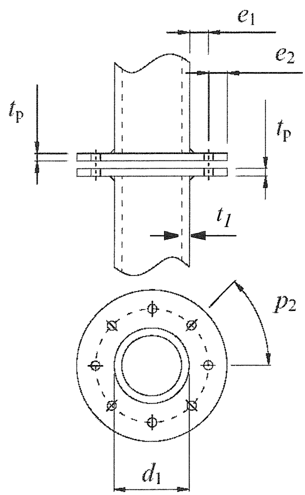


Figure 3.14 End-to-end joint with flange-plates, when having circular hollow sections

The calculation method can be used, when the joint fulfills the following conditions [19]:

- the bolts are positioned in the flange-plate at uniform spacing around the hollow section
- the number of bolts is: $n \geq 4$
- the nominal clearance of bolt holes is as specified in EN 1090-2 for normal round holes
- positioning of the holes (Figure 3.14):
  $e_1 \geq 1,2 d_0$, where d_0 = diameter of the bolt hole
  $e_2 \geq 1,2 d_0$ and $e_2 \leq 1,25 e_1$
  $2,4 d_0 \leq p_2 \leq \min[14 t_p \text{ ; } 200 \text{ mm}]$

To minimize the prying forces and to maximize the resistance of the joint in return, it is advisable to keep the distance e_1 as small as possible so that e_1 = (1,5 ... 2) $\times d_0$ (however, at least 5 mm shall be left between the bolt head and the weld in the flange-plate), and for the distance e_2 the value $e_2 \approx 1{,}25e_1$ should be chosen [19].

The design conditions for the joint are [19]:

$$N_{1.Ed} \leq \frac{t_p^2 f_{yp} \pi f_3}{2 \cdot \gamma_{M0}} \quad \text{complete yielding of the flange-plate} \qquad (3.43)$$

$$N_{1.Ed} \leq \frac{n \cdot 0{,}67 F_{t.Rd}}{1 - \dfrac{1}{f_3} + \dfrac{1}{f_3 \ln(r_1/r_2)}} \quad \text{bolt failure with simultaneous yielding of the flange-plate} \qquad (3.44)$$

$$N_{1.Ed} \leq n \cdot F_{t.Rd} \quad \text{tension resistance of the bolts} \qquad (3.45)$$

$$N_{1.Ed} \leq n \cdot B_{p.Rd} \quad \text{punching shear resistance of the bolts} \qquad (3.46)$$

where $N_{1.Ed}$ is the design value of the normal force acting in the hollow section at ultimate limit state
$F_{t.Rd}$ is the tension resistance of one bolt according to expression (3.38)
$B_{p.Rd}$ is the punching shear resistance of one bolt according to expression (3.39)
n_p is the number of bolts
t_p is the thickness of the flange-plate
f_{yp} is the nominal yield strength of the flange-plate
γ_{M0} is the partial safety factor for resistance (Table 2.5)

Additionally the resistance of the weld between the hollow section and the flange-plate shall be checked according to clause 3.3.

Expression (3.44) above has been supplemented with reduction factor 0,67 (which is missing in [19]) with which the effect of the prying forces is taken into account [12].

The parameters needed in expressions (3.43) - (3.44) are calculated as follows [19]:

$$f_3 = \frac{1}{2k_1}\left(k_3 - \sqrt{k_3^2 - 4k_1}\right) \qquad (3.47)$$

$$k_1 = \ln(r_1/r_3) \qquad (3.48)$$

$$k_3 = k_1 + 2 \qquad (3.49)$$

$$r_1 = \frac{d_1}{2} + e_1 + e_{eff} \qquad (3.50)$$

$$r_2 = \frac{d_1}{2} + e_1 \qquad (3.51)$$

$$r_3 = \frac{d_1 - t_1}{2} \qquad (3.52)$$

$$e_{eff} = \min[e_2\ ;\ 1{,}25e_1] \qquad (3.53)$$

The dimensionless factor f_3 is alternatively obtained from Figure 3.15.

The required thickness for the flange-plate and number of bolts can be derived directly from the preceding expressions (3.43) - (3.44):

$$t_p \geq \sqrt{\frac{2 \cdot N_{1.Ed} \cdot \gamma_{M0}}{f_{yp} \pi f_3}} \tag{3.54}$$

$$n \geq \frac{N_{1.Ed}}{0,67F_{t.Rd}} \cdot \left[1 - \frac{1}{f_3} + \frac{1}{f_3 ln(r_1/r_2)}\right] \tag{3.55}$$

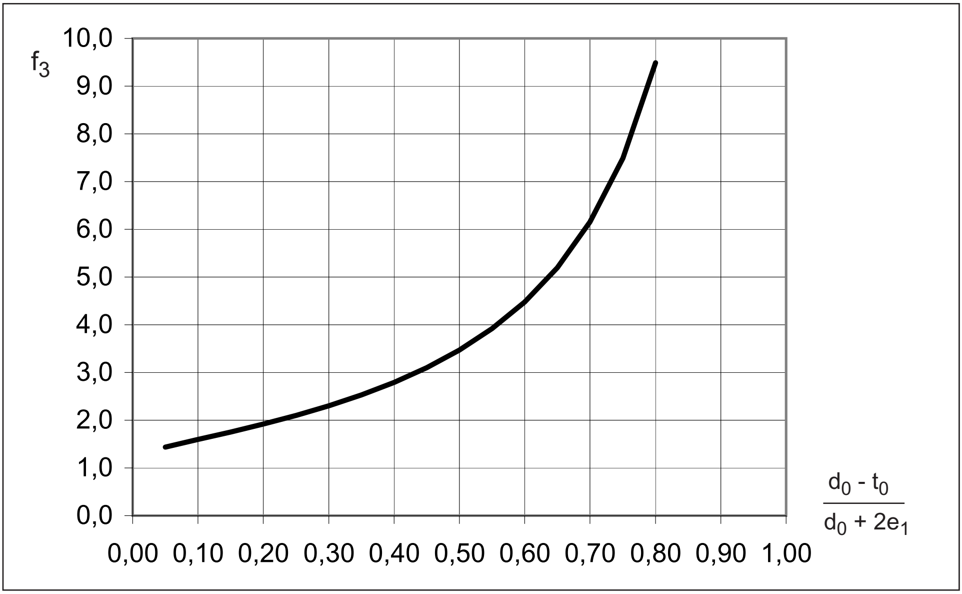


Figure 3.15 Geometry-dependent parameter f_3 for a flange-plate joint on circular hollow sections

3.4.1.2 In-line tension joint with splice plates

An in-line tension joint applies well as a splice joint of the bottom chord in a lattice structure, because the loading direction is parallel to the plates. When the joint is executed as in Figure 3.16, there is no risk of lamellar tearing of the splice plates.

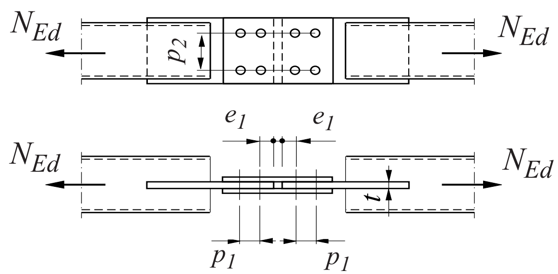


Figure 3.16 In-line tension joint with splice plates

The resistance of the joint is determined by calculating separately the resistance of the bolts and the resistance of the splice plates. The bolts transmit the force acting in the joint by their shear strength. The shear resistance of a bolt per one shear plane is calculated as follows [4,5,6]:

- when the shear plane passes through the threaded portion of the bolt:

$$F_{v.Rd} = 0,6 f_{ub} A_s / \gamma_{M2} \qquad \textit{for bolt classes 4.6 , 5.6 , 8.8} \qquad (3.56)$$

$$F_{v.Rd} = 0,5 f_{ub} A_s / \gamma_{M2} \qquad \textit{for bolt classes 4.8 , 5.8 , 6.8 , 10.9} \qquad (3.57)$$

- when the shear plane passes through the unthreaded portion of the bolt:

$$F_{v.Rd} = 0,6 f_{ub} A / \gamma_{M2} \qquad \textit{for all bolt classes} \qquad (3.58)$$

where $F_{v.Rd}$ is the shear resistance of the bolt per shear plane
f_{ub} is the ultimate tensile strength of the bolt
A_s is the tensile stress area of the bolt (area in the threaded portion)
A is the cross-section area of the bolt
in the unthreaded portion of the bolt
γ_{M2} is the partial safety factor for resistance (Table 2.5)

In important load carrying shear joints it is always recommendable to use partially-threaded bolts so that the shear plane passes through the unthreaded portion of the bolt. In secondary joints (e.g. when joining the stairs of a stairway to its side girder), fully threaded bolts can be used.

If it may occur, that due to the tolerances of the bolts and the tolerances of the parts to be joined, there is a risk that the shear plane may pass through the threaded portion of the bolt, it is reasonable to make the calculations already in the beginning by an assumption that the threads shall be in the shear plane.

In respect to the splice plates the following shall be checked:

- the tension resistance of the net cross-section $N_{t.Rd}$
- the bearing resistance of the bolted joint $F_{b.Rd}$
- the block tearing resistance of the bolted joint $V_{eff.Rd}$

The tension resistance of the net cross-section of the splice plate can be determined using the same principle as the tension resistance of the hollow section (clause 2.5.1).

The bearing resistance of the splice plate depends also on the positioning of the holes and strength of the bolts as follows [4,5,6]:

$$F_{b.Rd} = k_1 \alpha_b f_u d t / \gamma_{M2} \tag{3.59}$$

where f_u is the nominal ultimate tensile strength of the part to be considered
d is the nominal diameter of the bolt
t is the thickness of the part to be considered
γ_{M2} is the partial safety factor for resistance (Table 2.5)

The factors k_1 and α_b needed in expression (3.59) are determined as follows:

- in the direction parallel to the force:

$$\alpha_b = min\left[1,0 \; ; \; \frac{f_{ub}}{f_u} \; ; \; \frac{e_1}{3d_0}\right] \qquad \textit{for end bolts} \tag{3.60a}$$

$$\alpha_b = min\left[1,0 \; ; \; \frac{f_{ub}}{f_u} \; ; \; \left(\frac{p_1}{3d_0} - \frac{1}{4}\right)\right] \qquad \textit{for inner bolts} \tag{3.60b}$$

- in the direction perpendicular to the force:

$$k_1 = min\left[2,5 \; ; \; \left(2,8\frac{e_2}{d_0} - 1,7\right) \; ; \; \left(1,4\frac{p_2}{d_0} - 1,7\right)\right] \qquad \textit{for edge bolts} \tag{3.60c}$$

$$k_1 = min\left[2,5 \; ; \; \left(1,4\frac{p_2}{d_0} - 1,7\right)\right] \qquad \textit{for inner bolts} \tag{3.60d}$$

where f_u is the nominal ultimate tensile strength of the part to be considered
f_{ub} is the nominal ultimate tensile strength of the bolt
d_0 is the nominal diameter of the bolt hole
γ_{M2} is the partial safety factor for resistance (Table 2.5)
e_1, p_1 and e_2, p_2 are the positioning of the bolt holes in the direction parallel to the force and perpendicular to the force according to Figure 3.17

The minimum distances for positioning of holes are presented in Table 3.15.

For fatigue loaded structures the minimum spacing between the hole centres and the minimum edge and end distances are presented in EN 1993-1-9 [8,9,10].

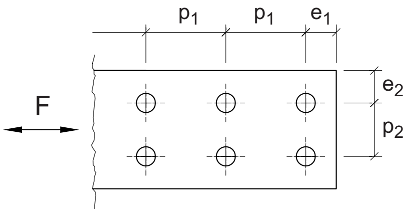


Figure 3.17 Symbols for the distances between fasteners [4,5,6]

Table 3.15 Requirements for minimum distances of holes [4,5,6]

End and edge distances and spacing between the hole centres (see Figure 3.17)	Minimum value
End distance e_1	$1,2d_0$
Edge distance e_2	$1,2d_0$
Centre distance p_1	$2,2d_0$
Centre distance p_2	$2,4d_0$
For fatigue loaded structures the minimum spacing between the hole centres and the minimum edge and end distances are presented in EN 1993-1-9.	

A group of bolts can break at the end of a joint-plate or at the end of the member according to Figure 3.18. The phenomenon is called block tearing, which is caused by tensile fracture of the parent metal in the face subject to tension force, while the face subject to shear force fails by yielding in shear. Breaking takes place in the net cross-section along the centre lines of the bolts. Block tearing can be governing the resistance of the joint, when high steel strength and small edge distances of the bolts are used.

Block tearing resistance is calculated by the following formulae [4,5,6]:

$$V_{eff.1.Rd} = f_u A_{nt} / \gamma_{M2} + (1/\sqrt{3}) f_y A_{nv} / \gamma_{M0} \qquad \textit{concentric load (Figure 3.19a)} \qquad (3.61)$$

$$V_{eff.2.Rd} = 0,5 f_u A_{nt} / \gamma_{M2} + (1/\sqrt{3}) f_y A_{nv} / \gamma_{M0} \qquad \textit{eccentric load (Figure 3.19b)} \qquad (3.62)$$

where f_u is the nominal ultimate tensile strength of the part to be considered
f_y is the nominal yield strength of the part to be considered
A_{nt} is the net cross-section area subject to tension (Figure 3.18)
A_{nv} is the net cross-section area subject to shear (Figure 3.18)
γ_{M2} and γ_{M0} are partial safety factors for resistance (Table 2.5)

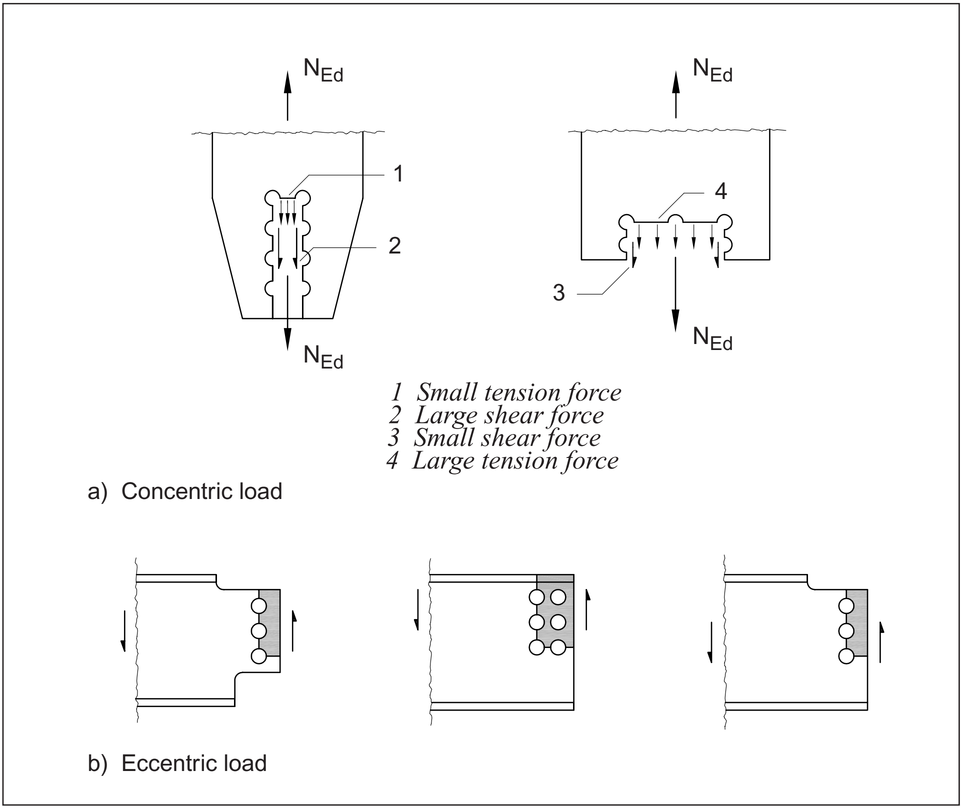
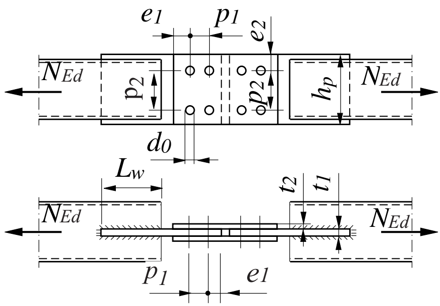


Figure 3.18 Block tearing [4,5,6]

Example 3.14

Calculate the tension resistance of the joint in the figure. The splice plates are of steel grade S355J2. The bolts are M24 in class 8.8. Partially-threaded bolts shall be applied. The length of the bolts shall be chosen so that the shear planes will not pass through the threaded portion of the bolts. The category of the bolted connection is assumed to be other than Category C.



The hollow section is 150×150×6. The steel grade is SSAB Domex Tube Double Grade, which fulfills the EN 10219 requirements for both steel grades S420MH and S355J2H. Thereby the design calculations may be performed at designer's own choice either according to grade S420 or grade S355. Grade S420 is chosen in this Example as design basis.

$A = 3363\ mm^2$ *(Annex 11.1)*
$f_y = 420\ N/mm^2$ *yield strength (hollow section)*
$f_u = 500\ N/mm^2$ *ultimate tensile strength (hollow section)*
$f_{yp} = 355\ N/mm^2$ *yield strength (splice plates)*
$f_{up} = 490\ N/mm^2$ *ultimate tensile strength (splice plates)*
$f_{ub} = 800\ N/mm^2$ *ultimate tensile strength (bolts)*
$\gamma_{M0} = 1,0$
$\gamma_{M2} = 1,25$

The parameters defining the joint geometry:

$t_1 = 20\ mm$
$t_2 = 10\ mm$
$h_p = 170\ mm$
$d = 24\ mm$
$d_0 = 26\ mm$
$e_1 = 40\ mm$
$e_2 = 45\ mm$
$p_1 = p_2 = 80\ mm$

Check the conditions for the positioning of the holes:

$$e_1 = 40\ mm \geq 1,2d_0 = 31,2\ mm \quad OK$$

$$e_2 = 45\ mm \geq 1,2d_0 = 31,2\ mm \quad OK$$

$$p_1 = 80\ mm \geq 2,2d_0 = 57,2\ mm \quad OK$$

$$p_2 = 80\ mm \geq 2,4d_0 = 62,4\ mm \quad OK$$

A. Tension resistance of the cross-section of the hollow section:

$$N_{t.Rd} = N_{pl.Rd} = \frac{Af_y}{\gamma_{M0}} = \frac{3363 \cdot 420}{1,0} = 1412\ kN$$

B. Shear resistance of the bolts:

The shear resistance per bolt, when the shear plane passes through the unthreaded portion of the bolt:

$$A = \frac{\pi d^2}{4} = \frac{\pi \cdot 24^2}{4} = 452\ mm^2 \qquad \text{M24 bolt, unthreaded cross-section area}$$

$$F_{v.Rd} = 0,6 f_{ub} A / \gamma_{M2} = 0,6 \cdot 800 \cdot 452 / 1,25 = 173,6\ kN \quad \text{per shear plane}$$

The joint has two shear planes (regarding the splice plate in the middle), thereby the shear resistance per bolt is:

$$F_{v.Rd} = 2 \cdot 173,6 = 347,2\ kN$$

In total 4 bolts, thereby the shear resistance for the whole joint is finally:

$$F_{v.Rd} = 4 \cdot 347,2 = 1389\ kN$$

C. Middlemost splice plate in the joint:

Tension resistance at the net cross-section:

The splice plates can be assessed respectively as any cross-section under tension. Thereby the resistance of a cross-section having holes can be calculated by applying expressions (2.15) and (2.16):

$$A = t_1 h_p = 20 \cdot 170 = 3400\ mm^2$$

$$A_{net} = t_1 (h_p - 2d_0) = 20 \cdot (170 - 2 \cdot 26) = 2360\ mm^2$$

$$N_{t.Rd} = N_{pl.Rd} = \frac{Af_{yp}}{\gamma_{M0}} = \frac{3400 \cdot 355}{1,0} = 1207\ kN$$

$$N_{t.Rd} = N_{u.Rd} = \frac{0,9 A_{net} f_{up}}{\gamma_{M2}} = \frac{0,9 \cdot 2360 \cdot 490}{1,25} = 832,6\ kN$$

The tension resistance of the splice plate in the middle is thereby governed by its tension resistance at the net cross-section 832,6 kN.

Bearing resistance:

The bearing resistance of the bolted connection in the splice plate, when having the positioning of the holes as given herein, is as follows:

in the direction parallel to the force:

$$\alpha_b = min\left[1,0 \; ; \; \frac{f_{ub}}{f_{up}} \; ; \; \frac{e_1}{3d_0}\right] = min[1,0 \; ; \; 1,633 \; ; \; 0,5128] = 0,5128 \qquad \text{end bolts}$$

$$\alpha_b = min\left[1,0 \; ; \; \frac{f_{ub}}{f_{up}} \; ; \left(\frac{p_1}{3d_0} - \frac{1}{4}\right)\right] = min[1,0;1,633;0,7756] = 0,7756 \qquad \text{inner bolts}$$

in the direction perpendicular to the force
(in this Example all bolts are so-called edge bolts):

$$k_1 = min\left[2,5 \; ; \left(2,8\frac{e_2}{d_0} - 1,7\right) ; \left(1,4\frac{p_2}{d_0} - 1,7\right)\right] = min[2,5 \; ; \; 3,146 \; ; \; 2,608] = 2,5$$

The bearing resistance per bolt is thereby:

$$F_{b.Rd} = k_1 \alpha_b f_u d t / \gamma_{M2} = 2,5 \cdot 0,5128 \cdot 490 \cdot 24 \cdot 20/1,25 = 241,2 \text{ kN} \qquad \text{end bolts}$$

$$F_{b.Rd} = k_1 \alpha_b f_u d t / \gamma_{M2} = 2,5 \cdot 0,7756 \cdot 490 \cdot 24 \cdot 20/1,25 = 364,8 \text{ kN} \qquad \text{inner bolts}$$

The design resistance of a group of bolts may be taken as the sum of the design bearing resistances $F_{b.Rd}$ of the individual bolts provided the design shear resistance $F_{v.Rd}$ of each individual bolt is greater than or equal to its design bearing resistance $F_{b.Rd}$. Otherwise the design resistance of a group of bolts shall be taken as the number of bolts multiplied by the smallest design resistance (either $F_{b.Rd}$ or $F_{v.Rd}$ whichever is the smaller) of any of the individual bolts [4,5,6].

It can be seen, that in respect to the inner bolts their shear resistance 347,2 kN is smaller than their bearing resistance 364,8 kN, thus the resistance of the entire joint cannot be determined here as the sum of bearing resistances.

⇒ the resistance of the entire joint shall be taken here as the number of bolts multiplied by the smallest design resistance of any of the individual bolts, i.e.:

$$4 \cdot 241,2 = 964,8 \text{ kN}$$

Block tearing resistance:

The block tearing resistance shall be calculated here for a block that consists of four bolts.

The net cross-section area subject to shear:

$$A_{nv} = 2t_1(e_1 + p_1 - d_0 - d_0/2) = 2 \cdot 20 \cdot (40 + 80 - 26 - 26/2) = 3240 \text{ mm}^2$$

The net cross-section area subject to tension:

$$A_{nt} = t_1(p_2 - d_0) = 20 \cdot (80 - 26) = 1080\ mm^2$$

The block tearing resistance for concentric tension load:

$$V_{eff.1.Rd} = f_{up}A_{nt}/\gamma_{M2} + (1/\sqrt{3})f_{yp}A_{nv}/\gamma_{M0}$$
$$= 490 \cdot 1080/1,25 + (1/\sqrt{3}) \cdot 355 \cdot 3240/1,0 = 1087\ kN$$

D. Outermost splice plates in the joint:

The edge distances of the bolts in the outermost splice plates are herein the same as edge distances in the middlemost plate. Thereby the resistances calculated above for the middle-most plate can be exploited also here:

Tension resistance at the net cross-section:

$$N_{t.Rd} = \frac{t_2}{t_1} \cdot 832,6 = \frac{10}{20} \cdot 832,6 = 416,3\ kN$$

$\Rightarrow$ *for two plates the resistance is in total 832,6 kN*

Bearing resistance:

$$F_{b.Rd} = \frac{t_2}{t_1} \cdot 241,2 = \frac{10}{20} \cdot 241,2 = 120,6\ kN \quad \text{per bolt (end bolts)}$$

$$F_{b.Rd} = \frac{t_2}{t_1} \cdot 364,8 = \frac{10}{20} \cdot 364,8 = 182,4\ kN \quad \text{per bolt (inner bolts)}$$

It can be seen, that in respect to the inner bolts their shear resistance 173,6 kN (in respect to an outermost splice plate, the joint has only one shear plane) is smaller than their bearing resistance 182,4 kN, thus the resistance of the entire joint cannot be determined here as the sum of bearing resistances.

$\Rightarrow$ *the resistance of the entire joint shall be taken here as the number of bolts multiplied by the smallest design resistance of any of the individual bolts, i.e.:*

$$4 \cdot 120,6 = 482,4\ kN$$

$\Rightarrow$ *for two plates the resistance is in total 964,8 kN*

Block tearing resistance:

$$V_{eff.1.Rd} = \frac{t_2}{t_1} \cdot 1087 = \frac{10}{20} \cdot 1087 = 543,5\ kN$$

$\Rightarrow$ *for two plates the resistance is in total 1087 kN*

E. Resistance of the welds:

The throat thickness for double-sided fillet weld shall be chosen here as a = 5 mm, since it can be welded with a single run.

According to the above obtained results the resistance of the joint is governed by the net section tension resistance of the splice plates $N_{t.Rd} = 832{,}6\ kN$ *(= resistance of the outermost splice plates in total = resistance of the middlemost splice plate).*

Next, determine the length of the weld L_w *(see the figure) to provide for the weld at least equal strength in respect to the tension resistance of the splice plates* $N_{t.Rd} = 832{,}6\ kN$.

The tension force N_{Ed} *applied in the joint causes only shear stress for the welds. The weld may be designed using expression (3.22):*

$\sigma_\perp = 0$ *(⇒ condition (3.23) does not need to be checked here)*

$\tau_\perp = 0$

$$\tau_{II} = \frac{N_{t.Rd}/4}{A_w} = \frac{N_{t.Rd}}{4L_w a}$$

$$\sqrt{\sigma_\perp^2 + 3(\tau_\perp^2 + \tau_\parallel^2)} = \sqrt{0 + 3\cdot(0 + \tau_\parallel^2)} = \sqrt{3}\tau_\parallel \le \frac{f_{uw}}{\beta_w \gamma_{M2}}$$

The required length for the weld can now be derived:

$$L_w \ge \frac{N_{t.Rd}}{4a} \cdot \frac{\beta_w \gamma_{M2}}{f_{uw}/\sqrt{3}}$$

The ultimate tensile strength of the weaker part to be joined shall be adopted for the ultimate tensile strength of the weld [4,5,6]:

$f_{uw} = min[f_u\ ;\ f_{up}] = min[500\ ;\ 490] = 490\ N/mm^2$ *(= S355)*

$\beta_w = 0{,}9$ *(strength factor for the weld, Table 3.8)* *(= S355)*

The required length for the weld is:

$$L_w \ge \frac{N_{t.Rd}}{4a} \cdot \frac{\beta_w \gamma_{M2}}{f_{uw}/\sqrt{3}} = \frac{832{,}6 \cdot 10^3}{4 \cdot 5} \cdot \frac{0{,}9 \cdot 1{,}25}{490/\sqrt{3}} = 165{,}5\ mm$$

⇒ $L_w = 170\ mm$ *shall be chosen.*

F. Resistance of the joint:

The net section tension resistance of the splice plates governs hence the resistance of the entire joint:

$N_{t.Rd} = 832{,}6\ kN.$

Comparison S420 vs S355:
In case the design calculations would be performed according to grade S355 in respect to the hollow section, the resistance of the joint would still remain the same 957 kN as governed by the splice plates. Also the length of the weld would remain the same 170 mm. Hence, increase of the hollow section's material strength S355 → S420 does not improve the resistance of the joint in this Example.

3.4.2 Bolted beam-to-column joints

A hollow section or an I-section can be joined to a hollow section column by several different techniques, as shown in Figures 3.19 - 3.25. To make the joint rigid requires the use of end-plates, which means that the tolerance on length must be more rigorous. In structures with multiple spans the variations of length may accumulate, why the length deviation must be evened out on one span by using packing plates. More elastic joints, where the bolts transmit shear forces, give more possibilities for adjusting. In elastic joints it should be remembered to consider also moments that are caused by eccentricity of shear force to column.

Fig. 3.19 A joint between I-section beam and hollow section column subject to shear, bending and normal forces. The joint resistance is usually limited by the column web by buckling or plastification in shear.

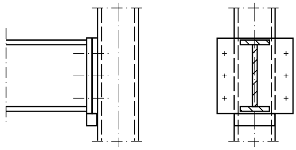


Figure 3.19

Fig. 3.20 A typical beam-to-column joint between I-section beam and hollow section column. The end of the I-section should be stiffened with a plate, which then transmits the shear force, through contact in bearing, to the stiff portion of the support component. The joint is suitable mainly for beams with minor shear force.

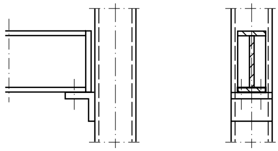


Figure 3.20

Fig. 3.21 Semi-rigidity of the joint is obtained by using very stiff end-plates. The most practical way is normally to assume the joint to be pinned and take the bending moment of the extension into account in the column design. Due to its simplicity, this type of joint is frequently used in lattice structures. Complex joint details are made by welding. Simple straight members are connected to the outstands (which are starting from the corners) by flange-plate joints. The joint stiffness between the column and the outstand can be estimated as given in Annex 11.4.

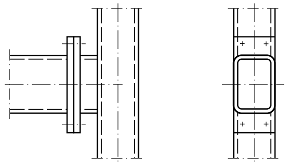


Figure 3.21

Fig. 3.22 The joints shown in Figures 3.22 a and b behave in a very similar way with each other. When the bolted joint is made as an ordinary joint, carrying the load by bolt shear, the clearances between the bolts and the holes make the joint indeterminate with regard to transmission of bending moment. With a friction-grip bolted joint, the forces and bending moments can be transferred from the beam to the support plate, preserving the rigidity.

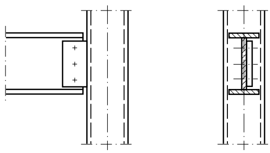
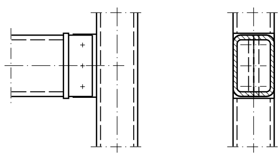


Figure 3.22a

If the joint is subject to shear force only, the support plate can be connected directly to the column flange. In a joint carrying bending moment or axial force, the column flange usually must be stiffened with a reinforcing plate.



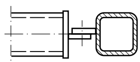


Figure 3.22b

Fig. 3.23 Figures a and b show examples of connecting a bracing to the column.

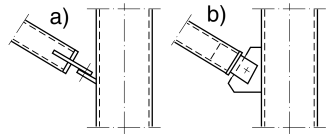


Figure 3.23

Fig. 3.24 End-to-end joint of a chord in a lattice structure. Diagonals should not be welded to the end-plates, but directly to the chord. The gap g of the joint is determined in this case as a distance between the brace member and the end-plate and it shall fulfill the requirements given in the joint resistance tables in Annex 11.3.

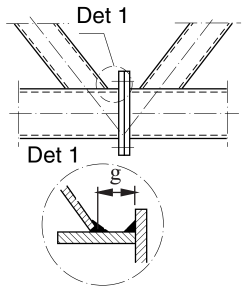


Figure 3.24

Fig. 3.25 Figures 3.25 a and b show beam-to-column joints in which the beam is continuous. For the proper functioning of this kind of joint, it is essential that the loads during erection and use of the structure are close to symmetrical. When bending moments and shear forces from the beams are unequal, the column shall be sufficiently strong to resist bending. The column flange-plate joint is taken as a hinge in relation to flexural buckling, unless the stiffness of the joint is increased with specific methods.

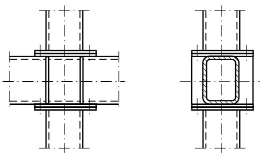


Figure 3.25a

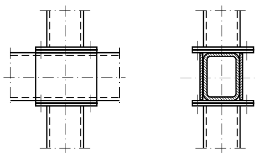


Figure 3.25b

3.4.2.1 Flange-plate joint of a hollow section subject to bending moment

The flange-plate joint is capable of transmitting both the bending moment and the shear force.

When calculating the bending resistance, the joint is divided into components, the most critical of which is governing the bending resistance of the entire joint. Components to be considered in the resistance of the joint are:

- bending resistance of the column web
- shear resistance of the column web
- tension resistance of the flange-plates and bolts
- resistances of the welds of the joint

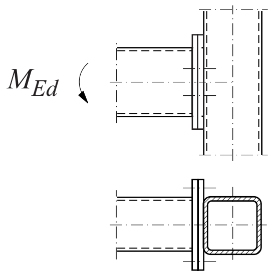


Fig. 3.26 Flange-plate joint of hollow sections

The welds of the joint are designed according to clause 3.3. In welded joints subject to bending moment the welds shall be designed such that the bending resistance of the entire joint is always governed by other components of the joint than the welds [4,5,6]. A conservative simplification is to design the welds always to have at least equal strength with the hollow sections to be joined.

The resistances for the other components of the flange-plate joint subject to bending moment can be checked as follows:

Bending resistance of the column web

The bending resistance of the column web can be estimated by applying the formulae for welded lattice joints, wherein the column is considered to represent a chord member (Table 11.3.11). In this context it shall be remembered to check that the column fulfills the requirements set in the resistance table for the welded lattice joint and the chord member in it. The flange-plate (welded on all four sides to the column) represents now a brace member having equal width with the chord member. Thereby the bending resistance of the column's web (i.e. the chord's web) can be obtained from the table as follows:

$$M_{ip.1.Rd} = 0,5 \cdot f_{y0} \cdot t_0 \cdot (h_1 + 5t_0)^2 / \gamma_{M5} \tag{3.63}$$

where h_1 is the depth of the cross-section of the beam (conservative simplification)
t_0 is the wall thickness of the column
f_{y0} is the nominal yield strength of the column
γ_{M5} is the partial safety factor for resistance (Table 2.5)

Shear resistance of the column web

The shear resistance of the column must also be checked, since the moment load is transferred from beam flanges to the column as shear force. It is assumed that the column has no external shear load, and the column shear force then consists of the joint load only. The bending resistance of the joint when governed by the shear is obtained by multiplying the shear resistance of the column webs by the beam depth:

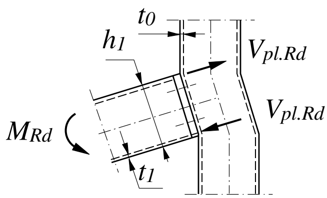


Figure 3.27 Calculation model for the bending resistance of the joint determined by the shear resistance of the column web

$$M_{Rd} = V_{pl.Rd}(h_1 - t_1) = A_V \cdot \frac{f_y/\sqrt{3}}{\gamma_{M0}} \cdot (h_1 - t_1) \tag{3.64}$$

where $V_{pl.Rd}$ is the design plastic shear resistance
A_V is the shear area of the column according to clause 2.7.1.1

h_1 is the depth of the cross-section of the beam (conservative simplification)
t_1 is the thickness of the beam flange
f_{y0} is the nominal yield strength of the column
γ_{M0} is the partial safety factor for resistance (Table 2.5)

Because expression (3.64) for bending resistance is based on the plastic shear resistance of the column, the slenderness of the column webs shall fulfill the conditions presented in clause 2.7.1 for exploiting the plastic shear resistance. Thereafter the column web is also deemed to have sufficient rotation capability, in case plastic global analysis should be applied for the structure.

Tension resistance of flange-plates and bolts

The resistance of the flange-plates and bolts can be calculated based on the same design method as presented in clause 3.4.1.1.1 for the flange-plate tension joint, provided the geometry of the joint fulfills the requirements presented therein. The design of the joint is split in this case into two separate parts (the beam and the flange-plate as one part, and the column and the flange-plate as the other part), wherein the tension resistance of the weaker part governs the bending resistance of the entire joint (see Figure 3.28).

In respect to the flange-plate joint and its bolts, the design can be simplified by considering it as a flange-plated end-to-end joint between two beam ends subjected to bending moment, where in place of the column there is a hollow section of depth equal to the beam but width equal to the column (a conservative simplification).

A joint is usually subject also to shear force, in which case the lowest bolt row (or some of the lowest bolt rows as needed) can be reserved in calculations to carry only the shear force. In calculations the remaining bolt rows carry in this case the bending moment alone, in which case the conservative simplification is to divide the bolt forces to different bolt rows according to elastic theory (elastic theory can always be used). When the topmost bolt row reaches its tension resistance, the forces acting simultaneously in the other bolt rows is distributed in proportion to the distance from the compression centre according to Figure 3.29. The compression centre is supposed to be located in the centreline of the wall thickness of the compressed flange of the beam.

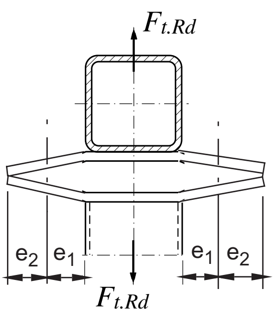


Figure 3.28 Calculation model for a flange-plate joint

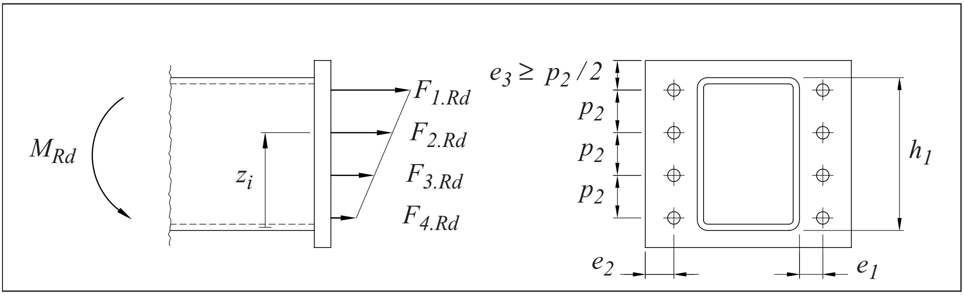


Figure 3.29 Distribution of bolt forces to different bolt rows according to elastic theory

The tension resistance of the topmost bolt row is calculated as follows:

$$F_{1.Rd} = \frac{t_p^2(1 + \delta\alpha_{Rd})n}{K} \tag{3.65}$$

where $n = 2$ is the number of bolts in the topmost bolt row

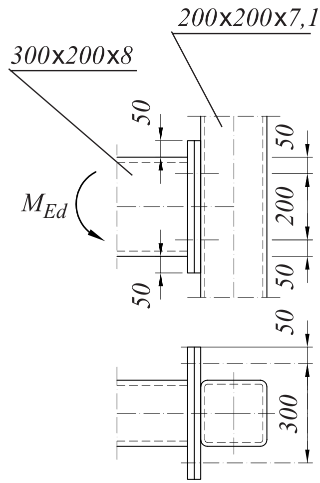
The other variables needed in expression (3.65) are determined according to clause 3.4.1.1.1, but the force acting in the bolt $F_{t.Ed}$ is substituded by the tension resistance of the bolt $F_{t.Rd}$.

The bending resistance of the joint (the beam to flange-plate, or the column to flange-plate) is now obtained from expression:

$$M_{Rd} = F_{1.Rd}\,z_1 + \sum F_{i.Ed}\,z_i \tag{3.66}$$

where z_i is the distance of the appropriate bolt row from the compression centre that is supposed to be located in the centreline of the wall thickness of the compressed flange, see Figure 3.29.

Example 3.15



Calculate the bending resistance of the flange-plate joint in the figure. The thickness of the flange-plates is 20 mm and the steel grade is S355J2. The bolts are M22 in class 8.8.

The column is a hollow section 200×200×7,1 and the beam is a hollow section 300×200×8. The steel grade is SSAB Domex Tube Double Grade, which fulfills the EN 10219 requirements for both steel grades S420MH and S355J2H. Thereby the design calculations may be performed at designer's own choice either according to grade S420 or grade S355. Grade S420 is chosen in this Example as design basis.

f_y = 420 N/mm^2 *yield strength (tube)*
f_u = 500 N/mm^2 *ultimate tensile strength (tube)*
f_{yp} = 355 N/mm^2 *yield strength (plate)*
f_{up} = 490 N/mm^2 *ultimate tensile strength (plate)*
f_{ub} = 800 N/mm^2 *ultimate tensile strength (bolt)*
γ_{M0} = 1,0
γ_{M2} = 1,25
γ_{M5} = 1,0

The parameters defining the positioning of the holes in the flange-plate:
d = 22 mm
d_0 = 24 mm
p_2 = 200 mm
e_1 = 50 mm
e_2 = 50 mm
e_3 = 100 mm (see Figure 3.29)

The bending resistance of the cross-section of the hollow sections is:
300×200×8: $M_{pl.Rd}$ = 318,0 kNm (Annex 11.1)
200×200×7,1: $M_{pl.Rd}$ = 159,3 kNm (Annex 11.1)

Usually there is also shear force present in a joint, which needs to be taken into account in design. Therefore the lowest bolt row may be reserved herein to carry the potential shear force, and the bending resistance may be determined based on the topmost bolt row alone (in respect to the bending resistance, the role of the lowest bolt row would anyway be quite marginal, only about 4 %).

A. Bending resistance of the column web:

The bending resistance of the column web is determined by applying the formulae for lattice joints, wherein the column shall fulfill the requirements set for the chord in the considered *resistance table (Annex 11.3.11). It may be noted, that the column in this Example fulfills the aforementioned requirements, though the verifications are not presented herein.*

The bending resistance of the column web is:

$$M_{ip.1.Rd} = 0,9 \cdot 0,5 f_{y0} t_0 (h_1 + 5t_0)^2 / \gamma_{M5} \qquad \text{(S420: resistance factor = 0,9)}$$

$$= 0,9 \cdot 0,5 \cdot 420 \cdot 7,1 \cdot (300 + 5 \cdot 7,1)^2 / 1,0 = 151,0 \text{ kNm}$$

B. Shear resistance of the column web:

The shear resistance of the column is calculated as presented in Chapter 2.
Check the condition for the slenderness of the web in respect to the plastic shear resistance:

$$\frac{h}{t} = \frac{200}{7,1} = 28,2 \le \frac{72\varepsilon}{\eta} + 3 = \frac{72 \cdot \sqrt{235/420}}{1,0} + 3 = 56,9 \quad OK$$

$A = 5305 \text{ mm}^2$ *cross-section area of the column*

$A_V = A \cdot \frac{h}{b+h} = 5305 \cdot \frac{200}{200+200} = 2653 \text{ mm}^2$ *shear area of the column*

$V_{pl.Rd} = A_V \cdot \frac{f_y/\sqrt{3}}{\gamma_{M0}} = 2653 \cdot \frac{420/\sqrt{3}}{1,0} = 643,3 \text{ kN}$ *plastic shear resistance of the column*

Based on the shear resistance of the column web and the depth of the beam, the bending resistance of the joint is hence:

$$M_{Rd} = V_{pl.Rd}(h_1 - t_1) = 643,3 \cdot 10^3 \cdot (300 - 8) = 187,8 \text{ kNm}$$

C. Resistance of the flange-plates and the bolts:

C.1 Bolts and the flange-plate in the beam:

Resistance of the bolts:

The tension resistance per bolt is:

$A_s = 303 \text{ mm}^2$ *tensile stress area of a M22 bolt (area in the threaded portion)*

$$F_{t.Rd} = \frac{k_2 f_{ub} A_s}{\gamma_{M2}} = \frac{0,9 \cdot 800 \cdot 303}{1,25} = 174,5 \text{ kN}$$

Calculate the punching shear resistance in respect to the chosen flange-plate thickness:

$d_m = 35,7 \text{ mm}$ *(M22 bolt, punching shear diameter of the nut or the bolt head)*

$$B_{p.Rd} = 0,6 \pi d_m t_p f_u / \gamma_{M2} = 0,6 \cdot \pi \cdot 35,7 \cdot 20 \cdot 490 / 1,25 = 527,6 \text{ kN}$$

$\Rightarrow$ *the resistance of the bolts is governed by the tension resistance 174,5 kN*

Resistance of the flange-plate joint:

Check the conditions for the positioning of the holes:

$e_1 = 50\ mm \geq 1{,}2d_0 = 28{,}8\ mm$ OK
$e_2 = 50\ mm \geq 1{,}2d_0 = 28{,}8\ mm$ OK
$e_2 = 50\ mm \leq 1{,}25e_1 = 62{,}5\ mm$ OK
$e_3 = 100\ mm \geq p_2 / 2 = 100\ mm$ OK
$p_2 = 200\ mm \geq 2{,}4d_0 = 57{,}6\ mm$ OK

$$p_2 = 200\ mm \leq min[14t_p\ ;\ 200\ mm] = min[280\ mm\ ;\ 200\ mm] = 200\ mm \quad OK$$

Check the condition for the number of bolts:

$$4 \leq n = 4 \leq 2(h_1/p_2) + 2 = 2 \cdot (300/200) + 2 = 5 \quad OK$$

Calculate the auxiliary parameters needed:

$$\delta = 1 - \frac{d_0}{p_2} = 1 - \frac{24}{200} = 0,88$$

$$b' = e_1 - 0,5d + t_1 = 50 - 0,5 \cdot 22 + 8 = 47\ mm$$

$$K = \frac{4b'}{0,9 \cdot (f_{yp}/\gamma_{M0}) \cdot p_2} = \frac{4 \cdot 47}{0,9 \cdot (355/1,0) \cdot 200} = 2,942 \cdot 10^{-3}\ mm^2/N$$

Check the conditions regarding the thickness of the flange-plate:
since the force applied to a bolt is here unknown, and the task is to determine the joint's resistance at ultimate limit state, the force acting in a bolt $F_{t.Ed}$ shall be assumed to be equal to the tension resistance of the bolt $F_{t.Rd} = 174{,}5\ kN$.

The minimum and maximum values for the thickness of the flange-plate are calculated by the formula:

$$\sqrt{\frac{K \cdot F_{t.Rd}}{1 + \delta}} \leq t_p \leq \sqrt{K \cdot F_{t.Rd}}$$

$$\sqrt{\frac{2,942 \cdot 10^{-3} \cdot 174,5 \cdot 10^3}{1 + 0,88}} \leq t_p \leq \sqrt{2,942 \cdot 10^{-3} \cdot 174,5 \cdot 10^3}$$

$$16,5\ mm \leq t_p \leq 22,7\ mm$$

$\Rightarrow t_p = 20\ mm = OK$

The tension resistance of the topmost bolt row of the flange-plate joint is hence:

$$\alpha_{Rd} = \left(\frac{K \cdot F_{t.Rd}}{t_p^2} - 1\right) \cdot \left[\frac{e_2 + 0,5d}{\delta \cdot (e_2 + e_1 + t_1)}\right]$$

$$= \left(\frac{2,942 \cdot 10^{-3} \cdot 174,5 \cdot 10^3}{20^2} - 1\right) \cdot \left[\frac{50 + 0,5 \cdot 22}{0,88 \cdot (50 + 50 + 8)}\right] = 0,1819 \geq 0$$

$$F_{1.Rd} = \frac{t_p^2(1 + \delta\alpha_{Rd})n}{K} = \frac{20^2 \cdot (1 + 0,88 \cdot 0,1819) \cdot 2}{2,942 \cdot 10^{-3}} = 315,5\ kN \text{ (topmost bolt row: n=2)}$$

In respect to the flange-plate attached in the beam, the bending resistance of the flange-plate joint is (since the lowest bolt row is reserved to carry the potential shear force):

$z_1 = 200 + 50 - 8/2 = 246\ mm$ *distance of the topmost bolt row from the compression centre*

$$M_{Rd} = F_{1.Rd} z_1 + \sum F_{i.Ed} z_i = 315,5 \cdot 10^3 \cdot 246 + 0 = 77,6\ kNm$$

C.2 Bolts and the flange-plate in the column:

Since the thickness of the flange-plate is the same as in case of the beam, the results obtained above in clause C.1 can directly be exploited here.

Resistance of the bolts:

The same as above in clause C.1.

Resistance of the flange-plate joint:

The conditions for the positioning of the holes and for the number of bolts have already been checked in clause C.1.

Calculate the auxiliary parameters needed:

$$\delta = 1 - \frac{d_0}{p_2} = 1 - \frac{24}{200} = 0,88$$

$$b' = e_1 - 0,5d + t_1 = 50 - 0,5 \cdot 22 + 7,1 = 46,1\ mm$$

$$K = \frac{4b'}{0,9 \cdot (f_{yp}/\gamma_{M0}) \cdot p_2} = \frac{4 \cdot 46,1}{0,9 \cdot (355/1,0) \cdot 200} = 2,886 \cdot 10^{-3}\ mm^2/N$$

The minimum and maximum values for the thickness of the flange-plate are calculated by the formula:

$$\sqrt{\frac{K \cdot F_{t.Rd}}{1+\delta}} \le t_p \le \sqrt{K \cdot F_{t.Rd}}$$

$$\sqrt{\frac{2,886 \cdot 10^{-3} \cdot 174,5 \cdot 10^3}{1+0,88}} \le t_p \le \sqrt{2,886 \cdot 10^{-3} \cdot 174,5 \cdot 10^3}$$

$$16,4\ mm \le t_p \le 22,4\ mm$$

$\Rightarrow t_p = 20\ mm = OK$

The tension resistance of the topmost bolt row of the flange-plate joint is hence:

$$\alpha_{Rd} = \left(\frac{K \cdot F_{t.Rd}}{t_p^2} - 1\right) \cdot \left[\frac{e_2 + 0,5d}{\delta \cdot (e_2 + e_1 + t_1)}\right]$$

$$= \left(\frac{2,886 \cdot 10^{-3} \cdot 174,5 \cdot 10^3}{20^2} - 1\right) \cdot \left[\frac{50 + 0,5 \cdot 22}{0,88 \cdot (50+50+7,1)}\right] = 0,1676 \ge 0$$

$$F_{1.Rd} = \frac{t_p^2(1+\delta\alpha_{Rd})n}{K} = \frac{20^2 \cdot (1+0,88 \cdot 0,1676) \cdot 2}{2,886 \cdot 10^{-3}} = 318,1\ kN \text{ (topmost bolt row: n=2)}$$

In respect to the flange-plate attached in the column, the bending resistance of the flange-plate joint is (since the lowest bolt row is reserved to carry the potential shear force):

$z_1 = 200 + 50 - 8/2 = 246\ mm$ *distance of the topmost bolt row from the compression centre*

$$M_{Rd} = F_{1.Rd} z_1 + \sum F_{i.Ed} z_i = 318,1 \cdot 10^3 \cdot 246 + 0 = 78,3\ kNm$$

C.3 Bending resistance of the flange-plates:

The bending resistance of the beam's flange-plate and bolts is smaller than that of the column, thereby the beam's bolts and flange-plate governs the bending resistance of the whole flange-plate joint:

$M_{Rd} = 77,6\ kNm$

D. Design of the welds:

Beam to flange-plate:

In welded joints subject to bending moment the welds shall be designed such that the bending resistance of the entire joint is always governed by other components of the joint than the welds [4,5,6].

A conservative simplification is to design the welds always to have at least equal strength with the hollow sections to be joined. In case of hollow sections this principle is generally *adopted, because in hollow section joints the stiffness of the joint varies around the joint, which causes for the weld an uneven distribution of the forces. By designing the weld to have equal strength to the hollow section wall, the sufficient resistance and deformation capability is ascertained for the weld to be able to even out the stresses.*

In respect to the beam to flange-plate weld, the throat thickness needed for the fillet weld (when welded around the perimeter of the hollow section) by applying the Simplified method, is $a \geq 1{,}67 \cdot t$ (Table 3.13). If applying the more accurate Directional method, the sufficient throat thickness would be $a \geq 1{,}36 \cdot t$ (Table 3.12), when the beam is subjected to only bending, which causes normal stress.

Hence, the required throat thickness for the weld is:

$a \geq 1{,}36 \cdot t = 1{,}36 \cdot 8 = 10{,}9 \ mm$

$\Rightarrow$ *a = 11 mm shall be chosen around the whole hollow section.*

Column to flange-plate:

The flange-plate is welded to the column on all four sides of the plate. The weld is designed to have equal strength with the column (correspondingly as the weld between the beam and the flange-plate earlier), whereby the required throat thickness for the weld is:

$a \geq 1{,}36 \cdot t = 1{,}36 \cdot 7{,}1 = 9{,}7 \ mm$

$\Rightarrow$ *a = 10 mm shall be chosen around the whole hollow section.*

E. Bending resistance of the whole joint:

The resistance of the bolts and flange-plate attached at the beam end is the smallest, thus it governs the bending resistance of the whole joint:

$M_{Rd} = 77{,}6 \ kNm$

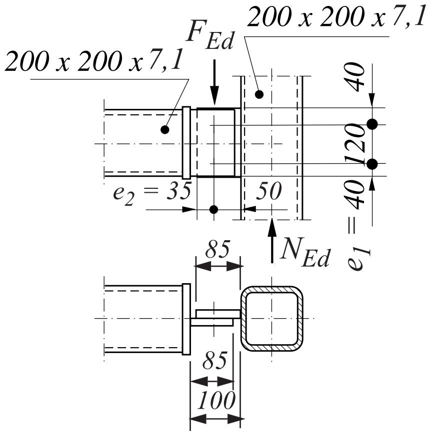
Note:
In case the joint would be subject in addition to the bending moment also to shear force (as the case usually is), the following shall also be checked in addition to the preceding verifications:

- *in respect to both flange-plates, the resistance of the vertical welds to the applied shear force too*
- *in respect to the lowest bolt row, the shear resistance to the applied shear force*

Comparison S420 vs S355:
In case the design calculations would be performed according to grade S355 in respect to the hollow section, the resistance of the joint would still remain the same 77,6 kNm as governed by the the bolts and flange-plate at the beam end. In respect to the weld, a somewhat smaller throat thickness would be sufficient: $a \geq 1{,}36 \cdot t \rightarrow 1{,}15 \cdot t$ (Table 3.12). However, increase of the hollow section's material strength S355 $\rightarrow$ S420 does not improve the resistance of the joint in this Example.

Example 3.16

Check the resistance of the lap joint in the figure.



The normal force in the column is N_{Ed} = 300 kN (compression). The joint is subjected to a force F_{Ed} = 150 kN at beam end. The thickness of the splice plates is 15 mm and the steel grade is S355J2. The bolts are M20 in class 8.8.

Hollow section 200×200×7,1 is chosen for the column and for the beam. The steel grade is SSAB Domex Tube Double Grade, which fulfills the EN 10219 requirements for both steel grades S420MH and S355J2H. Thereby the design calculations may be performed at designer's own choice either according to grade S420 or grade S355. Grade S420 is chosen in this Example as design basis.

The bolted connection presented in the figure is supposed to act as nominally pinned and thereby not transmitting any moment. However, the eccentricity of the vertical load F_{Ed} causes the following moments:

- *the moment applied to the root of the splice plate connected to the column (and the welds therein):*
  $M_{Ed} = 150 \cdot 0{,}05 = 7{,}5$ kNm
- *the moment applied to the column (to be considered in the design of the column):*
  $M_{Ed} = 150 \cdot (0{,}05 + 0{,}2/2) = 22{,}5$ kNm
- *the moment applied to the root of the splice plate connected to the end-plate of the beam (and the welds therein):*
  $M_{Ed} = 150 \cdot 0{,}05 = 7{,}5$ kNm
- *the moment applied to the end-plate of the beam (and the welds therein):*
  $M_{Ed} = 150 \cdot (0{,}05 + 0{,}015) = 9{,}75$ kNm

A	= 5305 mm^2	*(Annex 11.1)*	
f_y	= 420 N/mm^2	*yield strength*	*(hollow section)*
f_u	= 500 N/mm^2	*ultimate tensile strength*	*(hollow section)*
f_{yp}	= 355 N/mm^2	*yield strength*	*(plates)*
f_{up}	= 490 N/mm^2	*ultimate tensile strength*	*(plates)*
f_{ub}	= 800 N/mm^2	*ultimate tensile strength*	*(bolts)*
γ_{M0}	= 1,0		
γ_{M2}	= 1,25		
γ_{M5}	= 1,0		

The parameters defining the joint geometry:

t_p = 15 mm
d = 20 mm
d_0 = 22 mm
e_1 = 40 mm
e_2 = 35 mm
p_1 = 120 mm
h_p = 200 mm

Check the conditions for the positioning of the holes:

$e_1 = 40\ mm \geq 1,2d_0 = 26,4\ mm$ OK

$e_2 = 35\ mm \geq 1,2d_0 = 26,4\ mm$ OK

$p_1 = 120\ mm \geq 2,2d_0 = 48,4\ mm$ OK

A. Bolts; shear resistance:

The shear resistance per bolt, when the shear plane passes through the threaded portion of the bolt (conservative assumption):

$A_s = 245\ mm^2$ *tensile stress area of a M20 bolt (area in the threaded portion)*

$F_{v.Rd} = 0,6 f_{ub} A_s / \gamma_{M2} = 0,6 \cdot 800 \cdot 245/1,25 = 94,1\ kN$

The joint has two bolts and one shear plane per bolt
$\Rightarrow F_{v.Rd} = 2 \cdot 94,1 = 188,2\ kN \geq F_{Ed}$ OK

B. Splice plates; bearing resistance:

The bearing resistance of the splice plates is calculated as in Example 3.14:

in the direction parallel to the force:

$$\alpha_b = min\left[1,0\ ;\ \frac{f_{ub}}{f_{up}}\ ;\ \frac{e_1}{3d_0}\right] = min[1,0\ ;\ 1,633\ ;\ 0,6061] = 0,6061 \quad \text{end bolts}$$

$$\alpha_b = min\left[1,0\ ;\ \frac{f_{ub}}{f_{up}}\ ;\ \left(\frac{p_1}{3d_0} - \frac{1}{4}\right)\right] = min[1,0;1,633;\ 1,568] = 1,0 \quad \text{inner bolts}$$

in the direction perpendicular to the force
(in this Example all bolts are so-called edge bolts):

$$k_1 = min\left[2,5\ ;\ \left(2,8\frac{e_2}{d_0} - 1,7\right)\ ;\ \left(1,4\frac{p_2}{d_0} - 1,7\right)\right] = min[2,5\ ;\ 2,755\ ;\ -\ ] = 2,5$$

The bearing resistance per bolt is thereby:

$$F_{b.Rd} = k_1 \alpha_b f_u d t / \gamma_{M2} = 2,5 \cdot 0,6061 \cdot 490 \cdot 20 \cdot 15/1,25 = 178,2\ kN \quad \text{end bolt}$$

$$F_{b.Rd} = k_1 \alpha_b f_u d t / \gamma_{M2} = 2,5 \cdot 1,0 \cdot 490 \cdot 20 \cdot 15/1,25 = 294,0\ kN \quad \text{inner bolt}$$

The design resistance of a group of bolts may be taken as the sum of the design bearing resistances $F_{b.Rd}$ of the individual bolts provided the design shear resistance $F_{v.Rd}$ of each individual bolt is greater than or equal to its design bearing resistance $F_{b.Rd}$. Otherwise the design resistance of a group of bolts shall be taken as the number of bolts multiplied by the smallest design resistance (either $F_{b.Rd}$ or $F_{v.Rd}$ whichever is the smaller) of any of the individual bolts [4,5,6].

It can be seen, that each bolt has $F_{v.Rd} < F_{b.Rd}$ (see preceding clause A)
$\Rightarrow$ thus the resistance of the entire joint can be determined here by multiplying the number of bolts by the smallest design resistance of any of the individual bolts, i.e.:

$$2 \cdot 94,1 = 188,2\ kN \geq F_{Ed} \quad OK$$

C. Splice plates; block tearing resistance:

The block tearing resistance shall be calculated here for a block that consists of two bolts.

The net cross-section area subject to shear:

$$A_{nv} = t_p(e_1 + p_1 - d_0 - d_0/2) = 15 \cdot (40 + 120 - 22 - 22/2) = 1905\ mm^2$$

The net cross-section area subject to tension:

$$A_{nt} = t_p(e_2 - d_0/2) = 15 \cdot (35 - 22/2) = 360\ mm^2$$

The block tearing resistance for eccentric load:

$$V_{eff.2.Rd} = 0,5 f_{up} A_{nt}/\gamma_{M2} + (1/\sqrt{3}) f_{yp} A_{nv}/\gamma_{M0}$$

$$= 0,5 \cdot 490 \cdot 360/1,25 + (1/\sqrt{3}) \cdot 355 \cdot 1905/1,0 = 461,0\ kN \geq F_{Ed} \quad OK$$

D. Splice plates; shear resistance of the net cross-section:

The shear resistance of the net cross-section at fastener holes:

$$A_{V.net} = (h_p - 2d_0) t_p = (200 - 2 \cdot 22) \cdot 15 = 2340\ mm^2$$

$$V_{pl.net.Rd} = A_{V.net} \cdot \frac{f_y/\sqrt{3}}{\gamma_{M0}} = 2340 \cdot \frac{355/\sqrt{3}}{1,0} = 479,6\ kN \geq F_{Ed} \quad OK$$

E. Splice plates; resistance of the gross cross-section at the root of the plate:

The shear resistance of the gross cross-section at the root of the plate:

$$A_V = h_p t_p = 200 \cdot 15 = 3000\ mm^2$$

$$V_{pl.Rd} = A_V \cdot \frac{f_y / \sqrt{3}}{\gamma_{M0}} = 3000 \cdot \frac{355/\sqrt{3}}{1,0} = 614,9\ kN \geq F_{Ed} \quad OK$$

The bending resistance of the splice plate is:

$$W_{el.p} = \frac{t_p h_p^2}{6} = \frac{15 \cdot 200^2}{6} = 100 \cdot 10^3\ mm^3$$

$$M_{el.p.Rd} = \frac{W_{el.p} f_{yp}}{\gamma_{M0}} = \frac{100 \cdot 10^3 \cdot 355}{1,0} = 35,5\ kNm \geq M_{Ed} = 7,5\ kNm \quad OK$$

Combined effect of shear force and bending moment:

$$V_{Ed} = 150\ kN \leq 0,5 \cdot V_{pl.Rd} = 0,5 \cdot 614,9 = 307,5\ kN$$

$\Rightarrow$ *shear force does not reduce the bending resistance of the splice plate*

F. Welded plate-to-column joint; resistance of the joint:

The resistance of the joint is calculated by applying the formulae given for welded lattice joints. A joint between a plate and a hollow section is presented in Annex 11.3 in Table 11.3.15. In this context, the column and the plate must fulfill the requirements given in Table 11.3.15. It may be noted, that the aforementioned conditions are met, although the verifications are not presented herein.

The normal stress $\sigma_{0.Ed}$ in the column (i.e. chord) face has an impact on the joint's resistance by the parameter k_m (compression is positive):

$$n = \frac{\sigma_{0.Ed}}{f_{y0}/\gamma_{M5}} = \frac{N_{0.Ed}}{A_0 f_{y0}/\gamma_{M5}} + \frac{M_{0.Ed}}{W_{el.0} f_{y0}/\gamma_{M5}} = \frac{300 \cdot 10^3}{5305 \cdot 420/1,0} + 0 = 0,1346$$

$$k_m = 1,3 - 1,3|n| = 1,3 - 1,3 \cdot 0,1346 = 1,125 > 1 \qquad \Rightarrow k_m = 1,0$$

Chord (i.e. column) face failure by yielding
(S420: resistance factor = 0,9):

$$\begin{aligned} M_{1.Rd} &= 0,5 N_{1.Rd} h_1 = 0,5 \cdot h_1 \cdot [0,9 \cdot k_m \cdot f_{y0} \cdot t_0^2 \cdot (2h_1/b_0 + 4\sqrt{1 - t_1/b_0})]/\gamma_{M5} \\ &= 0,5 \cdot 200 \cdot [0,9 \cdot 1,0 \cdot 420 \cdot 7,1^2 \cdot (2 \cdot 200/200 + 4\sqrt{1 - 15/200})]/1,0 \\ &= 11,1\ kNm \geq M_{Ed} = 7,5\ kNm \quad OK \end{aligned}$$

G. Welds; resistance of the welds:

Weld between the column and the splice plate:

In welded joints subject to bending moment the welds shall be designed such that the bending resistance of the entire joint is always governed by other components of the joint than the welds [4,5,6]. A conservative simplification is to design the welds always to have at least equal strength with the parts to be joined.

Since the root of the splice plate is now subjected to simultaneous bending and shear, Table 3.13 should be applied. Thereby the throat thickness for a single fillet weld would be $a = 1{,}67 \cdot t_p$. *However, herein the splice plate shall be welded with a double-sided fillet weld, thus the required throat thickness for each weld should be* $a = 0{,}84 \cdot t_p = 0{,}84 \cdot 15 = 12{,}6\ mm \Rightarrow 13\ mm$. *The weld size would thus become quite big.*

It is more favourable herein to design the weld in respect to the actual shear force, while requiring at the same time also at least equal bending resistance to the above calculated bending resistance of the column face (that one being less than the bending resistance of the splice plate), i.e. $V_{Ed} = 150\ kN$ *and* $M_{Ed} = M_{1.Rd} = 11{,}1\ kNm$. *By applying the Directional method presented in clause 3.3.3.1, the stress components of the weld at the edge of the splice plate (top edge and bottom edge) are:*

$$\tau_{||} = \frac{V_{Ed}}{2ah_p}$$

$$\tau_{\perp} = \sigma_{\perp} = \frac{M_{Ed}}{W_{el.p}} \cdot \frac{t_p}{2a} \cdot \frac{1}{\sqrt{2}}$$

The design conditions for the weld are (the ultimate tensile strength of the weaker part to be joined shall be adopted for the ultimate tensile strength of the weld):

$$\sqrt{\sigma_{\perp}^2 + 3(\tau_{\perp}^2 + \tau_{||}^2)} \le \frac{f_u}{\beta_w \gamma_{M2}}$$

$$\sigma_{\perp} \le \frac{0{,}9 f_u}{\gamma_{M2}}$$

Try throat thickness 3 mm which is the smallest permitted throat thickness for a load carrying fillet weld (see clause 3.3.3). Thereby the stresses of the weld, when having the fillet weld on both sides of the plate, will be:

$$\tau_{||} = \frac{V_{Ed}}{2ah_p} = \frac{150 \cdot 10^3}{2 \cdot 3 \cdot 200} = 125\ N/mm^2$$

$$\tau_{\perp} = \sigma_{\perp} = \frac{M_{1.Rd}}{W_{el.p}} \cdot \frac{t_p}{2a} \cdot \frac{1}{\sqrt{2}} = \frac{11{,}1 \cdot 10^6}{100 \cdot 10^3} \cdot \frac{15}{2 \cdot 3} \cdot \frac{1}{\sqrt{2}} = 196{,}2\ N/mm^2$$

Check the both design conditions:

$$\sqrt{\sigma_\perp^2 + 3(\tau_\perp^2 + \tau_{||}^2)} = 448,2\ N/mm^2 > \frac{f_u}{\beta_w \cdot \gamma_{M2}} = \frac{490}{0,9 \cdot 1,25} = 435,6\ N/mm^2 \quad \text{not OK}$$

$$\sigma_\perp = 196,2\ N/mm^2 \leq \frac{0,9 f_u}{\gamma_{M2}} = \frac{0,9 \cdot 490}{1,25} = 352,8\ N/mm^2 \quad \text{OK}$$

⇒ throat thickness 3 mm is not adequate, however it can easily be seen that a = 4 mm would be sufficient

In respect to the cooling rate of the weld (see clause 3.3.3.):

$a \geq \sqrt{t}\ mm - 0,5\ mm = \sqrt{15} - 0,5 = 3,4\ mm$ *(t = thicker of the parts to be joined)*

⇒ a = 4 mm shall be chosen (the resistance and the cooling rate are both OK)

<u>Weld between the beam end-plate and the splice plate</u>:

The same throat thickness can be applied for the weld between the beam end-plate and the splice plate.

<u>Welds between the beam and the beam end-plate</u>:

The depth and the width of the end-plate is chosen so that the end-plate shall exceed the beam's cross-section by 10 mm on each side in order to leave enough space for the fillet weld running around the beam.

The vertical welds shall be designed to carry alone only the shear force, and the horizontal welds shall be designed to carry alone only the bending moment.

Since the welds in the end-plate are subjected to the same shear force V_{Ed} = 150 kN as the welds in the root of the splice plate, and since the amount of the welds carrying the shear force is the same in both cases, the same throat thickness which is applied for the splice plate will be sufficient also for the vertical welds, i.e. the same a = 4 mm shall be chosen.

The horizontal welds are designed to carry alone the bending moment which is:

$$M_{Ed} = 150 \cdot (0,05 + 0,015) = 9,75\ kNm$$

Check whether the smallest permitted throat thickness a = 3 mm for a load carrying fillet weld would be sufficient herein:

$$\tau_{||} = 0$$

$$\tau_\perp = \sigma_\perp = \frac{M_{Ed}}{h_1} \cdot \frac{1}{ab_1} \cdot \frac{1}{\sqrt{2}} = \frac{9,75 \cdot 10^6}{200} \cdot \frac{1}{3 \cdot 200} \cdot \frac{1}{\sqrt{2}} = 57,5\ N/mm^2$$

Check the design conditions for the weld:

$$\sqrt{\sigma_\perp^2 + 3(\tau_\perp^2 + \tau_{||}^2)} = 115,0\ N/mm^2 \leq \frac{f_u}{\beta_w \cdot \gamma_{M2}} = \frac{490}{0,9 \cdot 1,25} = 435,6\ N/mm^2 \quad \text{OK}$$

$$\sigma_\perp = 57,5\ N/mm^2 \leq \frac{0,9 f_u}{\gamma_{M2}} = \frac{0,9 \cdot 490}{1,25} = 352,8\ N/mm^2 \quad \text{OK}$$

It can be seen that a = 3 mm would be sufficient for the horizontal welds. However, the same a = 4 mm as already chosen for the vertical welds shall be adopted herein also for the horizontal welds.

Resistance of the joint:

Based on the preceding verifications, it can be concluded that the joint has sufficient resistance to carry the applied loads, provided the throat thicknesses of the welds are chosen as specified above.

Comparison S420 vs S355:
In case the design calculations would be performed according to grade S355 in respect to the hollow section, the resistance of the plate-to-column joint (clause F) would be 10,5 kNm (S420: 11,1 kNm), which would be sufficient. In respect to the welds, the same throat thicknesses would be chosen. Hence, increase of the hollow section's material strength S355 → S420 does not make any big difference in this Example.

3.5 Joint of a structural hollow section to the foundation

Column bases should be of sufficient size, stiffness and resistance to transmit the axial forces, bending moments and shear forces in columns to their foundations without exceeding the load carrying capacity of the foundations.

In the usual anchor bolt joint shown in Figure 3.30 a base plate is welded to the bottom of the column, and then fastened with anchor bolts (foundation bolts, holding-down bolts) to the concrete foundation. The number of anchor bolts shall be at least four if the column will be erected as free standing. Because of the loads while erecting, at least the bolt size M20 shall be used [20].

To ensure the immovability of the anchor bolts while casting the foundation, and especially to keep the distances between individual bolts within permitted tolerances, the bolts are often fastened to each other for example with angle sections according to Figure 3.30 so that a pre-fabricated group of bolts ("bolt basket") will be formed [20].

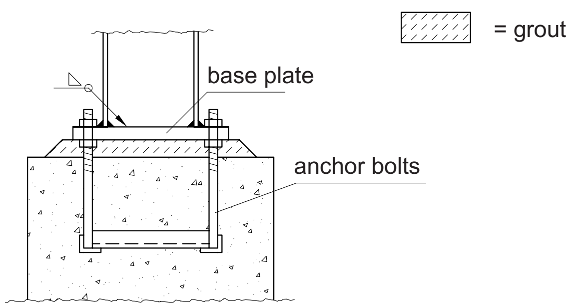


Figure 3.30 Column-to-foundation joint using anchor bolts [20]

The joint between the column and the base plate may need to be strengthened using stiffener plates. With the stiffeners the bearing pressure can be distributed more uniformly to the foundation, which makes it possible to reduce the thickness of the base plate. In Eurocode there are, however, no instructions regarding the influence of the stiffeners to the thickness of the base plate. When needed, instructions can be found e.g. in [20]. It is, however, often more economical to make the base plate thicker and leave the stiffeners away.

The tension and compression stresses acting in the column are transferred to the base plate such that they act perpendicular to the rolling direction of the plate (the corresponding situation exists also in the joints of beams, if executed using an end-plate welded to the end of the beam). Avoiding lamellar tearing is then especially important. Lamellar tearing is presented in Chapter 5.

The design of anchor bolts and foundation comprises different load cases to be checked. In order to determine the resistance of the foundation, the load case causing the highest bearing pressure on the foundation has to be identified. The size and thickness of the base plate shall be chosen so great that the compression stress of the concrete foundation does not exceed the design strength f_{jd} of the bearing pressure of the concrete foundation according to clause 3.5.1 [4,5,6].

The anchor bolts are chosen on the basis of tension resistance, shear resistance or combined tension and shear resistance. If the anchor bolts are subject to tension, then also the tension resistance of the flange-plate joint comprising the base plate and the column shall be checked (cf. flange-plate joint in clause 3.4.1.1.1). In some cases the loads under erection period may be the critical ones, wherein the anchor bolts are chosen on the basis of buckling resistance.

In a finished structure tension is usually critical when designing the anchor bolts. To transfer the tension forces to the foundation the bolts shall be anchored to concrete. If utilising the bond alone, the anchorage length would become very long. Therefore the anchor bolts are anchored using a hook, washer plate or some other anchoring device which is adequately tested and approved. For the design resistance of the anchor bolts the smaller of the following values is chosen: the design tension resistance of the (anchor) bolt according to clause 3.4.1.1.1 and the design bond resistance of the concrete on the anchor bolt according to EN 1992-1-1. The anchorage length is determined according to EN 1992-1-1. When there is a hook at the end of the anchor bolt, the anchorage length shall be chosen such that bond failure does not occur before yielding of the anchor bolt. The hook-type anchorage shall not be used, if the yield strength of anchor bolts f_{yb} is higher than 300 N/mm^2 [4,5,6].

One of the following methods is used to resist the shear force between the base plate and its support [4,5,6]:

- frictional design resistance at the joint between the base plate and its support

- the design shear resistance of the anchor bolts
  (determination of shear resistance for anchor bolts differs slightly from other bolts, see EN 1993-1-8: 6.2.2(7))

- the design shear resistance of the surrounding part of the foundation

If the above methods are not adequate to transfer the shear forces, it is possible to weld under the base plate a dowel section that is designed to carry the shear force in one direction or in both directions in regard to the principal axes. In this case the anchor bolts can be designed only for tension or for compression (buckling resistance) applied under the erection period.

The concrete structure of the foundation and its reinforcement are designed according to Part EN 1992-1-1 of Eurocode [1,2].

3.5.1 Joint of a normal force loaded column to the foundation

In case of a column subject to normal force the resistance of the foundation to the bearing pressure caused by the normal force shall be checked. The bearing pressure is supposed to be uniformly spread onto the area presented in Figure 3.31.

The bearing pressure causes bending moment in the base plate. In order to avoid too large deformations, the value of the bending moment is limited to the elastic bending resistance of the base plate. Thereby the design strength f_{jd} for the bearing pressure of the concrete determines the size of the base plate as follows:

$$t_p \geq \sqrt{\frac{6M_{p.Ed}}{l_{eff} \cdot f_{yp}/\gamma_{M0}}} \tag{3.67}$$

$$M_{p.Ed} = \frac{l_{eff} s^2 f_{jd}}{2} \tag{3.68}$$

where t_p is the thickness of the base plate
$M_{p.Ed}$ is bending caused by the bearing pressure of concrete to the base plate
l_{eff} is the effective length of the base plate according to Figure 3.31
s is the extension of the base plate outside the perimeter of the column, however neglecting the area exceeding the dimension c, i.e. $s \leq c$ as presented in Figure 3.31
f_{yp} is the nominal yield strength of the base plate
γ_{M0} is the partial safety factor for resistance (Table 2.5)

The design strength f_{jd} for the bearing pressure of the foundation is obtained as follows [1,2,4,5,6]:

$$f_{jd} = \frac{\beta_j F_{Rdu}}{b_{eff} l_{eff}} \tag{3.69}$$

$$F_{Rdu} = f_{cd} A_{co} \sqrt{\frac{A_{c1}}{A_{co}}} \leq 3,0 f_{cd} A_{co} \tag{3.70}$$

$$f_{cd} = \alpha_{cc} f_{ck}/\gamma_C \tag{3.71}$$

The expressions (3.69) - (3.71) are presented here in the same form as in Eurocode. When taking into account that $A_{co} = b_{eff} l_{eff}$, the expression for the design strength f_{jd} of concrete's bearing pressure is derived in the following form:

$$f_{jd} = \beta_j \cdot k_j \cdot \alpha_{cc} f_{ck} / \gamma_C \tag{3.72}$$

$$k_j = \sqrt{\frac{A_{c1}}{A_{co}}} \quad but \ \ k_j \leq 3,0 \tag{3.73}$$

where β_j = 2/3, when the characteristic strength of the grout is at least 0,2 times the characteristic strength of the concrete foundation and the thickness of the grout is not greater than 0,2 times the smallest width of the steel base plate. When the thickness of the grout is greater than 50 mm, the characteristic strength of the grout is selected to be at least equal to the characteristic strength of the concrete foundation.

k_j is the concentration factor; k_j = 1 as a conservative default value (more accurate calculation: see EN 1992-1-1: clause 6.7)

α_{cc} is a factor for the strength of concrete, the value is chosen in the National Annex between 0,8 and 1,0

Finnish National Annex to standard EN 1992-1-1 [3]:
The value α_{cc} = 0,85 is used.

f_{ck} is the characteristic compressive cylinder strength of concrete at 28 days

γ_C is the partial safety factor for concrete, for which the recommended value in EN 1992-1-1 is γ_C = 1,5

Finnish National Annex to standard EN 1992-1-1 [3]:
The recommended value of Eurocode γ_C = 1,5 is used.

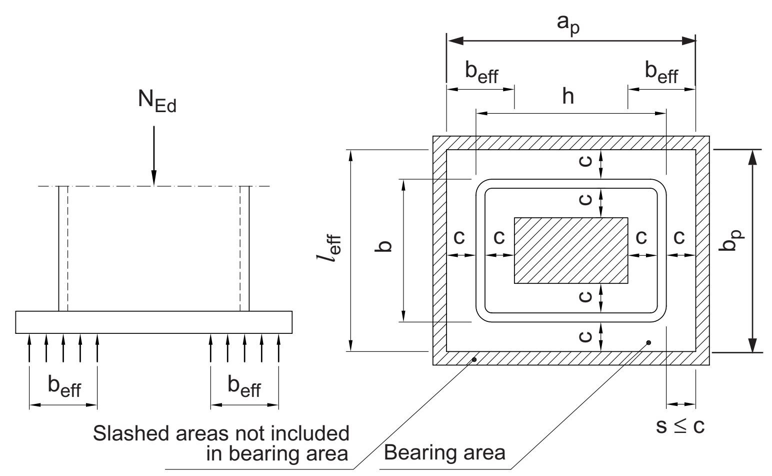


Figure 3.31 Compressed area under the base plate

The compressed area under the base plate is determined according to Figure 3.31. Dimension c which determines the extension of the bearing area is calculated as follows [4,5,6]:

$$c = t_p \sqrt{\frac{f_{yp}/\gamma_{M0}}{3f_{jd}}} \qquad (3.74)$$

The entire base plate is included in the bearing area, if the following conditions are fulfilled (see Figure 3.31):

$$(b - 2t) \le 2c \qquad \textit{the area inside the hollow section is fully effective} \qquad (3.75)$$

$$b_p \le b + 2c \quad ja \quad a_p \le h + 2c \qquad \textit{the area outside the hollow section is fully effective} \qquad (3.76)$$

where the dimensions a_p and b_p are the external dimensions of the base plate.

3.5.2 Joint of a normal force and bending moment loaded column to the foundation

Depending on the ratios between normal force and bending moment the base plate can be totally in compression or totally in tension, or one side of the joint may be in compression and the other in tension.

In the foundation joint of a column subject to normal force and bending moment, the tension resistance and anchorage resistance of the anchor bolts shall be checked in addition to the resistance of the base plate and the foundation, if the anchor bolts are subject to tension. Also the effect of shear force shall be taken into account.

When calculating the tension forces in the anchor bolts due to bending moments, the lever arm shall not be taken larger than the distance between the centroid of the bearing area on the compression side and the centroid of the bolt group on the tension side [4,5,6].

When the hollow section column is positioned concentrically to the base plate, the following equilibrium equations are obtained from Figure 3.32:

$$N_{Ed} = N_c - N_s = l_{eff}\, y f_{jd} - A_s f_{yb} \qquad (3.77)$$

$$M_{Ed} + N_{Ed}[0,5a_p - (a_p - d)] = N_c(d - 0,5y) \qquad (3.78)$$

From the latter equation, it is possible to calculate the depth of the concrete section in compression:

$$M_{Ed} + N_{Ed}(d - 0,5a_p) = l_{eff}\, y f_{jd}(d - 0,5y)$$

$$\Rightarrow 0,5 l_{eff} f_{jd} y^2 - l_{eff} f_{jd}\, dy + [M_{Ed} + N_{Ed}(d - 0,5a_p)] = 0$$

$$\Rightarrow y = \frac{l_{eff} f_{jd}\, d \pm \sqrt{(-l_{eff} f_{jd}\, d)^2 - 2 l_{eff} f_{jd}[M_{Ed} + N_{Ed}(d - 0,5a_p)]}}{l_{eff} f_{jd}} \qquad (3.79)$$

where l_{eff} is the effective width of the base plate on the compression side (see Figure 3.31).

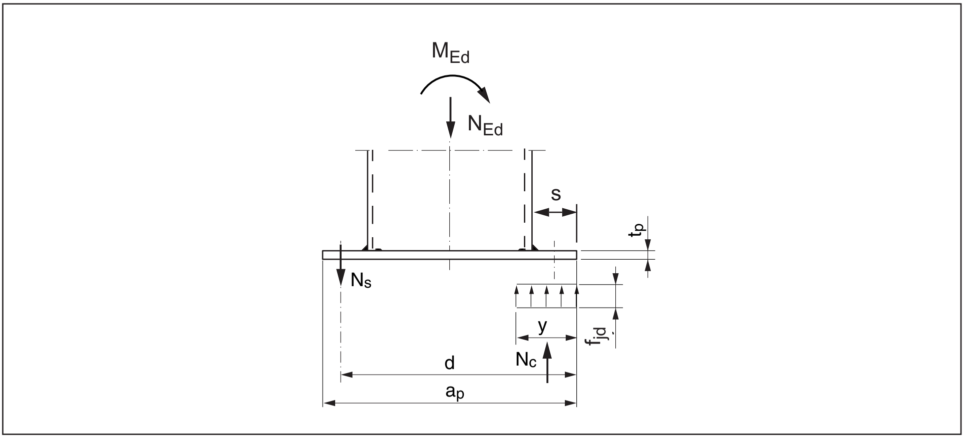


Figure 3.32 Structural model of a column subject to compression and bending

The thickness of the base plate shall be determined regarding both compression and tension side.

On the compression side, the thickness of the base plate shall be checked as presented in clause 3.5.1.

On the tension side, the tension resistance of the flange-plate joint formed by the base plate and the column shall be checked in the corresponding way as in clause 3.4.1.1.1. The joint shall in this case fulfill the following conditions:

- the bolt rows are positioned on two opposite sides of the flange-plate joint according to Figure 3.33
- the number of bolts is: $4 \leq n \leq 2(h / p_2) + 2$
- the nominal clearance of bolt holes is as specified in EN 1090-2 for normal round holes
- the positioning of the holes (Figure 3.33):
  $e_1 \geq 1{,}2 d_0$, where d_0 = the diameter of the bolt hole
  $e_2 \geq 1{,}2 d_0$ and $e_2 \leq 1{,}25 e_1$
  $e_3 \geq p_2 / 2$, the distance exceeding the lower limit shall not be, however, exploited in the resistance of the flange-plate
  $2{,}4 d_0 \leq p_2 \leq \min[14 t_p \,;\, 200 \text{ mm}]$, recommendation: $p_2 = (3...5) \times d_0$

It is advisable to keep the distance e_1 as small as possible so that $e_1 = (1{,}5 \ldots 2) \times d_0$ (however, at least 5 mm shall be left between the bolt head and the weld in the flange-plate) and for the distance e_2 the value $e_2 \approx 1{,}25 e_1$ should be chosen.

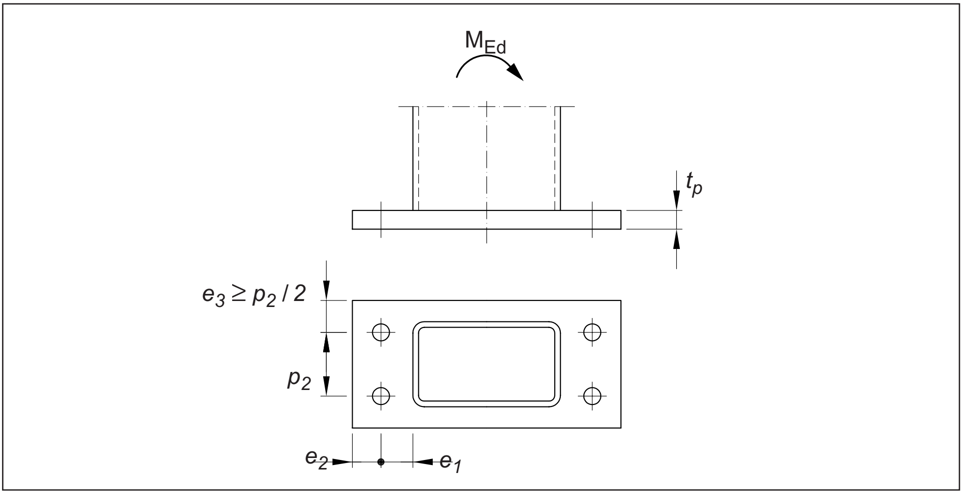


Figure 3.33 Positioning of the bolts in the base plate

The calculation is carried out as presented in clause 3.4.1.1.1, but in the case of base plate it is supposed that no prying forces exist because of the great elongation length of the anchor bolts and the thickness of the base plate. In this case the resistance of the base plate will be lower, and it is calculated instead of expression (3.34) as follows:

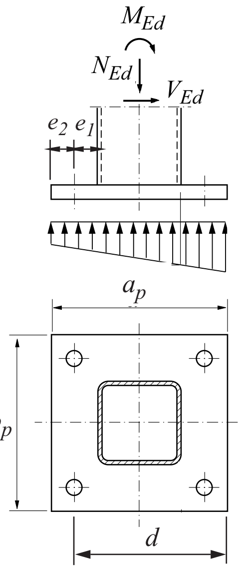
$$N_{1.Rd} = \frac{t_p^2 n_t}{K} \tag{3.80}$$

where n_t is the number of bolts on the tension side

The tension resistance of the anchor bolts $F_{t.Rd}$ shall be checked as presented in clause 3.4.1.1.1 (as a consequence of the thickness of the base plate, punching shear resistance $B_{p.Rd}$ will not become critical).

Also the resistance of the weld between the hollow section and the flange-plate shall be checked according to clause 3.3.

Example 3.17



Check the resistance of the column-to-foundation joint when using hollow section 200×200×8. The base plate is made of steel grade S355J2, and its dimensions are $a_p \times b_p$ = 400×400 and the thickness is 40 mm.

The steel grade of the hollow section column is SSAB Domex Tube Double Grade.

The bolts in the joint are not subjected to tension (this will be verified later), why the positioning of the bolts need not conform the conditions given for a flange-plate tension joint. Thus the joint may be constructed as presented in the adjacent figure.

The loads of the column are as follows:

N_{Ed} = 1500 kN
M_{Ed} = 35 kNm
V_{Ed} = 100 kN

Cross-section parameters:

$I_y = 3566 \cdot 10^4 \text{ mm}^4$ (Annex 11.1)
$W_{el.y} = 356{,}6 \cdot 10^3 \text{ mm}^3$ (Annex 11.1)
$A = 5924 \text{ mm}^2$ (Annex 11.1)

Plate dimensions:

t_p = 40 mm e_1 = 50 mm
a_p = 400 mm e_2 = 50 mm
b_p = 400 mm d = 350 mm

$f_{yp} = 355 \text{ N/mm}^2$, $f_{up} = 490 \text{ N/mm}^2$ when $t \leq 40$ mm

$\gamma_{M0} = 1{,}0$ (Table 2.5)
$\gamma_{M2} = 1{,}25$ (Table 2.5)
$\beta_w = 0{,}9$ (Table 3.8)

A. Base plate design:

<u>*Thickness of the base plate in respect to the compression side*</u>:

Strength of concrete:
C30/37

$f_{ck} = 30 \text{ N/mm}^2$
$\gamma_C = 1{,}5$ [1,2,3]
$\alpha_{cc} = 0{,}85$ [1,2,3]

Design strength of concrete's bearing pressure:

$$f_{jd} = \beta_j \cdot k_j \cdot \alpha_{cc} f_{ck} / \gamma_C = (2/3) \cdot 1 \cdot 0,85 \cdot 30/1,5 = 11,3\ N/mm^2$$

Extension of the bearing area:

$$c = t_p \sqrt{\frac{f_{yp}/\gamma_{M0}}{3 f_{jd}}} = 40 \cdot \sqrt{\frac{355/1,0}{3 \cdot 11,3}} = 129,4\ mm$$

Effectiveness of the base plate's bearing area inside the perimeter of the hollow section:

$$2c = 2 \cdot 129,4 = 258,8\ mm \geq b - 2t = 200 - 2 \cdot 8 = 184\ mm$$

$\Rightarrow$ *the bearing area inside the perimeter of the hollow section is fully effective*

Effectiveness of the base plate's bearing area outside the perimeter of the hollow section:

$$b + 2c = 200 + 2 \cdot 129,4 = 458,8\ mm \geq b_p = 400\ mm$$

$$h + 2c = 200 + 2 \cdot 129,4 = 458,8\ mm \geq a_p = 400\ mm$$

$\Rightarrow$ *the bearing area outside the perimeter of the hollow section is fully effective*

$\Rightarrow$ *the entire base plate is fully effective, hence:*
$l_{eff} = b_p = 400\ mm$

Extension of the base plate outside the perimeter of the column:

$$s = e_1 + e_2 = 50 + 50 = 100\ mm \leq c = 129,4\ mm$$

Check the thickness of the base plate in respect to compression:

$$M_{p.Ed} = \frac{l_{eff}\, s^2 f_{jd}}{2} = \frac{400 \cdot 100^2 \cdot 11,3}{2} = 22,6\ kNm$$

$$t_p \geq \sqrt{\frac{6 M_{p.Ed}}{l_{eff} \cdot f_{yp}/\gamma_{M0}}} = \sqrt{\frac{6 \cdot 22,6 \cdot 10^6}{400 \cdot 355/1,0}} = 30,9\ mm$$

$\Rightarrow t_p = 40\ mm = OK$

Thickness of the base plate in respect to the tension side:

First, check whether the anchor bolts will be subjected to tension when assuming the ultimate limit state:

$$y = \frac{l_{eff} f_{jd} d \pm \sqrt{(-l_{eff} f_{jd} d)^2 - 2 l_{eff} f_{jd} [M_{Ed} + N_{Ed}(d - 0,5 a_p)]}}{l_{eff} f_{jd}}$$

$$= \frac{400 \cdot 11,3 \cdot 350 \pm \sqrt{(-400 \cdot 11,3 \cdot 350)^2 - 2 \cdot 400 \cdot 11,3 \cdot [35 \cdot 10^6 + 1500 \cdot 10^3 \cdot (350 - 0,5 \cdot 400)]}}{400 \cdot 11,3}$$

$$= 263,7 \text{ mm (or 436,3 mm)}$$

$N_c = l_{eff} \cdot y \cdot f_{jd} = 400 \cdot 263,7 \cdot 11,3 = 1192 \text{ kN}$ *(compression)*

$N_s = N_c - N_{Ed} = 1192 - 1500 = -308 \text{ kN}$

⇒ the anchor bolts are not subjected to tension

⇒ the thickness of the base plate need not be checked in respect to tension

If the anchor bolts would be subjected to tension, the positioning of the bolts should fulfill the geometrical conditions presented in clause 3.5.2.

B. Anchor bolts:

Since the anchor bolts are not subjected to tension, only shear force needs to be considered in design. Choose ribbed steel bolts 4×M30 from the table of the Manufacturer. Check the resistance of the anchor bolts to conform the Manufacturer's instructions. The normal force resistance specified by the Manufacturer takes into account also the anchorage resistance. If the applied concrete differs from the strength class specified in the Manufacturer's table, a correction factor has to be applied. The Designer of the foundation designs also the reinforcements for the foundation, wherein the chosen anchor bolt type has an impact.

The Manufacturer's table is based on concrete strength class C25/30, thus the resistances given in the table can be directly adopted for the bolts.

Tension resistance of the bolts:

According to the Manufacturer's table the tension resistance of the bolt is:

$N_{t.Rd} = 222,1 \text{ kN}$ *OK (the bolts are not subjected to tension)*

Shear resistance of the bolts:

The shear force V_{Ed} is uniformly distributed to all four bolts. The shear force per bolt is thereby:

$$V_{Ed} = 100,0/4 = 25,0 \ kN$$

According to the Manufacturer's table the shear resistance of the chosen bolt is:

$$F_{v.Rd} = 27,2 \ kN \geq 25,0 \ kN \qquad OK$$

Combined effect of normal force and shear force:

The combined effect does not need to be checked in the finished foundation.

The 300 mm centre-to-centre distance of the bolts satisfies the minimum distance declared by the Manufacturer. Also the required minimum edge distances have to be checked, and appropriate deductions for the resistances shall be carried out according to the Manufacturer's instructions, when needed.

In addition, the resistance of the anchor bolts to the loads applied under the erection period has to be checked (possible flexural buckling of the bolts).

C. Weld design:

The welds in the flange of the column:

Check whether the welds in the column will be subjected to tension. On the tension side of the bending moment the stress at the centreline of the column wall thickness is:

$$\sigma_x = \frac{M_{Ed}}{I_y} \cdot \frac{h-t}{2} - \frac{N_{Ed}}{A} = \frac{35 \cdot 10^6}{3566 \cdot 10^4} \cdot \frac{200-8}{2} - \frac{1500 \cdot 10^3}{5924} = 94,2 - 253,2 = -159,0 \ kN$$

$\Rightarrow$ the bending moment is not big enough to cause tension in the column and its welds.

Compression can be transmitted to the foundation by exploiting contact in bearing, hence the welds need to be designed only for tension and shear forces. In this case contact bearing must be achieved between the base plate and the column base. This must be specified in the manufacturing drawings.

The welds in the webs of the column:

When the compression is transmitted by contact in bearing, the welds in the column webs can be designed to carry only the shear forces.

Try throat thickness 3 mm which is the smallest permitted throat thickness for a load carrying fillet weld (see clause 3.3.3):

$$\tau_{||} = \frac{V_{Ed}}{2ah} = \frac{100 \cdot 10^3}{2 \cdot 3 \cdot 200} = 83,3 \ N/mm^2$$

When the weld is subjected to shear alone, the design condition for the weld is (the ultimate tensile strength of the weaker part to be joined shall be adopted for the ultimate tensile strength of the weld):

$$\sqrt{\sigma_{\perp}^2 + 3(\tau_{\perp}^2 + \tau_{\parallel}^2)} = \sqrt{0 + 3 \cdot (0 + 83,3^2)} = 144,3 \text{ N/mm}^2$$

$$\leq \frac{f_u}{\beta_w \gamma_{M2}} = \frac{490}{0,9 \cdot 1,25} = 435,6 \text{ N/mm}^2 \quad OK$$

In respect to the cooling rate of the weld (see clause 3.3.3.):

$a \geq \sqrt{t}\ mm - 0,5\ mm = \sqrt{40} - 0,5 = 5,8\ mm$ *(t = thicker of the parts to be joined)*

$\Rightarrow a = 6$ *mm shall be chosen around the whole column*

In addition to the preceding verifications for the join'ts resistance, also the resistance of the column itself has to be checked to compression, bending, shear, and their interaction.

The concrete foundation shall be designed according to Part EN 1992-1-1 of Eurocode.

Comparison S420 vs S355:

The assessments carried out in this Example are focused on the base plate and the anchor bolts. Therefore the steel grade of the hollow section column may have influence only on the design of the welds between the column and the base plate. In respect to the welds however, in the calculation procedure in this Example it is the base plate that is governing, no matter which steel grade (S355J2H or S420MH) shall be chosen as design basis for the hollow section column

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4. FATIGUE RESISTANCE OF HOLLOW SECTION STRUCTURES

Hollow section structures always contain, in practice, also welded joints. The joints of lattice members are usually executed by welding the hollow sections directly to each other. The joints of beams and columns are in most cases executed as bolted joints, but even then there are first various joint-plates, flange-plates or end-plates welded to the hollow sections to enable a bolted joint. The fatigue resistance of the structural hollow section itself is seldom smaller than the fatigue resistance of the joint.

In fatigue design of hollow section structures, basically the same provisions should be applied as for welded structures in general. In fatigue design of hollow sections there are, however, some special features included which are presented in this Chapter amongst the provisions for fatigue design of welded structures, as necessary.

4.1 Fatigue loading

Fatigue loading may lead to failure of a structure within a certain time period, even though the resistance of the structure for static loading were sufficient. Fatigue loading may vary in direction, magnitude or location, and the variation may have constant amplitude or variable amplitude (Figure 4.1). A loading with constant amplitude causes at the point under consideration a constant stress fluctuation. Such kind of situation may be caused, for example, by machines working within a certain rotational speed domain. Usually fatigue loading has variable amplitude, which means that stress fluctuation at the considered location varies over time. For example, the loading caused by a crane to the crane runway beam is variable-amplitude loading, as well as the loading caused by vehicles to a bridge.

In dynamically loaded structures, the effect of vibration on the stresses must be taken into account. The increase of stress is significant, if the natural frequency of the structure is near to the vibration frequency of the load. In practice, structures are usually designed in such a manner that the lowest natural frequency is higher than the frequency of the dynamic load. In this way, the stress peaks due to resonance can be prevented. The frequency of the dynamic load may also be higher than the natural frequency, if the resonance frequency is passed through rapidly (e.g. machinery foundations).

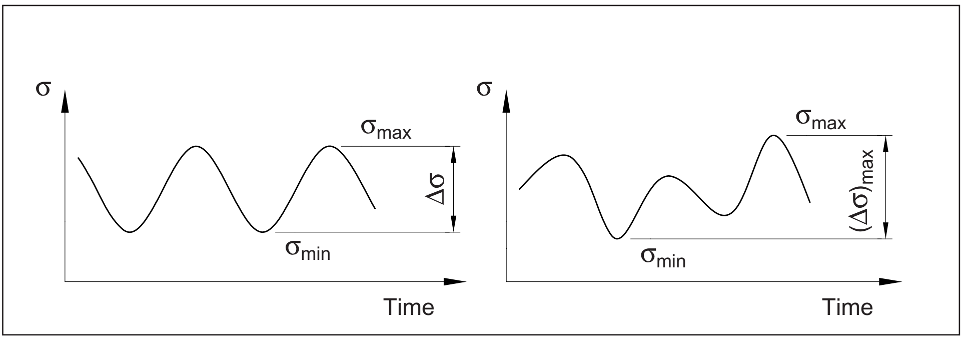


Figure 4.1 Constant-amplitude and variable-amplitude loading

4.2 Fatigue of a welded structure

There are always small initial cracks in a welded structure (Figure 4.2), which grow and propagate under influence of fatigue loading. Moreover, a crack may nucleate in a sound material due to fatigue loading, growing and propagating in the same way as the initial crack. High stress concentrations are developed at the tips of a crack promoting the growth of crack. Additionally the discontinuities of the structure create stress peaks (Figure 4.3). In a welded structure the critical place is the weld toe, i.e. the line between the weld and the parent metal. The crack may nucleate in the weld or adjacent to it, from where the damage can then propagate. The fatigue resistance of a welded joint is normally lower than that of the parent metal, why the quality of welding has a considerable effect on the fatigue resistance of the whole structure.

The nucleation of the crack and its growth begins with the first stress fluctuation. The number of stress cycles leading to rupture, i.e. the lifetime of the structure, depends on the character of the loading. In principle it is the tensile stress, not compression, that causes the fatigue damage. Normally, a loading where the stress fluctuation is partially on the compression side is more favourable in respect to fatigue, than if the whole stress fluctuation is on the tension side. In addition, when comparing different cases having equal stress fluctuations, the working life is shorter when having a higher mean stress level [1].

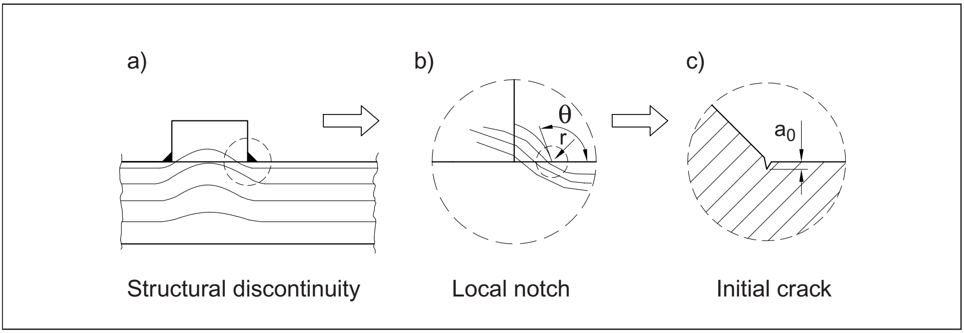


Figure 4.2 Different types of discontinuities in a welded structure and typical place of an initial crack [2]

The axiom above does not hold true in all cases but only for materials not having residual stresses. When it comes to the fatigue phenomenon, the most important factor on welded joints is the magnitude of the stress range $\Delta\sigma$. The role of the nominal mean stress is minor in structures in welded state, because the residual stresses caused by welding keep the real stress level high. The fatigue standards for welded structures in welded state are therefore based on assumption, that the stress fluctuates down from the tensile yield strength, no matter what is the sign of the nominal stress [1,2].

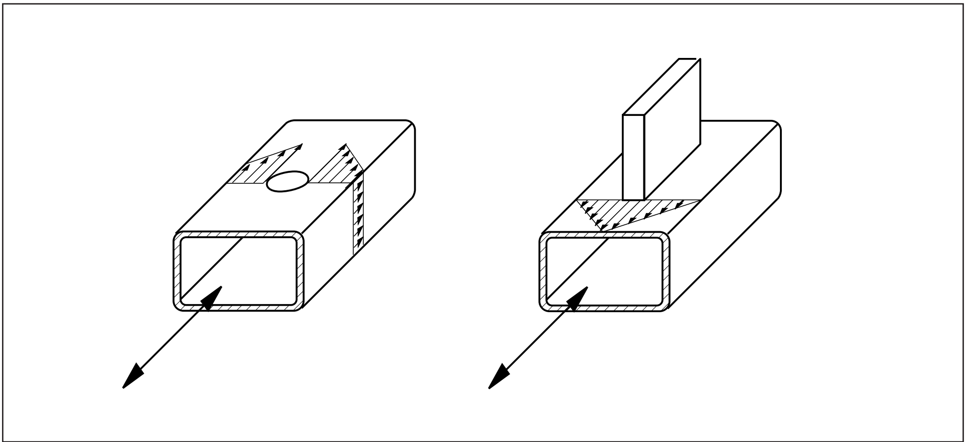


Figure 4.3 Discontinuities in structure

The fatigue resistance of a welded structure depends on the following factors:

- stress range
- number of stress cycles
- discontinuities in the structure (Figure 4.3)
- geometry of the welds
- size of the initial crack
- residual stresses and
- toughness of the steel.

However, the fatigue resistance of a structure in welded state is not dependent on the strength of the steel used: as stated above, in a welded structure there are always small initial cracks, and their growth takes place in all weldable structural steels in practically same rate independently of the yield strength of the steel. High strength steel is profitable only if it is possible to remove the initial cracks, since then the nucleation of the crack to the surface which is free from initial defects becomes more difficult [2].

In a fatigue-loaded welded structure, increase of the yield strength is profitable in case the portion of the permanent loads of the structure is high, or if the fatigue loading contains single high maximum loads, for which the static resistance of a lower grade steel is not sufficient [3].

4.3 Stress range spectrum, Reservoir method

The real stress fluctuation has rarely constant amplitude, but the stresses have random variation i.e. the stress has variable amplitude. For design, the variable-amplitude loading needs to be converted to usable form. There are many methods to convert the stress history into stress range spectrum (stress range accumulation), of which the so-called Reservoir method is commonly in use. In Reservoir method (also called as water tank method) the stress fluctuations are sorted out into stress range groups using suitable scaling, and the number of the cycles for each stress range group is counted. When sorted out to the order of magnitude, the stress fluctuations form the stress range spectrum according to Figure 4.4, which can be utilised in fatigue design when calculating the design value of the stress ranges or the Palmgren-Miner damage sum [1].

It is also possible that standards contain standardized stress range spectra. For example, for crane supporting structures there are in EN 1991-3 standardized stress range spectra presented for different crane classes using damage equivalent factors.

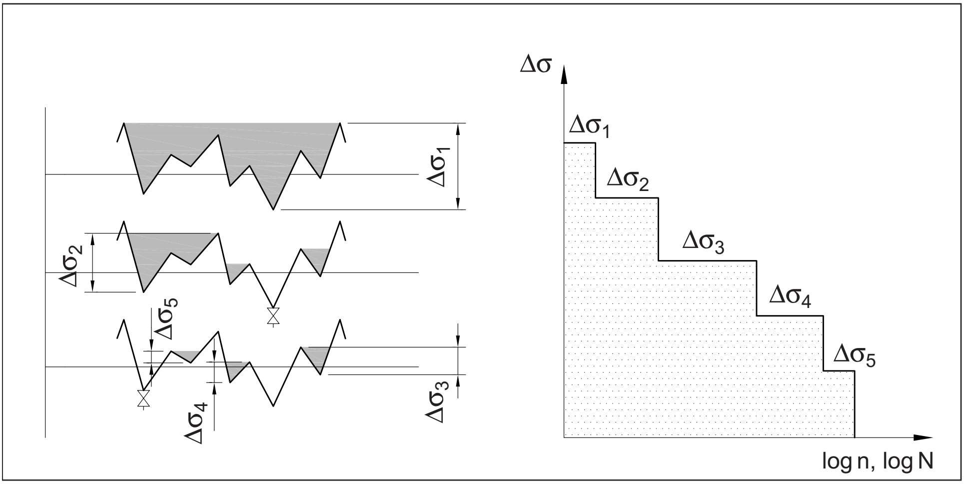
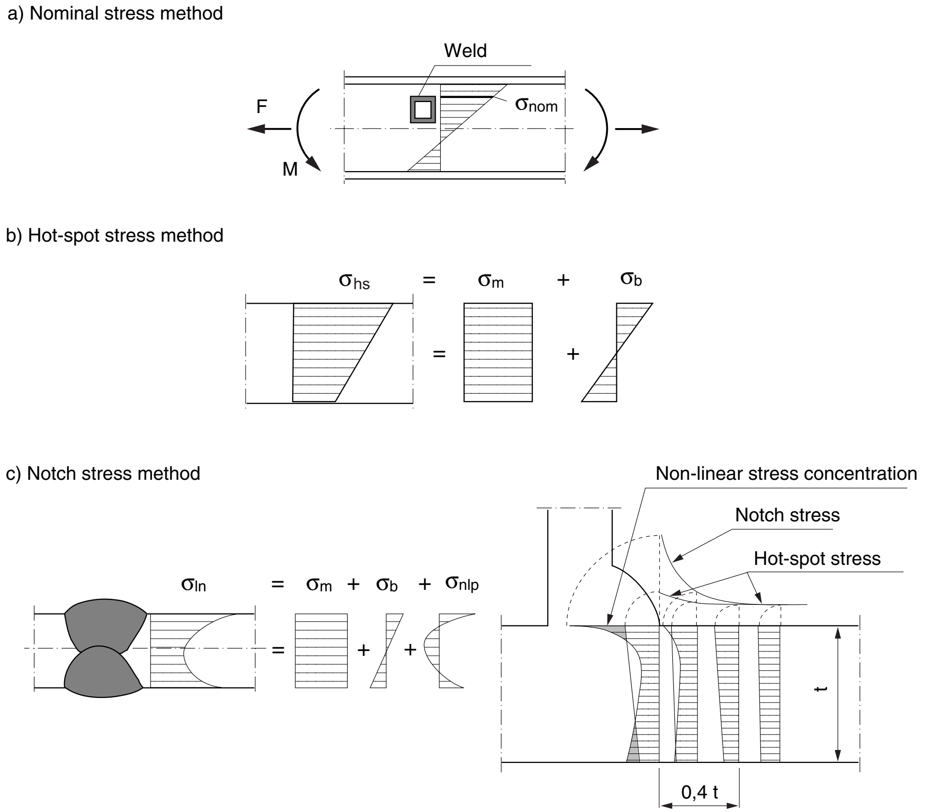


Figure 4.4 Reservoir method and stress range spectrum. In Reservoir method the stress history is imagined as a water tank full of water. The largest stress range $\Delta\sigma_1$ corresponds to lowering of the water level, when the drain valve is opened at the lowest place. Next, a valve is opened at a place where the next largest water level lowering is attained, and the result is $\Delta\sigma_2$. The process is continued, until all water is drained away [1,2]

4.4 Fatigue analysis methods for a welded structure

The methods used in fatigue analysis can basically be divided into two main groups [1]:

- methods basing on Wöhler curves i.e. S-N curves
- methods basing on fracture mechanics



σ_{nom}	is the nominal stress (stress according to the linear theory of elasticity)
σ_{hs}	is the hot-spot stress (stress at the critical point of the discontinuity, also called geometric stress)
σ_m	is the nominal or membrane stress
σ_b	is the shell bending stress
σ_{ln}	is the notch stress
σ_{nlp}	is the non-linear stress concentration due to the notch

Figure 4.5 Different types of stresses applied in fatigue assessment [4]

Methods basing on Wöhler curves i.e. S-N curves are:

- nominal stress method
- geometric (hot spot) stress method
- notch stress method
- method basing on the strain at notch location

In the methods basing on the S-N curves the fatigue strength is presented as curves according to Figure 4.7 obtained as a result of fatigue tests. Letter S stands for the word “Stress” (actually stress range) and letter N corresponds to the number of stress cycles leading to rupture.

In the **nominal stress method** (Figure 4.5a), the stresses in the structure are calculated according to the theory of elasticity and the effects of structural discontinuities are not taken into account, because they are already included in the loading tests. However, such macro-geometric effects which are not included in test results, should be taken into account when calculating the stresses.

Hot spot stress method (Figure 4.5b) is applicable only to such cases where the crack growth begins from the weld toe. The growth of the crack beginning from the root side will normally be assessed using nominal stresses. Hot spot stress is the stress at a critical point of the structural discontinuity, being therein dependent on the geometry of that point, and therefore called as "geometric" stress. The crack is supposed to nucleate in the discontinuity point. The geometric stress can be split into two parts, the membrane stress and the shell bending stress. The geometric stress includes the stress concentration caused by the discontinuity of the structure and is thus greater than the nominal stress at the critical point (hot spot point). The critical point is normally located at the weld toe. The structure is modelled using a suitable FEM program to determine the stresses, or the stresses are measured with strain gauges at the place to be investigated. Because the aim is to measure structural stress that does not include the peak stress existing adjacent to the notch, the strain gauges must not be set too near to the weld toe (the effect of the notch at the weld toe is included in the experimentally determined S-N curve). Depending on the detail to be investigated, two or three measuring points are used. If two measuring points are used, the hot spot stress is determined by measuring the stresses according to Figure 4.6 at the distance of 0,4 t and 1,0 t from the weld toe (t is the plate thickness) and using linear extrapolation to the point to be investigated. If three measuring points are used, extrapolation is performed in a non-linear manner. In some cases the hot spot stress can be calculated also by multiplying the nominal stress by a concentration factor. Concentration factors are presented in the literature [2,5,6]. Different symbols are used for concentration factor in different sources, such as k_f, K_s or SCF (Stress Concentration Factor). As a result of the multitude of the symbols, it should always be ascertained that the given stress concentration factor is intended specifically for calculating hot spot stress.

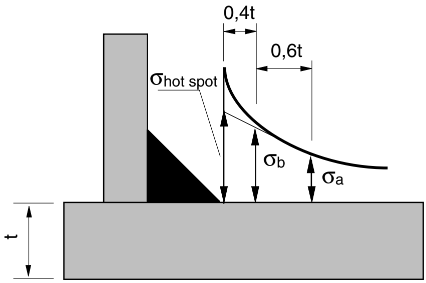


Figure 4.6 Extrapolation of hot spot stress when using two measuring points

Notch stress method (Figure 4.5c) is applicable to cases where fatigue begins either from the weld toe or from the root side of the weld. The notch stress is the real stress at the bottom of the notch. To calculate the notch stress, the non-linear peak stress, which is normally restricted within a distance of about $0{,}3t - 0{,}4t$ from the bottom of the notch, is added to those stresses present in the hot spot stress (the membrane stress and the shell bending stress). To calculate the notch stresses the structure shall be modelled using a suitable FEM program [2].

By using **fracture mechanics**, the fatigue life can be calculated on the basis of the growth rate of the crack. The size and shape of the initial crack have a considerable effect on the working life of the structure.

The above presented methods are described in more details e.g. in [2,3].

The fatigue assessment presented in Part EN 1993-1-9 of Eurocode is based mainly on the nominal stress method, which can also be used on structural hollow sections.

In Annex B of EN 1993-1-9 there are also presented, for some cases, the detail's fatigue categories for use with the hot spot stress method. However, in respect to hot spot stress method Eurocode is very brief, and instructions for hollow sections are missing, although the hot spot stress method is especially useful for the diverse forms of hollow section joints. In this Chapter later on, Eurocode-compatible hot spot stress design provisions are presented for lattice joints of structural hollow sections, based mainly on [5,6,7].

Other methods, such as the method basing on the strain at notch location or the method basing on fracture mechanics, are not incorporated into EN 1993-1-9.

4.5 S-N curves and their use in fatigue assessments

The instructions and formulae to be presented in the following apply to the basic principles of the S-N curves without the partial safety factors for design. The partial safety factors needed in actual design calculations for the loads and for the material, shall be taken into account as presented in clause 4.6.

4.5.1 S-N curves for constant-amplitude loading

S-N curves (Figure 4.7) present the fatigue strength by using detail's fatigue categories (in this context shortly: detail category), which are characteristic for different structural details, corresponding to detail categories presented in Tables 4.4 - 4.9. The detail category is indicated by the symbol $\Delta\sigma_C$ which corresponds to the fatigue strength when the number of stress cycles is $N = 2 \cdot 10^6$, wherein the fatigue limit was traditionally thought to be reached.

In Part EN 1993-1-9 of Eurocode there are in total 14 different detail categories presented, having the range of 36...160. In some sources also the designation FAT (Fatigue) is used for the detail category. Only on non-welded products (mill edge plates and sections) it is possible to reach the best detail category 160. The detail category for longitudinally welded structural hollow sections (when $t \leq 12{,}5$ mm) is the very first below that, i.e. 140. In Eurocode's tables it can be seen, that the detail categories for all other members and details fall below that.

In fatigue tests carried out for different joint types and structural details, it has been discovered that on each stress range $\Delta\sigma$ the number N of cycles leading to fatigue falls fairly well on a log-log scale to a descending straight line, the slope of which is 1:3. When $\log(\Delta\sigma)$ is plotted on the vertical axis and respectively $\log(N)$ on the horizontal axis, the sloping part of the S-N curve complies with the equation [2]:

$$N \cdot (\Delta\sigma)^3 = C = constant \quad \Rightarrow N = \frac{C}{(\Delta\sigma)^3} \qquad (4.1)$$

where N is the number of cycles leading to fatigue
$\Delta\sigma$ is the stress range
C is a constant that is characteristic for each detail type

The constant C needed in expression (4.1) can be derived by the considered detail category as follows:

$$C = 2 \cdot 10^6 \cdot (\Delta\sigma_C)^3 \qquad (4.2)$$

In the tests performed on constant-amplitude loading, it has been later on ended up to the outcome that the actual fatigue limit, i.e. the stress range below which the working life can be supposed to be infinite, is placed at $N = 5 \cdot 10^6$ cycles, why the S-N curve for constant-amplitude loading turns there to horizontal (Figure 4.7, the part with dashed lines). For the corresponding stress range, i.e. for the fatigue limit of the constant-amplitude loading, the symbol $\Delta\sigma_D$ is used.

The fatigue limit $\Delta\sigma_D$ for the constant-amplitude loading can be calculated by the considered detail category from the formula:

$$\Delta\sigma_D = (2/5)^{1/3}\Delta\sigma_C = 0,737 \cdot \Delta\sigma_C \qquad (4.3)$$

4.5.2 S-N curves for variable-amplitude loading

On variable-amplitude loading the role of fatigue limit is smaller than in constant-amplitude tests. If the stress fluctuation is higher than at fatigue limit, and if the number of cycles is great enough, they nucleate to the bottom of the notch a sharp crack, which the stress fluctuations even smaller than the fatigue limit are able to make grow. The greater the crack manages to grow, the smaller stress fluctuations are able to make it grow further [2].

In S-N curves this becomes visible so, that at the fatigue limit of constant-amplitude loading the curve does not turn to horizontal, but continues further downwards. However, this takes place with a slope of 1:5, being thus less steep than in the first part of the curve, as presented in Figure 4.7. Consequently the curve is then called double-slope S-N curve. The fatigue limit of variable-amplitude loading $\Delta\sigma_L$ is reached at $N = 1 \cdot 10^8$, where the curve turns to horizontal. The stress fluctuations smaller than the fatigue limit $\Delta\sigma_L$ can be neglected in the fatigue assessment of variable-amplitude loading. The curves in Figure 4.7 are used in the nominal stress method also for structural hollow sections.

Corresponding to expression (4.1), the part of the S-N curve having the smaller gradient complies hereby with the following formula:

$$N \cdot (\Delta\sigma)^5 = C = constant \quad \Rightarrow N = \frac{C}{(\Delta\sigma)^5} \qquad (4.4)$$

The lower fatigue limit (cut-off limit) $\Delta\sigma_L$ for variable-amplitude loading (see Figure 4.7) is obtained from the formula:

$$\Delta\sigma_L = (5/100)^{1/5}\Delta\sigma_D = 0,549 \cdot \Delta\sigma_D = 0,405 \cdot \Delta\sigma_C \qquad (4.5)$$

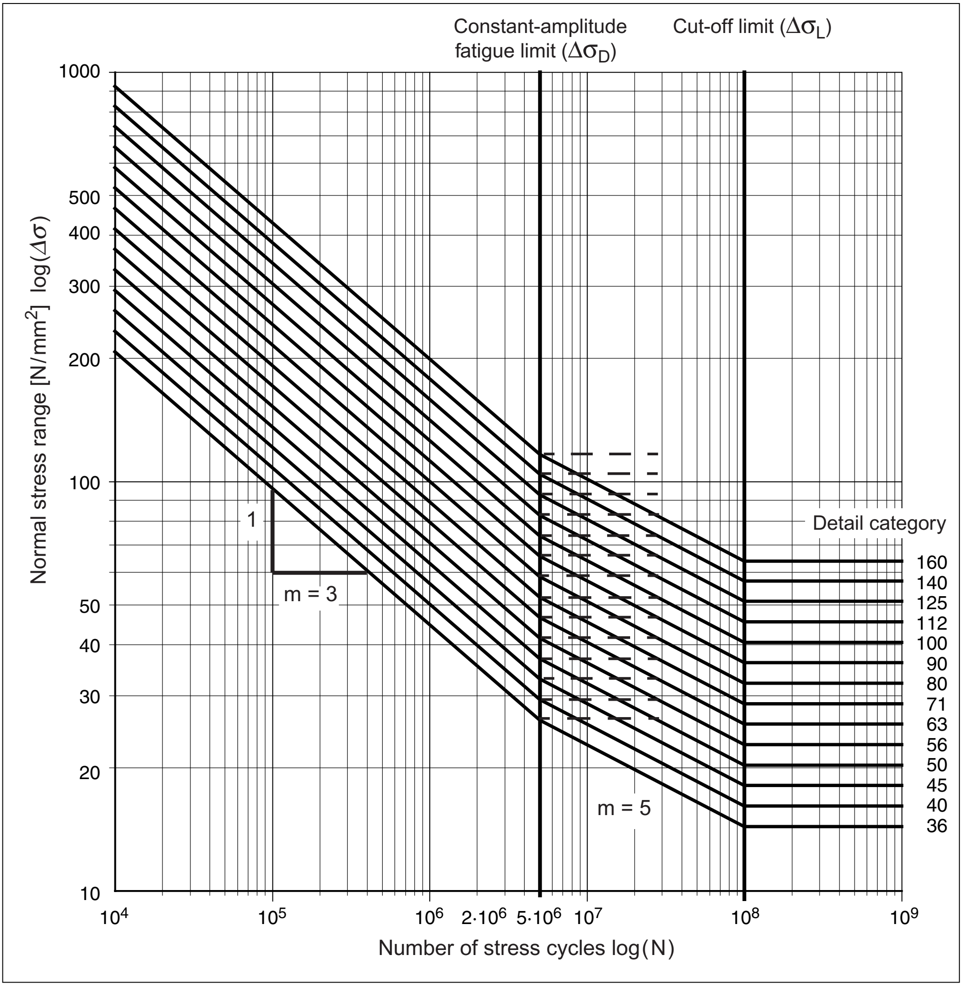


Figure 4.7 Fatigue strength for normal stress ranges presented on double-slope *S-N* curves on log-log scale

For shear stress ranges, a single-slope *S-N* curve having the slope of 1:5 is used on both constant-amplitude and variable-amplitude loading according to Figure 4.8. On shear stress ranges there is only one fatigue limit $\Delta\tau_L$ for each detail category, that takes place at load cycles $N = 1 \cdot 10^8$ where the curve turns to horizontal. The fatigue limit (cut-off limit) for a shear stress range can be calculated from the formula:

$$\Delta\tau_L = (2/100)^{1/5}\Delta\tau_C = 0,457 \cdot \Delta\tau_C \tag{4.6}$$

The curves in Figure 4.8 are used in the nominal stress method also for structural hollow sections, when subjected to shear stress range.

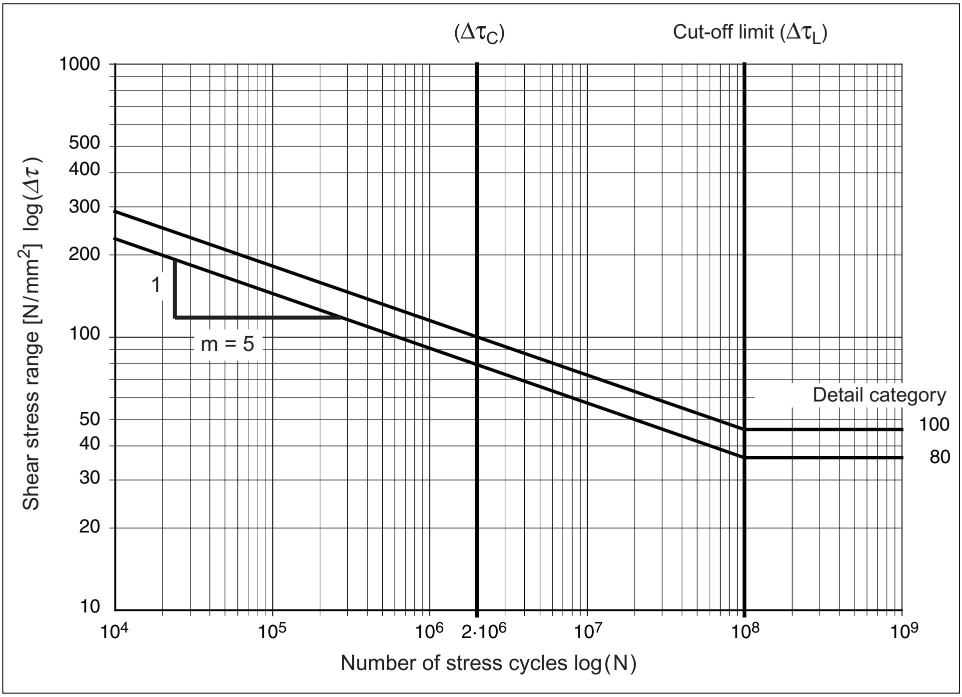


Figure 4.8 Fatigue strength for shear stress ranges presented on single-slope S-N curves on log-log scale

On variable-amplitude loading, the fatigue assessment for stress ranges having different magnitudes is carried out by using the so-called Palmgren-Miner rule, that is based on summation of cumulative damage so that every $\Delta\sigma_i$ -level damages the joint by the amount of n_i/N_i, whereby the fatigue life of the structure is considered to be reached when the cumulative damage reaches the value 1:

$$\sum\frac{n_i}{N_i} \le 1,0 \tag{4.7}$$

where n_i is the number of cycles having the stress range $\Delta\sigma_i$

N_i is the number of cycles (fatigue life) leading to rupture in the considered detail category according to S-N curve, when having the stress range $\Delta\sigma_i$

The calculation of Palmgren-Miner damage sum can be substituted by a so-called equivalent constant-amplitude stress range. The equivalent stress range can be determined either by setting the sum of partial damages equal to the total damage caused by the equivalent stress range and the original total number of cycles, or alternatively the equivalent stress range can be determined on the basis of any other reference number of load cycles. Often the equivalent stress range is determined by choosing the load cycle $N = 2 \cdot 10^6$ as the reference value. The equivalent stress range $\Delta\sigma_{E.2}$ corresponding to that is thereby:

$$\Delta\sigma_{E.2} = \sqrt[3]{\frac{\sum(n_i \cdot \Delta\sigma_i^3)}{2 \cdot 10^6}} \tag{4.8}$$

where n_i and $\Delta\sigma_i$ are cycles and stress ranges above the fatigue limit $\Delta\sigma_D$ of the detail category to be considered.

For double-slope S-N curve the expression (4.8) can be modified to the following form:

$$\Delta\sigma_{E.2} = \sqrt[3]{\frac{\sum(n_i \cdot \Delta\sigma_i^3) + \left(\frac{C}{5 \cdot 10^6}\right)^{-2/3} \cdot \sum(n_j \cdot \Delta\sigma_j^5)}{2 \cdot 10^6}} \tag{4.9}$$

where n_i and $\Delta\sigma_i$ are, in the detail category to be considered, stress ranges above the fatigue limit $\Delta\sigma_D$ and corresponding cycles

n_j and $\Delta\sigma_j$ are, in the detail category to be considered, stress ranges between the fatigue limit $\Delta\sigma_D$ and cut-off limit $\Delta\sigma_L$, and corresponding cycles (stress ranges below $\Delta\sigma_L$ are not taken into account)

C is the constant calculated from expression (4.2), corresponding to the detail category of the structural detail to be considered

4.6 Fatigue resistance of a welded structure according to Eurocode

4.6.1 Basic requirements

The provisions given in Part EN 1993-1-9 of Eurocode to verify the fatigue resistance of members, connections and joints are based on the fatigue strength determined in the S-N curves. The fatigue strengths of the S-N curves presented in the standard are based on full-scale fatigue tests on big-sized samples, so they include effects from geometric and structural discontinuities as well as effects of fabrication tolerances and residual stresses.

The provisions of EN 1993-1-9 are valid for all structural steels, stainless steels and unprotected weather resistant steels, when [8,9,10]:

- the stuctures are executed according to EN 1090-2 (quality of fabrication, tolerances etc.)
  (in the tables of EN 1993-1-9 related to different fatigue detail categories, there may be additional requirements given, see Tables 4.4 - 4.9 of this handbook)
- the material meets the toughness requirements according to EN 1993-1-10
- the structures operate under normal atmospheric conditions and with sufficient corrosion protection
- the temperature of the structures is at highest 150 ºC
- the nominal, modified nominal or geometric stress ranges due to frequent loads $\psi_1 Q_k$ are for normal stresses at highest $\Delta\sigma \le 1,5 f_y$ and for shear stresses at highest $\Delta\tau \le 1,5 f_y / \sqrt{3}$

In fatigue design, the reason to limit the maximum range of stresses is the need to prevent the risk of alternating plastification (cyclic plastification) in a fatigue-loaded member. With that in mind, the above presented limitation, basing on the design load $\psi_1 Q_k$ as written in Eurocode, is not sufficient and it is in contradiction with the instructions presented later on in Eurocode itself, where the actual calculation of stress ranges $\Delta\sigma$ and $\Delta\tau$ is determined by using the design load $\gamma_{Ff} Q_k$ (for example, see expressions (4.12) and (4.13)). Because of this, it is recommendable that the limitation of stress ranges should be determined by applying the corresponding design load as follows:

$$\Delta\sigma(\gamma_{Ff} Q_k) \le 1,5 f_y \qquad \textit{for normal stresses} \qquad (4.10)$$

$$\Delta\tau(\gamma_{Ff} Q_k) \le 1,5 f_y / \sqrt{3} \qquad \textit{for shear stresses} \qquad (4.11)$$

4.6.2 Reliability considerations

In fatigue design, the partial safety factor to be applied for the loads is given for example in EN 1993-2 (Steel bridges) and EN 1993-6 (Crane supporting structures), where the value γ_{Ff} = 1,0 is recommended. In general the value 1,0 can be applied for a fatigue load if not specifically otherwise stated.

The partial safety factors for resistance γ_{Mf} are determined in fatigue design according to Table 4.1 depending on the applied design method and the consequences of the possible failure. The National Annex may give provisions regarding the choice of the design method and the partial safety factors of the resistance to be applied with it.

Finnish National Annex to standard EN 1993-1-9 [11]:

The values according to Table 4.1, as recommended in Eurocode, are used. Usually the Safe life method is used.

Table 4.1 The values for partial safety factor γ_{Mf} in fatigue design [8...11]

Assessment method	Consequence of failure	
	Low consequence	High consequence
Damage tolerant	1,00	1,15
Safe life	1,15	1,35
- In this table the recommended values given in EN 1993-1-9 have been presented. - Finnish National Annex: the recommended values given in Eurocode are used.		

Damage tolerant method, i.e. the principle allowing the growth of the crack, provides that by using prescribed obligatory inspections regularly throughout the whole life of the structure, the growing cracks shall be detected and shall be repaired. The designer shall determine the inspection instructions and the greatest permitted defect size taking into account the inspectability and actual use of the structure, as well as give instructions how to repair the defect. In addition, it is provided that in case the fatigue failure however should occur, the redistribution of loads can take place between the different members and that the remaining structure is capable to carry at least the applied load combination without collapse [8,9,10].

Safe life method gives an acceptable level of reliability for the whole design life of the structure without the need of regular in-service inspection for fatigue damage. The Safe life method is applied in cases where local formation of cracks in one component could rapidly lead to failure of the structural element or the whole structure [8,9,10].

4.6.3 Fatigue assessments

Depending on the case, the fatigue assessment is carried out as follows [8,9,10]:

- by using **nominal stress range** according to the provisions given in Tables 4.4 - 4.9 for the structural detail (detail category) to be considered

- by using **modified nominal stress range** for the structural details, where in the fatigue design such kind of macro-geometric effects appear, which are not included in the considered case of Tables 4.4 - 4.9 (for example the cross-section changes rapidly nearby the crack nucleation point)

- by using a method basing on **the geometric (hot spot) stress**

4.6.3.1 Fatigue assessment by nominal stress method

The stresses are calculated at potential location of fatigue crack using loads at serviceability limit state according to the linear theory of elasticity. In Class 4 the stresses are calculated according to effective cross-section following the rules of Part EN 1993-1-5 of Eurocode [8,9,10].

In the calculation model of nominal stresses, all the effects of the discontinuities shall be considered, that have not been present in test specimens and thus are not implicitly included in the considered structural detail of the Tables 4.4 - 4.9.

The relevant stresses in the parent metal are:
- nominal normal stress σ
- nominal shear stress τ

The relevant stresses in the welds are according to Figure 4.9:
- normal stress transverse to the axis of the weld: $\sigma_{wf} = \sqrt{\sigma_{\perp f}^2 + \tau_{\perp f}^2}$
- shear stress longitudinal to the axis of the weld: $\tau_{wf} = \tau_{\| f}$

The method above differs from the method of EN 1993-1-8 presented herewith in clause 3.3.3 for fillet welds at ultimate limit state. A separate verification shall be done for both of the stresses σ_{wf} and τ_{wf}.

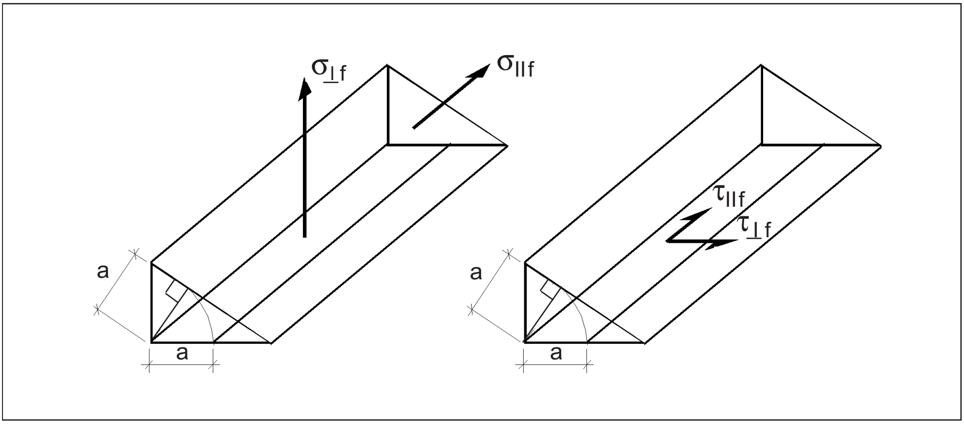


Figure 4.9 Relevant stresses in the fillet weld [8,9,10]

Welded joints of hollow sections are normally executed as full penetration butt welds, or as fillet welds designed to meet the equal strength condition (see clause 3.3). In both cases the most critical point in respect to fatigue resistance is at the weld toe where the fatigue crack will nucleate. In this case a separate fatigue assessment is normally not needed for the weld itself. Instead of that, the fatigue assessment is based on the stress fluctuations of the parent metal, no matter whether the fatigue assessment is performed by using the nominal stress method (in respect to the detail's fatigue category as given in the tables for the considered joint) or by using the hot spot stress method.

The design value of stress range to be used in fatigue assessments is $\gamma_{Ff}\Delta\sigma_{E.2}$, where $\Delta\sigma_{E.2}$ is the equivalent stress range at load cycle $N = 2 \cdot 10^6$ (see clause 4.5.2).

The design values of the nominal stress ranges $\gamma_{Ff}\Delta\sigma_{E.2}$ and $\gamma_{Ff}\Delta\tau_{E.2}$ are determined as follows [8,9,10]:

$$\gamma_{Ff}\Delta\sigma_{E.2} = \lambda_1 \cdot \lambda_2 \cdot \lambda_i \cdot \ldots \cdot \lambda_n \cdot \Delta\sigma(\gamma_{Ff}Q_k) \quad \textit{for normal stresses} \qquad (4.12)$$

$$\gamma_{Ff}\Delta\tau_{E.2} = \lambda_1 \cdot \lambda_2 \cdot \lambda_i \cdot \ldots \cdot \lambda_n \cdot \Delta\tau(\gamma_{Ff}Q_k) \quad \textit{for shear stresses} \qquad (4.13)$$

where the coefficients λ_i are damage equivalent factors that depend on the character of the loading and on the stress range spectrum. These are obtained from the relevant Parts of Eurocode, for example for crane supporting structures from EN 1991-3: Table 2.12.

If λ_i -factors are not known, the nominal stress range can be determined according to clauses 4.3 and 4.5. The peak values of the stress ranges that present less than 1 % of the total damage, and small stress ranges that fall below the cut-off limit $\Delta\sigma_L$, can be neglected [8,9,10].

Fatigue strength:

Adopting the symbols by EN 1993-1-9, the fatigue strengths $\Delta\sigma_R$ and $\Delta\tau_R$ are calculated for constant-amplitude nominal stress range as follows [8,9,10]:

$$\Delta\sigma_R^m N_R = \Delta\sigma_C^m \cdot 2 \cdot 10^6 \qquad with\ \ m = 3\ \ for\ \ N \leq 5 \cdot 10^6 \tag{4.14}$$

$$\Delta\tau_R^m N_R = \Delta\tau_C^m \cdot 2 \cdot 10^6 \qquad with\ \ m = 5\ \ for\ \ N \leq 1 \cdot 10^8 \tag{4.15}$$

For variable-amplitude loading the fatigue strengths of the normal stress $\Delta\sigma_R$ are calculated based on the double-slope S-N curve as follows [8,9,10]:

$$\Delta\sigma_R^m N_R = \Delta\sigma_C^m \cdot 2 \cdot 10^6 \qquad with\ \ m = 3\ \ for\ \ N \leq 5 \cdot 10^6 \tag{4.16}$$

$$\Delta\sigma_R^m N_R = \Delta\sigma_D^m \cdot 5 \cdot 10^6 \qquad with\ \ m = 5\ \ for\ \ 5 \cdot 10^6 \leq N \leq 1 \cdot 10^8 \tag{4.17}$$

The fatigue strengths according to expressions (4.14) - (4.17) correspond to the S-N curves presented in Figures 4.7 and 4.8, that are used also for structural hollow sections.

The fatigue test results for some structural details do not fully comply with the S-N curves of fatigue strength according to Figure 4.7. To avoid the risk of non-conservative design, the structural details in Tables 4.4 - 4.9 having detail categories marked with asterisk ($\Delta\sigma_C$*) are determined one detail category lower than the detail category corresponding to $2 \cdot 10^6$ cycles requires. Alternatively it is possible to do so, that the detail category of the structural detail marked with asterisk is increased by one category, provided that the constant-amplitude fatigue limit $\Delta\sigma_D$ is then determined according to Figure 4.10 as a reduced fatigue limit at $1 \cdot 10^7$ cycles, using the value m = 3 for the slope of the S-N curve [8,9,10].

The reduced fatigue limit $\Delta\sigma_{D.red}$ corresponding to load cycles $N = 1 \cdot 10^7$ can be calculated from the formula:

$$\Delta\sigma_{D.red} = (2/10)^{1/3}\Delta\sigma_C = 0,585 \cdot \Delta\sigma_C \tag{4.18}$$

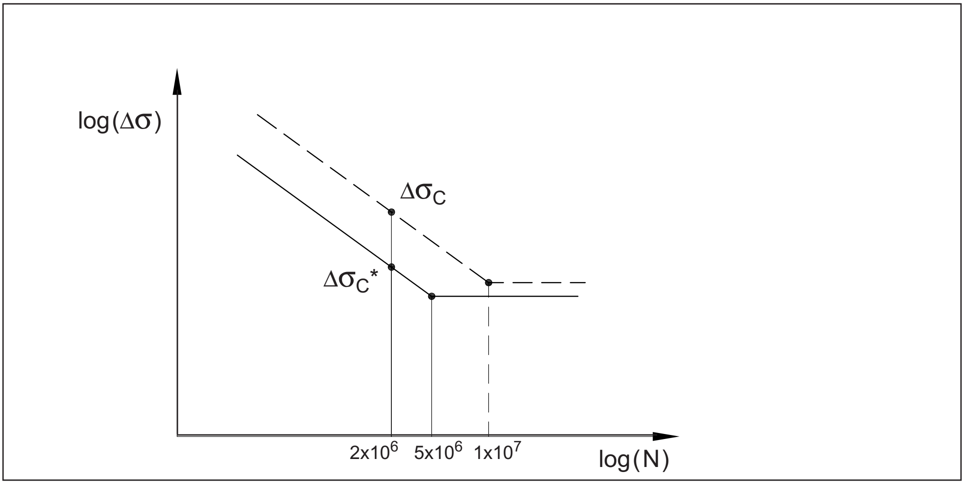


Figure 4.10 Alternative S-N curve for fatigue strength of structural details classified as $\Delta\sigma_C$* [8,9,10]

Fatigue verification:

The design conditions for fatigue verifications are [8,9,10]:

$$\frac{\gamma_{Ff}\,\Delta\sigma_{E.2}}{\Delta\sigma_C/\gamma_{Mf}} \leq 1,0 \qquad \textit{for normal stress ranges} \qquad (4.19)$$

$$\frac{\gamma_{Ff}\,\Delta\tau_{E.2}}{\Delta\tau_C/\gamma_{Mf}} \leq 1,0 \qquad \textit{for shear stress ranges} \qquad (4.20)$$

where $\gamma_{Ff}\Delta\sigma_{E.2}$ and $\gamma_{Ff}\Delta\tau_{E.2}$ are determined according to expressions (4.12) and (4.13)
γ_{Ff} is the partial safety factor for fatigue load (clause 4.6.2)
$\Delta\sigma_C$ and $\Delta\tau_C$ are detail categories of the considered structural detail according to Tables 4.4 - 4.9
γ_{Mf} is determined according to clause 4.6.2

Regarding some structural details, design conditions (4.19) and (4.20) shall be checked in respect to the principle stress ranges (see Table 4.7).

If not otherwise stated, with the structural details presented in Tables 4.4 - 4.9 also the following interaction formula shall be verified when having combined stress ranges $\Delta\sigma_{E.2}$ and $\Delta\tau_{E.2}$ [8,9,10]:

$$\left[\frac{\gamma_{Ff}\,\Delta\sigma_{E.2}}{\Delta\sigma_C/\gamma_{Mf}}\right]^3 + \left[\frac{\gamma_{Ff}\,\Delta\tau_{E.2}}{\Delta\tau_C/\gamma_{Mf}}\right]^5 \leq 1,0 \qquad (4.21)$$

Some structural details and corresponding detail categories presented in EN 1993-1-9 in respect to structural hollow sections and their applications, are presented in Tables 4.4 - 4.9.

4.6.3.2 Fatigue assessment by modified nominal stress method

In some situations the nominal stresses, stress ranges and fatigue strengths calculated according to clause 4.6.3.1 shall be modified using correction factors before fatigue verification. The provisions given in EN 1993-1-9 for the required modification are presented herewith in full, although in respect to structural hollow sections only the last case is needed in practice (Lattice joints of structural hollow sections).

Stress cycles in compression:

As presented on Figure 4.11, Eurocode allows the reduction of the compressive portion of the stress range by factor 0,6, if the residual stresses have been removed from the welded structure by stress relief, or if there are no welds in the structure [8,9,10].

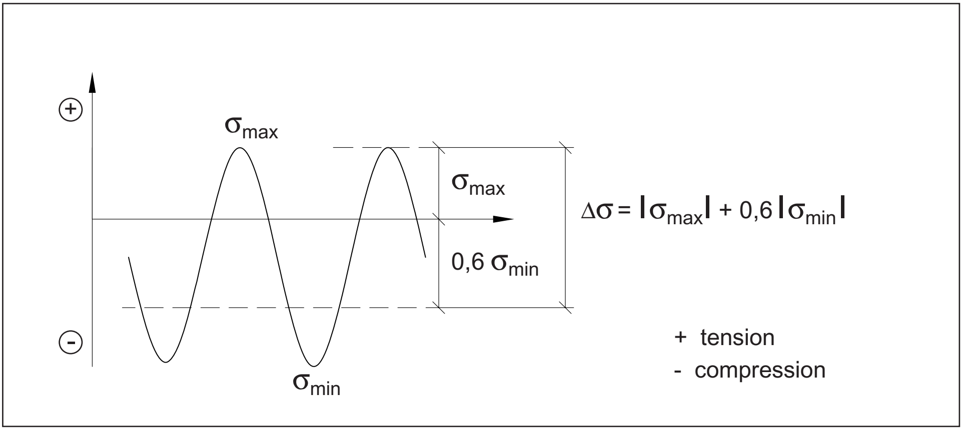


Figure 4.11 Reduction of the compressive portion of the stress range, when residual stresses have been relieved from the welded structure or there are no welds in the structure [8,9,10]

Size effect:

The size effect due to thickness or other dimensional effects shall be taken into account according to the provisions given in Tables 4.4 - 4.9. The fatigue strength is then determined as follows [8,9,10]:

$$\Delta\sigma_{C.red} = k_s \cdot \Delta\sigma_C \tag{4.22}$$

where the factor k_s is obtained from Tables 4.4 - 4.9.

The plate thickness has an effect on the fatigue strength in three ways [2]:
- as a geometric effect
- as a statistical effect
- as a technological effect

Geometric effect arises in the following way: The effect of notches on the surface of a thick material extends deeper into the material than a in thinner member. That is why a crack started to grow may proceed the growing a longer time in the region of the peak stress. Consequently the working life of a thick member becomes shorter than normal. In plated structures the thickness limit is supposed to be 25 mm, whereafter the fatigue strength is reduced for a thicker material [2].

Statistical effect arises from the fact that there exists more weld in heavy structures, and therefore a higher probability for an initial defect greater than normal to exist [2].

Technological effect arises from the fact that thicker materials are welded with greater values of the welding process, why residual stresses and initial defects may be greater [2].

In EN 1993-1-9 the effect of plate thickness has been considered through reduction factor k_s applied in expression (4.22) and determined as follows [8,9,10]:

$$k_s = \left(\frac{25mm}{t}\right)^{0,2} \quad for \;\; t > 25mm \tag{4.23}$$

Lattice joints of structural hollow sections:

The structural analysis of an uniplanar truss made of structural hollow sections can be performed using a simplified structural model where the chords are modelled as continuous but the joints between the brace members and chords are supposed to be pinned. Regarding the eccentricity of the joints the provisions given in Part EN 1993-1-9 of Eurocode are defective. In this handbook the interpretation is that the joint eccentricity can be ignored when it does not exceed the limits presented for lattice joints in Table 4.6.

In fact the joints have, however, stiffness that the simplified pinned model does not take into account. The stiffness causes to the members secondary bending moments. To take them into account the design value of modified nominal stress range $\gamma_{Ff}\Delta\sigma_{E.2}$ is determined as follows [8,9,10]:

$$\gamma_{Ff}\Delta\sigma_{E.2} = k_1 \cdot (\gamma_{Ff}\Delta\sigma^*_{E.2}) \tag{4.24}$$

where $\gamma_{Ff}\Delta\sigma^*_{E.2}$ is the design value of the stress range calculated using the simplified pinned model of the lattice

γ_{Ff} is the partial safety factor for fatigue load (clause 4.6.2)

k_1 is the magnification factor according to Table 4.3

Alternatively the effect of magnification factor k_1 can be taken into account by multiplying the member forces (or the stresses) obtained from structural analysis directly by that factor. The values of the magnification factor k_1 presented in Table 4.3 are approximate empirical or experimental values.

On the lattice joints of structural hollow sections, the same single-slope S-N curve having the slope 1:5 is applied for the normal stress ranges both for constant-amplitude and for variable-amplitude loading according to Figure 4.12. The stress ranges have in each detail category only one fatigue limit (cut-off limit) $\Delta\sigma_L$ that is placed at load cycles $N = 1 \cdot 10^8$, where the curve turns to horizontal. The fatigue limits according to lattice joint S-N curves are presented in Table 4.2, where the values are calculated from the formula:

$$\Delta\sigma_L = (2/100)^{1/5}\Delta\sigma_C = 0,457 \cdot \Delta\sigma_C \tag{4.25}$$

The stress ranges $\Delta\sigma$ can be modified to equivalent stress range $\Delta\sigma_{E.2}$ by applying formula (4.8) as follows: in the formula the value 3 as root and as exponent is substituted by value 5 to correspond the single-slope lattice joint S-N curves having the slope 1:5. The stress ranges below the cut-off limit $\Delta\sigma_L$ are not taken into account.

When the design value of the modified nominal stress range $\gamma_{Ff}\Delta\sigma_{E.2}$ has been determined according to above, the verification shall be performed as usual by using the design condition (4.19).

The fatigue assessment shall be performed separately for each of the members of the joint [8,9,10].

On lattice structures, the nominal stress method gives only a rough estimate of the fatigue resistance of the structure. For example, by the nominal stress method the combined effect of the stresses of the chord and brace members to the fatigue resistance is difficult to take into account in real. A more precise assessment using hot spot stress method is presented in clause 4.6.3.3 for the hollow section lattice joints.

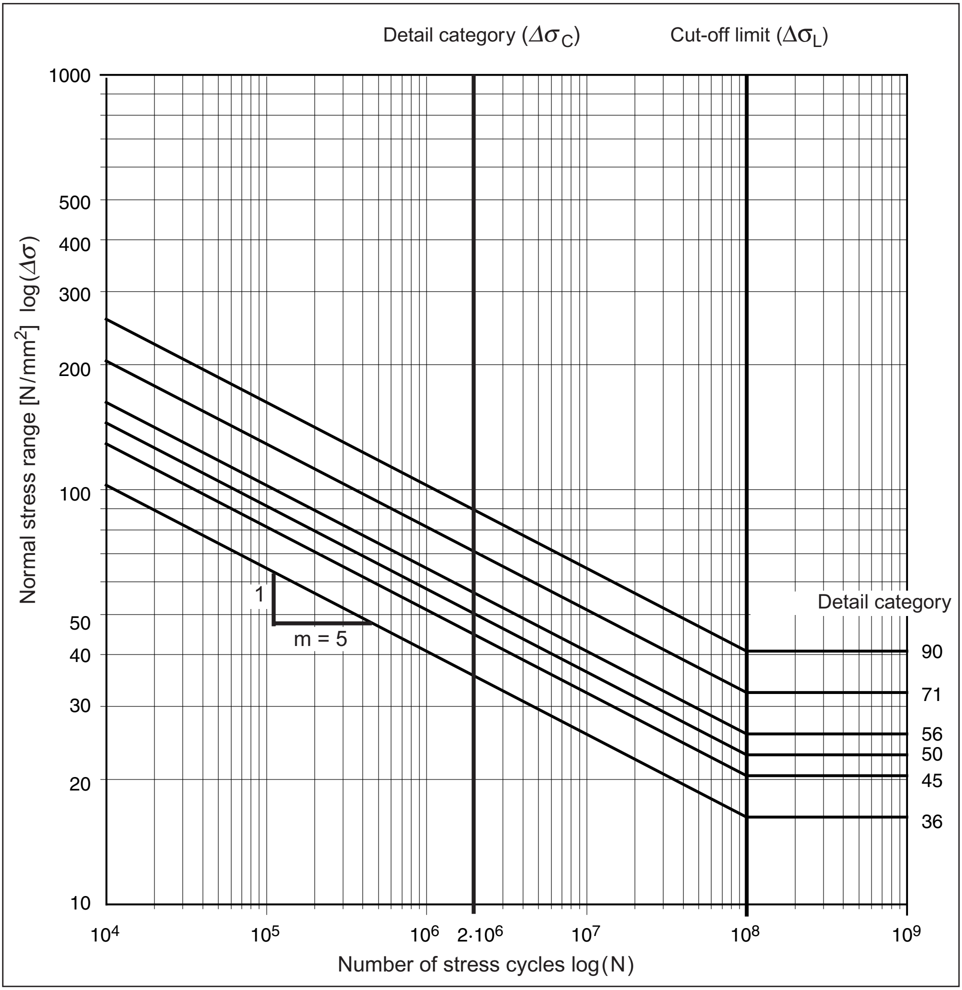


Figure 4.12 Single-slope S-N curves for hollow section lattice joints when using the nominal stress method

Table 4.2 Cut-off limit $\Delta\sigma_L$ for the nominal stress range applied in the S-N curves of hollow section lattice joints [7]

Detail category (N/mm^2)	Cut-off limit $\Delta\sigma_L$ (N/mm^2)
90	41
71	32
56	26
50	23
45	21
36	16

Table 4.3 k_1 -factors for hollow section lattice joints [8,9,10]

Type of joint		Chords	Verticals	Diagonals
Circular hollow sections				
Gap joint	K joint	1,5	–	1,3
	N joint / KT joint	1,5	1,8	1,4
Overlap joint	K joint	1,5	–	1,2
	N joint	1,5	1,65	1,25
Square and rectangular hollow sections				
Gap joint	K joint	1,5	–	1,5
	N joint / KT joint	1,5	2,2	1,6
Overlap joint	K joint	1,5	–	1,3
	N joint / KT joint	1,5	2,0	1,4

Table 4.4 Fatigue categories for structural hollow sections, longitudinally welded structural hollow sections [8,9,10]

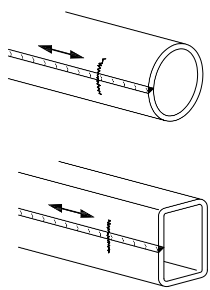
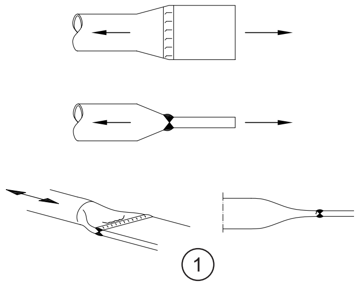
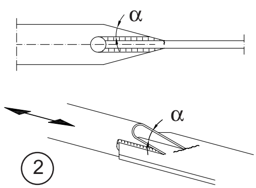
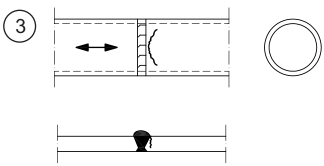
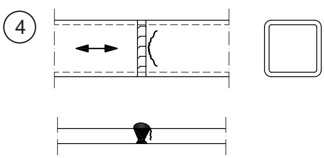
Detail category	Constructional detail	Description	Requirements
140		Automatic or fully mechanized longitudinal seam weld without stop/start positions in hollow sections. a)	Wall thickness t ≤ 12,5 mm
125		Automatic or fully mechanized longitudinal seam weld without stop/start positions in hollow sections. a)	Wall thickness t > 12,5 mm
90		With stop/start positions. a)	
a) The longitudinally welded structural hollow sections by SSAB do not contain stop/start positions of welds.			

Table 4.5 Splice joints of structural hollow sections, when $t \leq 12{,}5$ mm [8,9,10]

Detail category	Constructional detail	Description	Requirements
71		1) Tube-plate joint, tube's end flattened, butt weld (X-groove).	1) $\Delta\sigma$ is calculated in tube. Only valid for tube diameter less than 200 mm.
71 63	$\alpha \leq 45°$ $\alpha > 45°$ 	2) Tube-plate joint, tube slitted and welded to plate. Holes at end of slit.	2) $\Delta\sigma$ is calculated in tube. Shear cracking in the weld shall be verified using Table 4.8, detail 3).
71		Transverse butt welds: 3) Butt-welded end-to-end connections between circular hollow sections.	<u>Details 3) and 4):</u> - Weld convexity ≤ 10 % of weld width, with smooth transitions. - Welded in flat position, inspected and found free from defects outside the tolerances of EN 1090. - Classify 2 detail categories higher if $t > 8$ mm.
56		4) Butt-welded end-to-end connections between rectangular hollow sections.	

(continues)

Table 4.5 Splice joints of structural hollow sections, when t ≤ 12,5 mm [8,9,10] *(continued)*

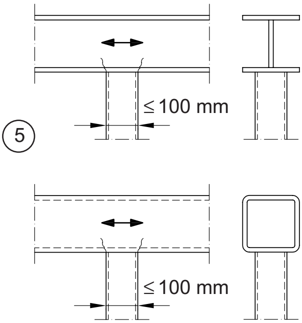
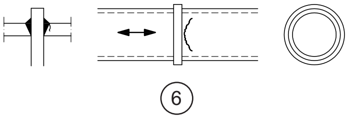
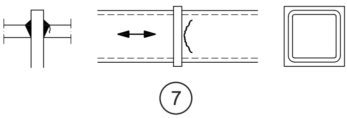
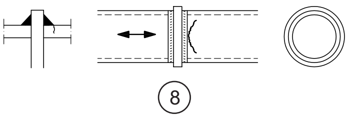
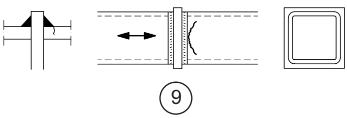
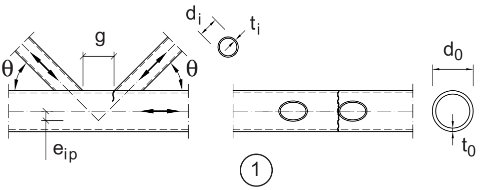
Detail category	Constructional detail	Description	Requirements
71		Welded attachments: 5) Circular or rectangular hollow sections, fillet-welded to another section.	5) - Non load-carrying welds. - Length of the weld in the direction of the stress l ≤ 100 mm. - Oher cases, see Table 4.7.
50		Welded splice joints: 6) Circular hollow sections, butt-welded end-to-end with an intermediate plate.	<u>Details 6) and 7):</u> - Load-carrying welds. - Welds inspected and found free from defects outside the tolerances of EN 1090. - Classify 1 detail category higher if t > 8 mm.
45		7) Rectangular hollow sections, butt-welded end-to-end with an intermediate plate.	
40		8) Circular hollow sections, fillet-welded end-to-end with an intermediate plate.	<u>Details 8) and 9):</u> - Load-carrying welds. - Wall thickness t ≤ 8 mm
36		9) Rectangular hollow sections, fillet-welded end-to-end with an intermediate plate.	

Table 4.6 Welded lattice joints of structural hollow sections, when $t \leq 8$ mm [8,9,10]

<table>
<tr><th>Detail category</th><th colspan="2">Constructional detail</th><th>Requirements</th></tr>
<tr><td>90
m = 5</td><td>$t_0 / t_i \geq 2,0$</td><td rowspan="2">Gap joints: K and N joints, circular hollow sections:
</td><td rowspan="8"><u>Details 1)...4):</u>
– Separate assessments needed for the chords and the braces.

– For intermediate values of the ratio t_0 / t_i the detail catogory shall be interpolated linearly between the given detail categories.

– Fillet welds are permitted for braces with wall thickness $t \leq 8$ mm.

– t_0 and $t_i \leq 8$ mm
– $35° \leq \theta \leq 50°$
– $(b_0 / t_0) \times (t_0 / t_i) \leq 25$
– $(d_0 / t_0) \times (t_0 / t_i) \leq 25$
– $0,4 \leq b_i / b_0 \leq 1,0$
– $0,25 \leq d_i / d_0 \leq 1,0$
– $b_0 \leq 200$ mm
– $d_0 \leq 300$ mm
– $-0,5h_0 \leq e_{ip} \leq 0,25h_0$
– $-0,5d_0 \leq e_{ip} \leq 0,25d_0$
– $e_{op} \leq 0,02b_0$
– $e_{op} \leq 0,02d_0$

(e_{op} is out-of-plane eccentricity

<u>Detail 2):</u>
$0,5(b_0 - b_i) \leq g \leq 1,1(b_0 - b_i)$
and
$g \geq 2t_0$

<u>Details 3) and 4):</u>
– $30\% \leq$ overlap $\leq 100\%$
– Overlap = $(q / p) \times 100\%$

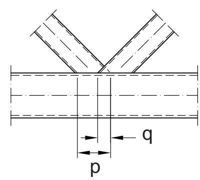
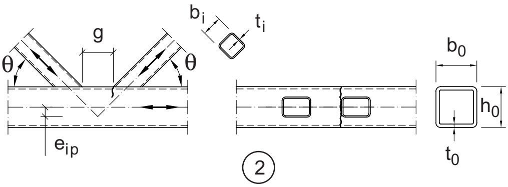
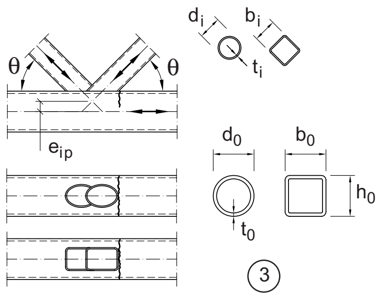
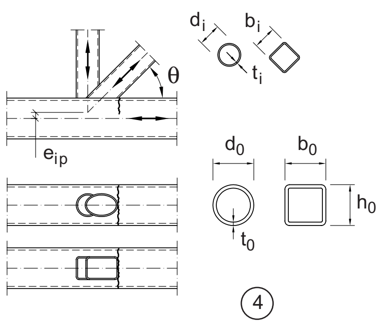
Definition of p and q:
</td></tr>
<tr><td>45
m = 5</td><td>$t_0 / t_i = 1,0$</td></tr>
<tr><td>71
m = 5</td><td>$t_0 / t_i \geq 2,0$</td><td rowspan="2">Gap joints: K and N joints, rectangular hollow sections:
</td></tr>
<tr><td>36
m = 5</td><td>$t_0 / t_i = 1,0$</td></tr>
<tr><td>71
m = 5</td><td>$t_0 / t_i \geq 1,4$</td><td rowspan="2">Overlap joints: K joints, circular or rectangular hollow sections:
</td></tr>
<tr><td>56
m = 5</td><td>$t_0 / t_i = 1,0$</td></tr>
<tr><td>71
m = 5</td><td>$t_0 / t_i \geq 1,4$</td><td rowspan="2">Overlap joints: N joints, circular or rectangular hollow sections:
</td></tr>
<tr><td>50
m = 5</td><td>$t_0 / t_i = 1,0$</td></tr>
</table>

Table 4.7 Welded attachments and stiffeners [8,9,10]

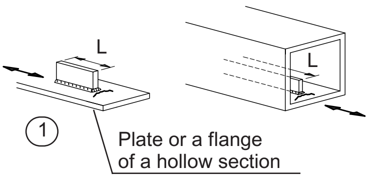
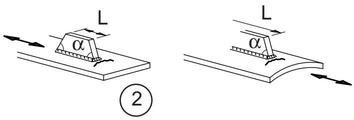
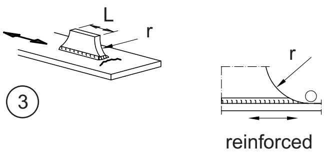
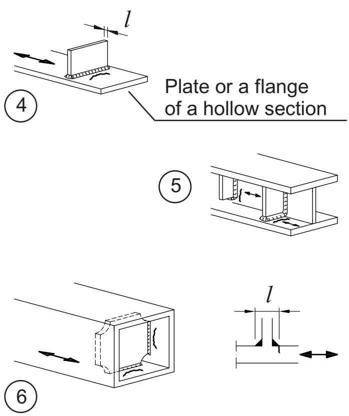
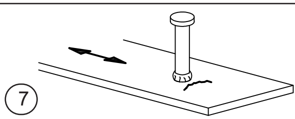
Detail category	Constructional detail		Description	Requirements
80	$L \leq 50$ mm		Longitudinal attachments: 1) The detail category depends on the length L of the attachment.	The thickness of the attachment must be less than its height.
71	$50 < L \leq 80$ mm			
63	$80 < L \leq 100$ mm			
56	$L > 100$ mm			
71	$L > 100$ mm $\alpha < 45°$		2) Longitudinal attachments to plate or tube.	
80	$r > 150$ mm		3) Longitudinal fillet welded gusset with radius transition to plate or tube; end of fillet weld reinforced (full penetration); length of reinforced weld > r.	3) Smooth transition radius r formed by initially machining or gas cutting the gusset plate before welding, then subsequently grinding the weld area parallel to the direction of the arrow so that the transverse weld toe is fully removed.
80	$l \leq 50$ mm		Transverse attachments: 4) Welded to plate. 5) Vertical stiffeners welded to a beam or plate girder. 6) Diaphragm of box girders welded to the flange or the web. May not be possible for small hollow sections. The values are also valid for ring stiffeners.	Details 4) and 5): 4) Ends of welds to be carefully ground to remove any undercut that may be present. 5) $\Delta\sigma$ to be calculated using principle stresses if the stiffener terminates in the web, see left side.
71	$50 < l \leq 80$ mm			
80			7) The effect of welded shear studs on base material.	

Table 4.8 Load carrying welded joints [8,9,10]

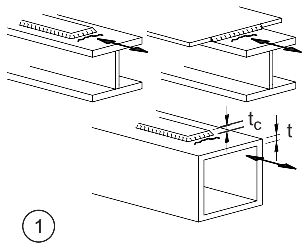
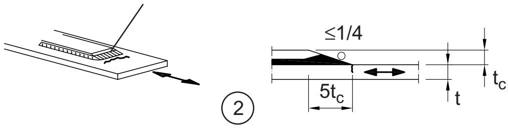
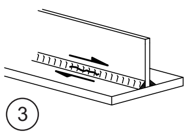
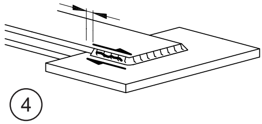
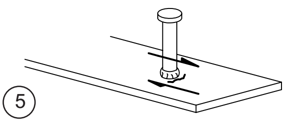
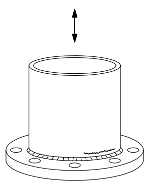
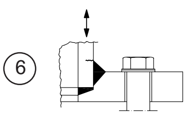
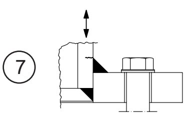
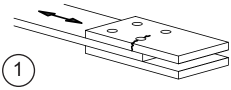
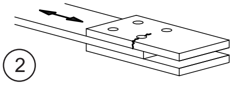
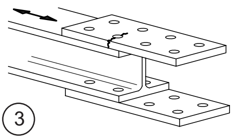
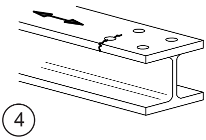
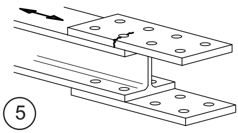
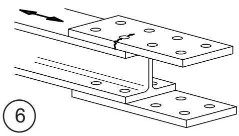
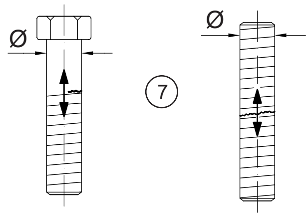
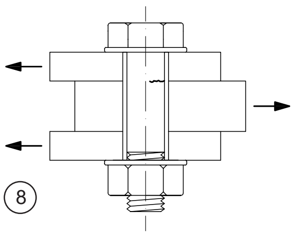
Detail category	Constructional detail			Description	Requirements
	$t_c < t$	$t_c \geq t$		<u>Cover plates in beams and plate girders:</u> 1) End zones of single or multiple welded cover plates, with or without transverse end weld.	1) If the cover plate is wider than the flange, a transverse end weld is needed. This weld shall be carefully ground to remove undercut. The minimum length of the cover plate is 300 mm.
56*	$t \leq 20$	–			
50	$20 < t \leq 30$	$t \leq 20$			
45	$30 < t \leq 50$	$20 < t \leq 30$			
40	$t > 50$	$30 < t \leq 50$			
36	–	$t > 50$			
56				2) Cover plates in beams and plate girders. $5t_c$ is the minimum length of the reinforcement weld.	2) Transverse end weld is ground smooth. In addition, if $t_c > 20$ mm, front of the plate at the end to be ground with a slope < 1:4.
80 m = 5	 			3) Continuous fillet welds transmitting a shear flow, such as web-to-flange welds in plate girders. 4) Fillet welded lap joint.	3) $\Delta\tau$ to be calculated from the weld throat area. 4) $\Delta\tau$ to be calculated from the weld throat area considering the total length of the weld.
see EN 1994-2 (90 m = 8)				<u>Welded stud shear connectors:</u> 5) For composite application.	5) $\Delta\tau$ to be calculated from the nominal cross-section of the stud.
71	 			6) Tube socket joint with 80 % full penetration butt welds.	6) Weld toe to be ground. $\Delta\sigma$ is calculated in tube.
40				7) Tube socket joint with fillet welds.	7) $\Delta\sigma$ is calculated in tube.
* the detail category may be increased by one category, if the modified fatigue strength curve is used, see Figure 4.10					

Table 4.9 Fatigue categories for bolted joints [8,9,10]

Detail category	Constructional detail	Description	Requirements	
112		1) Double covered symmetrical joint with preloaded high strength bolts.	1) $\Delta\sigma$ to be calculated in the gross cross-section.	<u>For bolted connections (details 1...6) in general:</u> End distance: $e_1 \geq 1{,}5$ d Edge distance: $e_2 \geq 1{,}5$ d Spacing: $p_1 \geq 2{,}5$ d Spacing: $p_2 \geq 2{,}5$ d Detailing according to EN 1993-1-8: Figure 3.1.
		1) Double covered symmetrical joint with preloaded injection bolts.	1) ... gross cross-section.	
90		2) Double covered joint with fitted bolts.	2) ... net cross-section.	
		2) Double covered joint with non-preloaded injection bolts.	2) ... net cross-section.	
		3) One sided connection with preloaded high strength bolts.	3) ... gross cross-section.	
		3) One sided connection with preloaded injection bolts.	3) ... gross cross-section.	
		4) Structural element with holes subject to bending and axial forces.	4) ... net cross-section.	
80		5) One sided connection with fitted bolts.	5) ... net cross-section.	
		5) One sided connection with non-preloaded injection bolts.	5) ... net cross-section.	
50		6) One sided or double covered symmetrical connection with non-preloaded bolts in normal clearance holes. No load reversals.	6) ... net cross-section.	

(continues)

Table 4.9 Fatigue categories for bolted joints [8,9,10]
(continued)

Detail category	Constructional detail	Description	Requirements
50	 Size effect for t > 30 mm: $k_s = (30/t)^{0,25}$	7) Bolts and rods with rolled or cut threads in tension. For large diameters (anchor bolts) the size effect has to be taken into account with k_s.	7) $\Delta\sigma$ to be calculated using the tensile stress area of the bolt. Bending and tension resulting from prying effects and bending stresses from other sources must be taken into account. For preloaded bolts, the reduction of the stress range may be taken into account.
100 m = 5		<u>Bolts in single or double shear:</u> Thread is not in the shear plane: - fitted bolts - normal bolts without load reversal (bolts of grade 5.6, 8.8 or 10.9)	8) $\Delta\tau$ is calculated from the non-threaded cross-section of the bolt shank.

4.6.3.3 Fatigue assessment by hot spot stress method

In EN 1993-1-9 the design value of geometric (hot spot) stress range $\gamma_{Ff}\Delta\sigma_{E.2}$ is determined as follows [8,9,10]:

$$\gamma_{Ff}\Delta\sigma_{E.2} = k_f \cdot (\gamma_{Ff}\Delta\sigma^*_{E.2}) \qquad (4.26)$$

where the factor k_f is a stress concentration factor, that can be obtained from handbooks, FEM analysis or from strain gauge measurements. The guidelines presented in EN 1993-1-9 for the hot spot stress method are adressed for fatigue assessment of the 'ordinary' welded plate structures adjacent to the weld, hence the corresponding detail's fatigue categories presented in the standard (EN 1993-1-9: Annex B) include the effect of the stress concentration factor such that it makes it possible to use directly the same S-N curves for fatigue strength as used in the nominal stress method.

The guidelines in EN 1993-1-9 are not as such applicable to hot spot assessment of hollow section lattice joints. This is because in the hot spot assessment of lattice joints the fatigue strength S-N curves have slightly different slope angles (consequently, for example, the calculation formulae presented by the nominal strength method for the equivalent stress range $\Delta\sigma_{E.2}$ are not valid any more).

Lattice joints of structural hollow sections:

In the following the Eurocode-compatible hot spot design provisions are presented for hollow section lattice joints, based mainly on [5,6,7].

In the hot spot assessment, the stresses of the lattice members are first calculated as in the nominal stress method, i.e. the nominal stresses (or the stress ranges) calculated using the simplified pinned joint model are multiplied using the magnification factors k_1 presented in Table 4.3, but the joint eccentricity can be ignored when it does not exceed the limits presented for lattice joints in Tables 4.12 and 4.13.

The nominal stresses (or the stress ranges) of the considered point are then multiplied by the stress concentration factor k_f that is assigned for the hot spot stress. The design value of the hot spot stress range $\gamma_{Ff}\Delta\sigma_{hs.E}$ is determined as follows:

$$\gamma_{Ff}\Delta\sigma_{hs.E} = k_f \cdot k_1 \cdot (\gamma_{Ff}\Delta\sigma^*_{E}) \qquad (4.27)$$

where
- $\Delta\sigma^*_E$ is the nominal stress range calculated using the simplified pinned model of the lattice (see clause 4.6.3.2: Lattice joints of structural hollow sections)
- γ_{Ff} is the partial safety factor for fatigue load (clause 4.6.2)
- k_1 is the magnification factor according to Table 4.3
- k_f is the stress concentration factor for the hot spot stress

Determination of the stress concentration factor k_f is presented later on in more details.

For hollow section lattice joints, the S-N curves and fatigue strengths by hot spot stress ranges are presented in Figure 4.13. There is a separate curve for each hollow section wall thickness (cf. the size effect in clause 4.6.3.2). As with the S-N curves for the nominal stresses, also with the curves in Figure 4.13 the fatigue limit $\Delta\sigma_D$ for constant-amplitude loading is located at $N = 5 \cdot 10^6$ cycles, and the fatigue limit (cut-off limit) $\Delta\sigma_L$ for variable-amplitude loading is reached at $N = 1 \cdot 10^8$. As stated earlier, the slopes of the curves differ, however, from the slopes of the S-N curves for nominal stresses, why the formulae presented for nominal stress S-N curves cannot be directly applied to the curves of Figure 4.13.

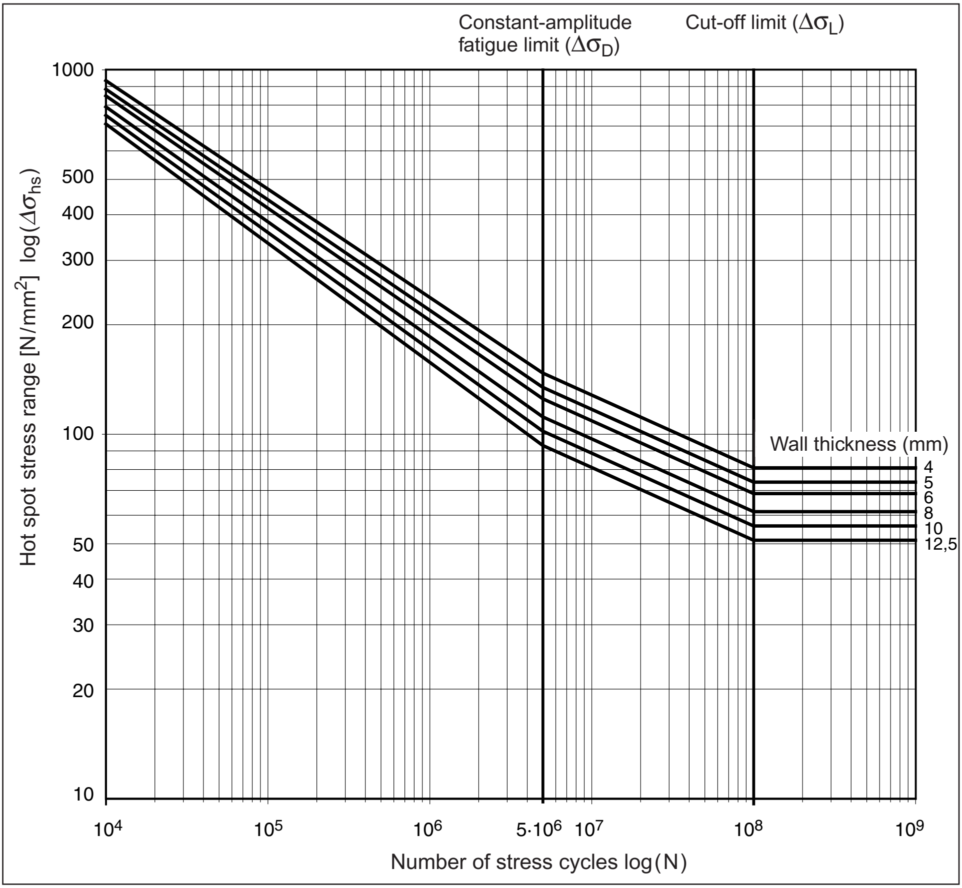


Figure 4.13 Hot spot stress based S-N curves and fatigue strength for hollow section lattice joints. On wall thicknesses 2,5 - 4 mm the curve for 4 mm can be used [7].

Table 4.10 Hot spot stress fatigue limits $\Delta\sigma_D$ and $\Delta\sigma_L$ applied in the S-N curves of hollow section lattice joints [7]

Wall thickness t (mm)	Constant-amplitude loading Fatigue limit $\Delta\sigma_D$ (N/mm^2)	Variable-amplitude loading Cut-off limit $\Delta\sigma_L$ (N/mm^2)
4	147	81
5	134	74
6	125	69
8	111	61
10	102	56
12,5	93	51

In the curves of Figure 4.13 the stress range for the fatigue strength and the corresponding number of load cycles are calculated from the following formulae [5,6,7]:

- **when $1 \cdot 10^3 \le N \le 5 \cdot 10^6$:**

$$log(\Delta\sigma_{hs.R}) = \frac{1}{3} \cdot \left[12,476 - log(N_R) + 0,18 \cdot log(N_R) \cdot log\left(\frac{16}{t}\right)\right] \tag{4.28}$$

$$log(N_R) = \frac{12,476 - 3 \cdot log(\Delta\sigma_{hs.R})}{1 - 0,18 \cdot log\left(\frac{16}{t}\right)} \tag{4.29}$$

or:

$$\Delta\sigma_{hs.R} = 10^{\frac{1}{3} \cdot \left[12,476 - log(N_R) + 0,18 \cdot log(N_R) \cdot log\left(\frac{16}{t}\right)\right]} \tag{4.30}$$

$$N_R = 10^{\frac{12,476 - 3 \cdot log(\Delta\sigma_{hs.R})}{1 - 0,18 \cdot log\left(\frac{16}{t}\right)}} \tag{4.31}$$

- **when $5 \cdot 10^6 \le N \le 1 \cdot 10^8$:**

$$log(\Delta\sigma_{hs.R}) = \frac{1}{5} \cdot \left[16,327 - log(N_R) + 2,01 \cdot log\left(\frac{16}{t}\right)\right] \tag{4.32}$$

$$log(N_R) = 16,327 - 5 \cdot log(\Delta\sigma_{hs.R}) + 2,01 \cdot log\left(\frac{16}{t}\right) \tag{4.33}$$

or:

$$\Delta\sigma_{hs.R} = 10^{\frac{1}{5} \cdot \left[16,327 - log(N_R) + 2,01 \cdot log\left(\frac{16}{t}\right)\right]} \tag{4.34}$$

$$N_R = 10^{16,327 - 5 \cdot log(\Delta\sigma_{hs.R}) + 2,01 \cdot log\left(\frac{16}{t}\right)} \tag{4.35}$$

Because it is not possible to determine the design value of the equivalent stress range $\gamma_{Ff}\Delta\sigma_{hs.E.2}$ for hot spot stresses of lattice joints, the verification of fatigue resistance is performed as follows:

$$\frac{\gamma_{Ff}\Delta\sigma_{hs.E}}{\Delta\sigma_{hs.R}/\gamma_{Mf}} \leq 1,0 \quad or \quad \frac{n_E}{N_R} \leq 1,0 \quad \textit{for constant-amplitude loading} \tag{4.36}$$

$$\sum \frac{n_{Ei}}{N_{Ri}} \leq 1,0 \quad \textit{for variable-amplitude loading} \tag{4.37}$$

where:

- for constant-amplitude loading:

  if the verification is based on the stress ranges:

$\gamma_{Ff}\Delta\sigma_{hs.E}$	is the design value of the hot spot stress range according to expression (4.27) (stress ranges below the fatigue limit $\Delta\sigma_D$ are not taken into account, Table 4.10)
γ_{Ff}	is the partial safety factor for fatigue load (clause 4.6.2)
$\Delta\sigma_{hs.R}$	is the fatigue strength for hot spot stress range at the considered number of cycles n_E according to the S-N curves presented in Figure 4.13
γ_{Mf}	is determined according to clause 4.6.2

  or

  if the verification is based on the number of cycles leading to the failure:

n_E	is the number of cycles corresponding to the considered stress range $\gamma_{Ff}\Delta\sigma_{hs.E}$ (stress ranges below the fatigue limit $\Delta\sigma_D$ are not taken into account, Table 4.10)
N_R	is the working life (expressed in cycles) to the failure, which is determined from the curve $[(\Delta\sigma_{hs.R}/\gamma_{Mf}), N_R]$ (see instructions for calculations below)

- for variable-amplitude loading:

  if the verification is based on the number of cycles leading to the failure:

n_{Ei}	is the number of cycles corresponding to each of the considered stress range $\gamma_{Ff}\Delta\sigma_{hs.Ei}$ (stress ranges below the cut-off limit $\Delta\sigma_L$ are not taken into account, Table 4.10)
N_R	is the working life (expressed in cycles) to the failure, which is determined from the curve $[(\Delta\sigma_{hs.R}/\gamma_{Mf}), N_R]$ (see instructions for calculations below)

The assessment on the basis of the number of cycles is based on the Palmgren-Miner damage sum. In order to carry out the verification, the S-N curve of the fatigue strength shall be first 'dropped' downwards, as presented above, by amount of the partial safety factor γ_{Mf}, and the number of cycles N_{Ri} leading to the failure is calculated for each stress range $\Delta\sigma_{hs.Ei}$ (multiplied by partial safety factor γ_{Ff}) from the hereby 'dropped' curve. The partial safety factors are thereby taken into account both in respect to the load (γ_{Ff}) and in respect to the resistance (γ_{Mf}) in the corresponding way as in the verification based on the stress ranges in expression (4.36). The lowering of the S-N curves can be performed (by sufficient accuracy) by substituting the wall thickness t by an increased design thickness t_d, which is calculated from formula (4.38). When calculating the damage sum, each N_{Ri} is thereby calculated as follows:

- in expressions (4.31) and (4.35) the hollow section's wall thickness t is substituted by the increased design thickness t_d, which is calculated from expression (4.38) and
- stress range $\Delta\sigma_{hs.R}$ is substituted by the design value $\gamma_{Ff}\Delta\sigma_{hs.Ei}$ of each stress range.

The design value of the hollow section's wall thickness is calculated as follows (Note: this procedure is applied only when the verification is based on the number of stress cycles):

$$t_d = t \cdot (\gamma_{Mf})^3 \qquad (4.38)$$

where t is the wall thickness
γ_{Mf} is determined according to clause 4.6.2

The stress concentration factor k_f can be obtained from handbooks, FEM analysis or from strain gauge measurements. Depending on the source, also the symbols K_s or SCF are applied for the stress concentration factor.

The calculation of the concentration factors for stresses in hollow section lattice joints is studied in [5,6]. The concentration factors have been expressed in parametric form, where the parameters are determined by the dimensions of the chord and the braces, and the dimensions of the joint (gap or overlap, joint angle). The formulae for concentration factors for T, X and K joints made of square hollow sections are presented in Tables 4.11 - 4.13 using the symbol SCF from [6]. In [6] parametric formulae for concentration factors also for joints made of circular hollow sections have been presented.

The value of the stress concentration factor varies in different places of the same lattice joint and on different sides of the same joint (weld – chord or weld – brace). In Tables 4.11 - 4.13 the stress concentration factors are presented for T , X and K joints at their critical points. The fatigue assessment shall be performed separately for each critical point of the joint.

For T and X joints the critical points are presented in Table 4.11 using letters A ... E. The member forces of the brace members and chords are calculated from the simultaneously acting loads of the structure. The total value of the hot spot stress ($\Delta\sigma_{hs.0}$ or $\Delta\sigma_{hs.i}$) at each point A ... E to be considered is calculated by superposing the separate hot spot stresses induced therein by the brace member and by the chord according to the following principle [5,6]:

chord (point B, C or D):

$$\Delta\sigma_{hs.0} = \frac{SCF_{0.N} \cdot (k_1 \cdot \Delta N_0)}{A_0} + \frac{SCF_{0.M(ip)} \cdot (k_1 \cdot \Delta M_{ip.0})}{W_{ip.0}} + \frac{SCF_{i.N} \cdot (k_1 \cdot \Delta N_i)}{A_i} + \frac{SCF_{i.M(ip)} \cdot (k_1 \cdot \Delta M_{ip.i})}{W_{ip.i}} + \frac{SCF_{i.M(op)} \cdot (k_1 \cdot \Delta M_{op.i})}{W_{op.i}} \quad (4.39)$$

brace member (point A or E):

$$\Delta\sigma_{hs.i} = \frac{SCF_{i.N} \cdot (k_1 \cdot \Delta N_i)}{A_i} + \frac{SCF_{i.M(ip)} \cdot (k_1 \cdot \Delta M_{ip.i})}{W_{ip.i}} + \frac{SCF_{i.M(op)} \cdot (k_1 \cdot \Delta M_{op.i})}{W_{op.i}} \quad (4.40)$$

where:

k_1	is the magnification factor for the nominal stresses (Table 4.3)
$\Delta\sigma_{hs.0}$	is the hot spot stress range in the chord
$SCF_{0.N}$	is the concentration factor at the considered point (A ... E) for the stress caused by normal force in the chord
$SCF_{0.M(ip)}$	is the concentration factor at the considered point (A ... E) for the stress caused by in-plane bending moment of the chord
ΔN_0	is the normal force range of the chord
$\Delta M_{ip.0}$	is the in-plane bending moment range of the chord
A_0	is the cross-section area of the chord
$W_{ip.0}$	is the in-plane section modulus of the chord
$\Delta\sigma_{hs.i}$	is the hot spot stress range in the brace member
$SCF_{i.N}$	is the concentration factor at the considered point (A ... E) for the stress caused by normal force in the brace member
$SCF_{i.M(ip)}$	is the concentration factor at the considered point (A ... E) for the stress caused by in-plane bending moment of the brace member
$SCF_{i.M(op)}$	is the concentration factor at the considered point (A ... E) for the stress caused by out-of-plane bending moment of the brace member
ΔN_i	is the normal force range of the brace member
$\Delta M_{ip.i}$	is the in-plane bending moment range of the brace member
$\Delta M_{op.i}$	is the out-of-plane bending moment range of the brace member
A_i	is the cross-section area of the brace member
$W_{ip.i}$	is the in-plane section modulus of the brace member
$W_{op.i}$	is the out-of-plane section modulus of the brace member

As it can be seen from expressions (4.39) and (4.40), the simultaneous loading state in the brace members (normal force, bending moment) has an effect on the hot spot stress of the chord. Instead, the loading state of the chord has no effect on the hot spot stress of a brace member. The above expressions present only the basic principle to be applied in the assesment. The actual verification shall be done separately on each critical point A ... E of the brace member and chord.

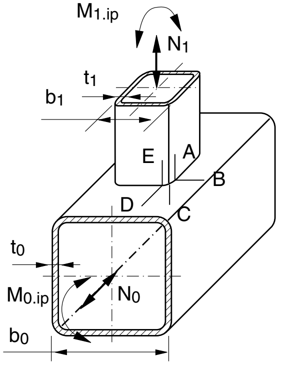
The stresses do not necessarily vary in the chords and brace members in same phase. The stress range of the chord is calculated by the maximums of the range of the forces in the chord. To the stress ranges of the chord, the values of the stress ranges of the brace members are added, the latter being calculated by the forces acting simultaneously with the maximum forces of the chord. Regarding brace members the procedure is similar, but now the maximum values are calculated for the stress ranges of brace members.

For verication, the hot spot stress ranges calculated from expressions (4.39) and (4.40) shall be multiplied by partial safety factor for fatigue load γ_{Ff} (see expression 4.27), but the magnification factor k_1 shall not be applied again any more.

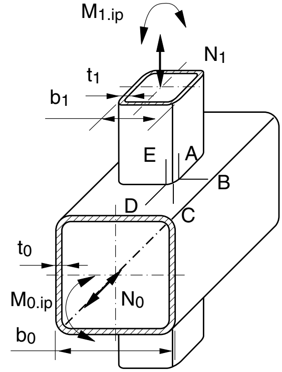
On K joints the calculation is performed in a slightly different way. For K joints there are in Tables 4.12 and 4.13 calculation formulae directly given for the maximum values of the concentration factors of the chord and brace members, why it is not needed to consider the stresses at all points A ... E. At first the loading state in the joint is splitted for the assessment into two 'partial loads' in the way presented in Tables 4.12 and 4.13. After this the both partial loads are considered separately using the stress concentration factors (SCF) according to Tables 4.12 and 4.13. At last, the total value of the hot spot stress is calculated by superposing the effects of the partial loads 1 and 2 at the place to be considered (i.e. at the considered member) using the same principle as presented in expressions (4.39) and (4.40), i.e. the simultaneous loading state of the brace members has an effect on the hot spot stress of the chord, but the loading state of the chord has no effect on the hot spot stress of the brace member. When calculating the hot spot stresses, it should be remembered to apply the magnification factor k_1 of the nominal stresses in the corresponding way as in expressions (4.39) and (4.40).

Table 4.11 Stress concentration factors for T and X joints, when brace member is square hollow section and chord is square or rectangular hollow section [6]

T joint



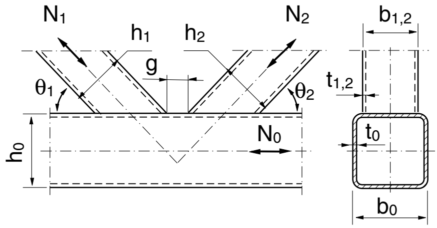
X joint



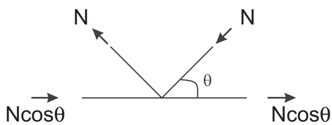
Loading:	**Stress concentration factor SCF :** - the minimum value for each concentration factor is 2
Brace member loading (normal force)	Effect to chord, points B, C, D: $SCF_{i.N.B} = (0,143-0,204\cdot\beta+0,064\cdot\beta^2)\cdot(2\gamma)^{1,377+1,715\cdot\beta-1,103\cdot\beta^2}\cdot\tau^{0,75}$ $SCF_{i.N.C} = (0,077-0,129\cdot\beta+0,061\cdot\beta^2-0,0003\cdot2\gamma)\cdot(2\gamma)^{1,565+1,874\cdot\beta-1,103\cdot\beta^2}\cdot\tau^{0,75}$ $SCF_{i.N.D} = (0,208-0,387\cdot\beta+0,209\cdot\beta^2)\cdot(2\gamma)^{0,925+2,389\cdot\beta-1,881\cdot\beta^2}\cdot\tau^{0,75}$ X-joints, when β = 1,0: - $SCF_{i.N.C}$ is multiplied by 0,65 - $SCF_{i.N.D}$ is multiplied by 0,50
	Effect to brace member, points A, E: - when fillet welds: SCF_A and SCF_E are multiplied by 1,40 $SCF_{i.N.A} = SCF_{i.N.E} = (0,013+0,693\cdot\beta-0,278\cdot\beta^2)\cdot(2\gamma)^{0,790+1,898\cdot\beta-2,109\cdot\beta^2}$
Brace member loading (in-plane bending)	Effect to chord, points B, C, D: $SCF_{i.M(ip).B} = (-0,011+0,085\cdot\beta-0,073\cdot\beta^2)\cdot(2\gamma)^{1,722+1,151\cdot\beta-0,697\cdot\beta^2}\cdot\tau^{0,75}$ $SCF_{i.M(ip).C} = (0,952-3,062\cdot\beta+2,382\cdot\beta^2+0,0228\cdot2\gamma)\cdot(2\gamma)^{-0,690+5,817\cdot\beta-4,685\cdot\beta^2}\cdot\tau^{0,75}$ $SCF_{i.M(ip).D} = (-0,054+0,332\cdot\beta-0,258\cdot\beta^2)\cdot(2\gamma)^{2,084-1,062\cdot\beta+0,527\cdot\beta^2}\cdot\tau^{0,75}$
	Effect to brace member, points A, E: - when fillet welds: SCF_A and SCF_E are multiplied by 1,40 $SCF_{i.M(ip).A} = SCF_{i.M(ip).E} = (0,390-1,054\cdot\beta+1,115\cdot\beta^2)\cdot(2\gamma)^{-0,154+4,555\cdot\beta-3,809\cdot\beta^2}$
Chord loading (normal force and / or in-plane bending)	Effect to chord, points B, C, D: $SCF_{0.N.B} = SCF_{0.M(ip).B} = 0$ (no effect) $SCF_{0.N.C} = SCF_{0.M(ip).C} = 0,725\cdot(2\gamma)^{0,248\cdot\beta}\cdot\tau^{0,19}$ $SCF_{0.N.D} = SCF_{0.M(ip).D} = 1,373\cdot(2\gamma)^{0,205\cdot\beta}\cdot\tau^{0,24}$
	Effect to brace member, points A, E: $SCF_{0.N.A} = SCF_{0.M(ip).E} = 0$ (no effect)

Parameters:		Validity range:		chord:
	$\beta = b_1/b_0$		$0,35 \le \beta \le 1,0$	
	$\gamma = b_0/(2t_0)$		$12,5 \le 2\gamma \le 25$	$0,75 \le h_0/b_0 \le 1,5$
	$\tau = t_1/t_0$		$0,25 \le \tau \le 1,0$	

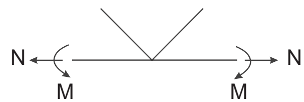
Table 4.12 The stress concentration factors for a gap K joint, when brace members are square hollow sections and chord is square or rectangular hollow section [6]



Dividing the loading to Loads 1 and 2:



Load 1:
balanced member forces
(normal forces in the brace members and chord)



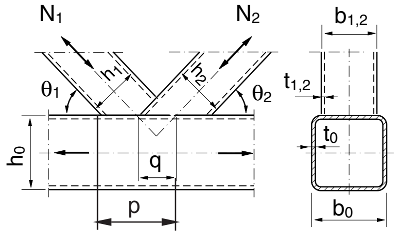
Load 2:
chord forces only
(normal force and / or bending moment)

Stress concentration factor SCF : - the minimum value for each concentration factor is 2

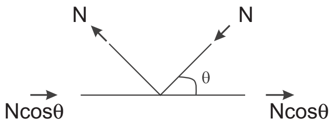
Load 1:	The maximum effect of brace member load N to the chord: $SCF_{N.0} = (0,48 \cdot \beta - 0,5 \cdot \beta^2 - 0,012/\beta + 0,012/g') \cdot (2\gamma)^{1,72} \cdot \tau^{0,78} \cdot (g')^{0,2} \cdot (\sin\theta)^{2,09}$
	The maximum effect of brace member load N to the brace member: $SCF_{N.i} = (-0,008 + 0,45 \cdot \beta - 0,34 \cdot \beta^2) \cdot (2\gamma)^{1,36} \cdot \tau^{-0,66} \cdot (\sin\theta)^{1,29}$
Load 2:	The maximum effect of chord load N, M to the chord: $SCF_{N.0} = SCF_{M.0} = (2,45 + 1,23 \cdot \beta) \cdot (g')^{-0,27}$
	The maximum effect of chord load N, M to the brace member $SCF_{N.i} = SCF_{M.i} = 0$ *(no effect)*

Parameters:		Validity range:	
	$\beta = b_1 / b_0$		$0,35 \le \beta \le 1,0$
	$\gamma = b_0 / (2t_0)$		$10 \le 2\gamma \le 35$
	$\tau = t_1 / t_0$		$0,25 \le \tau \le 1,0$
	$g' = g / t_0$		$g' \ge 2\tau$
		- eccentricity:	$-0,55 \le e / h_0 \le 0,25$
		- chord:	$0,75 \le h_0/b_0 \le 1,5$
		- brace members:	$30° \le \theta \le 60°$ brace members square, identical with each other and in same angle of degree

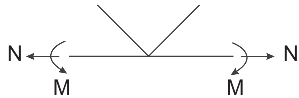
Table 4.13 The stress concentration factors for an overlap K joint, when brace members are square hollow sections and chord is square or rectangular hollow section [6]



Dividing the loading to Loads 1 and 2:



Load 1:
balanced member forces
(normal forces in the brace members and chord)



Load 2:
chord forces only
(normal force and / or bending moment)

	Stress concentration factor SCF : - the minimum value for each concentration factor is 2
Load 1:	The maximum effect of brace member load N to the chord: $SCF_{N.0} = \left(0,5 + 2,38 \cdot \beta - 2,87 \cdot \beta^2 - 2,18 \cdot \beta \cdot O_v + 0,39 \cdot O_v - 1,43 \cdot \sin\theta\right) \cdot (2\gamma)^{0,29} \cdot \tau^{0,7} \cdot O_v^{0,73 - 5,53 \cdot (\sin\theta)^2} \cdot (\sin\theta)^{-0,4 - 0,08 \cdot O_v}$
	The maximum effect of brace member load N to the brace member: $SCF_{N.i} = \left(0,15 + 1,1 \cdot \beta - 0,48 \cdot \beta^2 - 0,14/O_v\right) \cdot (2\gamma)^{0,55} \cdot \tau^{-0,3} \cdot O_v^{-2,57 + 1,62 \cdot \beta^2} \cdot (\sin\theta)^{0,31}$
Load 2:	The maximum effect of chord load N, M to the chord: $SCF_{N.0} = SCF_{M.0} = 1,2 + 1,46 \cdot \beta - 0,028 \cdot \beta^2$
	The maximum effect of chord load N, M to the brace member: $SCF_{N.i} = SCF_{M.i} = 0$ *(no effect)*

Parameters:	$\beta = b_1 / b_0$	Validity range:	$0,35 \le \beta \le 1,0$
	$\gamma = b_0 / (2t_0)$		$10 \le 2\gamma \le 35$
	$\tau = t_1 / t_0$		$0,25 \le \tau \le 1,0$
	O_v = (q / p) x 100 % (see the figure)		$50\ \% \le O_v \le 100\ \%$
		- eccentricity:	$-0,55 \le e / h_0 \le 0,25$
		- chord:	$0,75 \le h_0 / b_0 \le 1,5$
		- brace members:	$30° \le \theta \le 60°$
			brace members square, identical with each other and in same angle of degree

4.7 Design of fatigue-loaded hollow section structures

A fatigue-loaded structure shall always be designed also for static loads at ultimate limit state, as usual.

The selection of the steel grade is advisable to carry out, however, not until the member has been first designed on the basis of fatigue, after which the greatest static stress can be calculated.

High steel strength is not necessarily profitable in fatigue design, because the fatigue strength of a welded structure is not dependent on the steel strength, but on the stress fluctuations of the structure. Increasing the cross-section reduces the stress range, but increases the weight of the structure, while increasing the steel strength reduces the cross-section and weight of the structure, but usually increases the stress range.

In a fatigue-loaded welded structure, increase of the yield strength is, however, profitable in case the portion of the permanent loads of the structure is high, or if the fatigue loading contains single high maximum loads, for which the static resistance of a lower grade steel is not sufficient [3].

By careful design of the joints and by geometrical design of the structure it is possible to considerably influence the fatigue resistance of the entire structure. The joints are favourable to place in such locations where the stress range is as small as possible.

The most effective way to improve the fatigue strength of welded structures is to decrease the stress range either by placement of the joints or by increasing the cross-section of the member: if the stress range is cut to half, the working life of the structure becomes 8 times longer (when $m = 3$).

4.7.1 Design of welded joints

The welded joints are favourable to be placed at the least stressed locations in the structure, if possible (Table 4.14).

In respect to fatigue, a welded splice joint between the hollow sections is beneficial to execute without splice plates according to Figure 4.14b, in which case the detail's fatigue category is better (Table 4.5). Regarding the resistance of the weld, it is essential that the weld is free from defects especially on the root side, because the inspection of the root side is difficult. Use of steel backing is beneficial if root defects are possible to develop in welding (e.g. when having thick materials). When using steel backing it should be noted, however, that it arises a stress concentration, that may reduce the fatigue resistance.

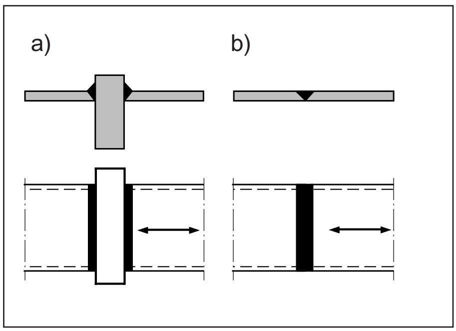


Figure 4.14 Fatigue resistance of a splice joint is lower when executed with a splice plate (a) than without it (b)

Enhancement of the weld quality level reduces the possibility of weld defects, whereby fatigue resistance improves. The required quality level for the weld is determined according to the execution class chosen in EN 1090-2 for the structure or the member, as presented in Table 3.6. Standard EN 1090-2 defines the execution class for a fatigue-loaded structure to be, depending on the case, EXC2 - EXC4. Thereby the weld quality level shall be B or C according to Table 3.6 [12]. The choice of weld quality level has, however, no effect on the calculation of the static resistance or the fatigue resistance of the welds.

High excess weld metal increases the stress peak at the weld toe, why a concave shape of the fillet weld is better in respect to fatigue. Smooth weld toe angle and big radius of curvature at the weld toe are factors that improve the fatigue strength of the weld.

Post-weld treatment

The fatigue resistance of the weld can be improved also by post-weld treatment. The methods can be divided into two main categories, i.e. improvement of the weld geometry and the residual stress methods. The weld geometry can be improved, for example, by grinding the weld toe or by re-melting the toe either with a TIG torch or a plasma torch. With these methods the initial cracks at the weld toe are removed and, in addition, the smoother shaping of the weld reduces the stress concentrations at the weld toe. The grinding radius to be recommended is 10 mm for materials having thickness less than 20 mm. Grinding is not recommended, if the thickness of the material is less than 10 mm [13].

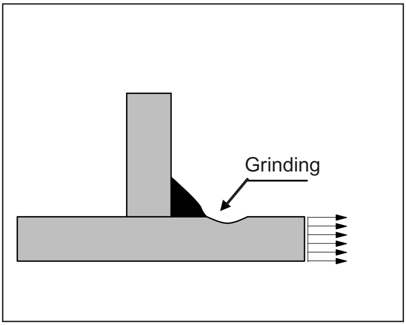


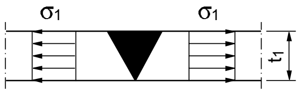
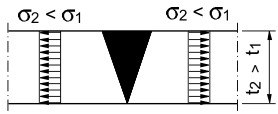
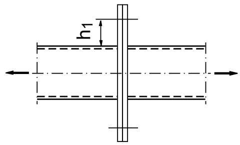
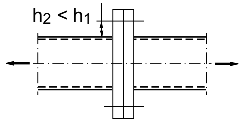
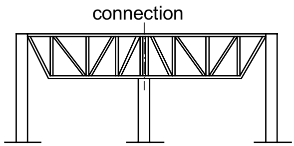
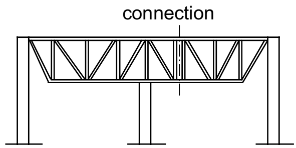
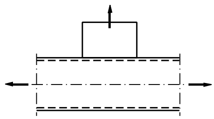
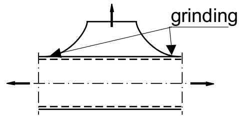
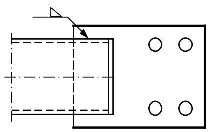
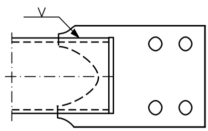
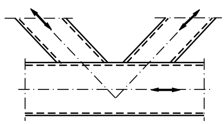
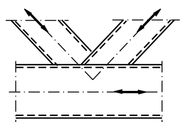
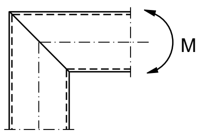
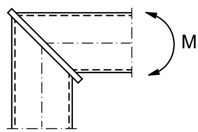
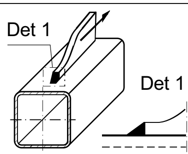
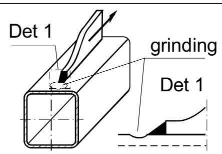
Figure 4.15 Improving the fatigue strength of a welded joint by grinding the weld toe

Residual stress methods are used to affect the residual stresses at the weld. By using mechanical cold-working, a compressive residual stress is developed in the surface layer of the material, which then prevents further opening of the crack and reduces the effective stress fluctuation. For example, shot blasting and peening are mechanical forming methods. It is also possible to use heat treatments to control the residual stresses. In stress-relief annealing, the yield limit of the material is temporarily decreased by rising the temperature. The internal stresses of the structure are relieved through plastic deformations. The heating and cooling rate should be slow to prevent new stresses to develop [13].

The post-weld treatment can be profitable when the area to be treated is small, or if the post-weld treatment can be automatized in the workshop. The post-weld treatment is also possible as a repairing measure of old structures. The effect of post-weld treatment has not been taken into account in the design provisions of EN 1993-1-9. An empirical observation is that grinding increases the fatigue strength by 30 - 100 % (when $N = 2 \cdot 10^6$ load cycles) and re-melting by 10 - 170 % (when $N = 2 \cdot 10^6$ load cycles). By shot blasting, the fatigue strength improvements of 30 - 170 % have been achieved (when $N = 2 \cdot 10^6$ load cycles) [13].

Various methods for improving fatigue strength are discussed in more details in [2,3,13].

Table 4.14 Methods for improving the fatigue resistance of a structure

Method	Original detail	Improved solution
Stress reduction		
Reduction of bending moment in flange plates		
Connection placement in a location subjected to lower load		
Smoother attachment plate geometry		
Smoother splice plate geometry		
Overlapping of a truss connection		
L joint reinforcement with an intermediate plate		
Welding method selection	Manual welding	Machine welding
Weld end rounding by grinding		

4.7.2 Design of bolted joints

In bolted joints the fabrication method of the holes has an effect on the fatigue resistance of a bolted joint or bolted connection. The fatigue resistance of a hole made by drilling is better than fatigue resistance of a hole made by punching. In EN 1090-2 the effect of the hole's fabrication method is taken into account as follows: punching is generally permitted, but in fatigue-loaded structures the hole shall be, after punching, reamed to its final size in such a way that the quality of the final hole corresponds to the quality of the drilled hole [12].

In bolted joints subject to tension the use of prestressed bolts improves the fatigue resistance of the joint significantly. The fatigue-inducing stress decreases, if the prestressing force is greater than the tension load of the joint. In respect to fatigue, long and elastic bolts are beneficial, because the elasticity of the plates in the joint does not substantially reduce the prestressing force of the long bolts. In a flange-plate joint subject to tension, the bolts should be placed as near to the weld as possible to reduce the additional stresses due to eccentricities.

The prestressing of the bolts is beneficial also in bolted joints subject to shear force. The prestressing reduces the bearing stress in the material at the edge of the hole, since part of the force is transmitted through friction between the splice plates. However, it is important to ensure that the friction between the splice plates is sufficient to prevent the bolt from slipping against the edge of the hole, which would result in the loss of the prestressing benefits.

4.7.3 Design of trusses

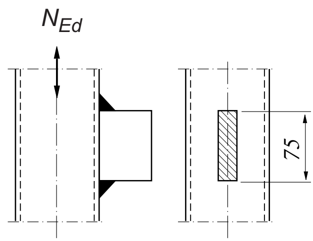
The fatigue resistance of a fatigue-loaded hollow section truss (i.e. lattice) can easily be improved by structural means by favouring the following practices in design [5,6]:

- If the truss needs to be provided with a splice joint, it is favourable to place at the zero-point of the moment (Table 4.14).
- An overlap K joint is better than a gap K joint (Table 4.6).
- On K joints, decreasing the degree of angle of the brace members improves the fatigue resistance. The recommended value is θ = 40º.
- A thick chord and thin-walled brace members are favourable. (i.e. low values of $\tau = t_i / t_0$ and $2\gamma = b_0 / t_0$ or $2\gamma = d_0 / t_0$, see Tables 4.6 and 4.11 - 4.13)
- Avoid β values within the range β = 0,5...0,7. The most beneficial values are near to β = 1,0 whereby the lowest SCF values are achieved. However, in respect to fabrication, the recommended value is β = 0,8.
- A butt weld is better than a fillet weld (see Table 4.11: factor 1,4 for fillet weld)

The welds in lattice joints should be made such that the start / stop positions of the weld are not located at the corners of the brace members or at the positions of stress concentrations in the joint, since the start / stop positions are critical regarding fatigue. The correct welding sequence is presented in Figures 8.10 - 8.11 of Chapter 8. To ensure the specified throat thickness when the joint angle is less than 60º, the end of the brace member must be bevelled (clause 8.5.4). In lattice joints, the throat thickness should be big enough to prevent the root side of the weld from becoming critical in respect to fatigue (see clause 4.6.3.1). The reinforcing of lattice joints should be considered case by case. Reinforcing the components improves the static resistance of the joint, but also generates discontinuities where stress concentrations arise. However, when properly applied, it is also possible to improve the fatigue resistance with reinforcement plates. More detailed instructions are presented in [6].

Example 4.1

Calculate the resistance of a hollow section 200×200×8 subjected to a fatigue load ΔN_{Ed}. A non-loaded plate is welded to the side of the hollow section.



The steel grade of the hollow section is SSAB Domex Tube Double Grade, which fulfills the EN 10219 requirements for both steel grades S420MH and S355J2H. Thereby the design calculations may be performed at designer's own choice either according to grade S420 or grade S355. Grade S420 is chosen in this Example as design basis.

The load of the hollow section varies as follows:

	ΔN_{Ed} (kN)	Number of load cycles (n_i)
1	220	$1{,}5 \cdot 10^6$
2	350	$1 \cdot 10^6$
3	50	$1 \cdot 10^7$
The values of ΔN_{Ed} presented herein do not include partial safety factor for load γ_{Ff}.		

Assume the Safe life method is chosen and the possible failure would lead to high consequences. Thereby the partial safety factor for resistance shall be:
$\gamma_{Mf} = 1{,}35$ *(Table 4.1)*

In fatigue design the stresses are determined according to theory of elasticity. Calculate the nominal stress range in the hollow section caused by the first load fluctuation:

$$\Delta\sigma_1 = \frac{\Delta N_{1.Ed}}{A} = \frac{220000}{5924} = 37{,}1\ N/mm^2$$

For all load fluctuations correspondingly:

	ΔN_{Ed} (kN)	Number of load cycles (n_i)	$\Delta\sigma$ (N/mm^2)
1	220	$1{,}5 \cdot 10^6$	37,1
2	350	$1 \cdot 10^6$	59,1
3	50	$1 \cdot 10^7$	8,4

Check the design condition given for fatigue assessment in respect to the maximum stress range (condition (4.10)):

$\gamma_{Ff} = 1,0$ *partial safety factor for fatigue load*

$$\gamma_{Ff} \cdot \Delta\sigma_{max} = 1,0 \cdot 59,1 = 59,1\ N/mm^2 \le 1,5 f_y = 1,5 \cdot 420 = 630\ N/mm^2 \quad OK$$

Perform fatigue assessment by the nominal stress method. The detail's fatigue category for a non-loaded welded plate attachment is 71, when the length of the plate is 75 mm (Table 4.7).

For normal stress range the fatigue limit in detail category 71 is (Figure 4.7):

$$\Delta\sigma_D = 0,737 \cdot \Delta\sigma_C = 0,737 \cdot 71 = 52\ N/mm^2 \quad \textit{for constant-amplitude loading}$$

$$\Delta\sigma_L = 0,405 \cdot \Delta\sigma_C = 0,405 \cdot 71 = 29\ N/mm^2 \quad \textit{for variable-amplitude loading}$$

The fatigue assessment can be performed by using equivalent stress range $\Delta\sigma_{E.2}$ or by using Palmgren-Miner damage sum. In this Example both methods shall be applied.

a) fatigue assessment by using Palmgren-Miner damage sum:

Stress range $\Delta\sigma_1$ falls on the less steep part of the S-N curve (m=5). The number of cycles leading to failure can be calculated from formula (4.17) by setting $\Delta\sigma_R = \Delta\sigma_1$ whereby the corresponding $N_{R.1}$ can be derived. When also taking into account the partial safety factors, the formula can be written in the following form (<u>if</u> the partial safety factor for load is already included in the original load values or stress ranges, it shall not be applied again any more):

$$N_{R.1} = 5 \cdot 10^6 \left(\frac{\sigma_D/\gamma_{Mf}}{\gamma_{Ff} \cdot \Delta\sigma_1}\right)^5 \quad , \quad for \quad \frac{\Delta\sigma_D}{\gamma_{Mf}} \ge \gamma_{Ff} \cdot \Delta\sigma_1 \ge \frac{\Delta\sigma_L}{\gamma_{Mf}}$$

$$N_{R.1} = 5 \cdot 10^6 \left(\frac{\sigma_D/\gamma_{Mf}}{\gamma_{Ff} \cdot \Delta\sigma_1}\right)^5 = 5 \cdot 10^6 \left(\frac{52/1,35}{1,0 \cdot 37,1}\right)^5 = 6,03 \cdot 10^6$$

Stress range $\Delta\sigma_2$ falls on the steep part of the S-N curve (m=3), whereby now formula (4.16) shall be applied respectively:

$$N_{R.2} = 2 \cdot 10^6 \left(\frac{\sigma_C/\gamma_{Mf}}{\gamma_{Ff} \cdot \Delta\sigma_2}\right)^3 = 2 \cdot 10^6 \left(\frac{71/1,35}{1,0 \cdot 59,1}\right)^3 = 1,41 \cdot 10^6 \quad \left(\gamma_{Ff} \cdot \Delta\sigma_2 \ge \frac{\Delta\sigma_D}{\gamma_{Mf}}\right)$$

Stress range $\Delta\sigma_3$ falls on the horizontal part of the S-N curve, whereby respectively:

$$N_{R.3} = \infty \qquad \left(\gamma_{Ff} \cdot \Delta\sigma_3 < \frac{\Delta\sigma_L}{\gamma_{Mf}}\right)$$

The resistance to the entire load spectrum can be verified by using damage summation according to formula (4.7):

$$\sum \frac{n_i}{N_{Ri}} = \frac{1,5 \cdot 10^6}{6,03 \cdot 10^6} + \frac{1 \cdot 10^6}{1,41 \cdot 10^6} + \frac{1}{\infty} = 0,958 \le 1,0 \quad OK$$

b) fatigue assessment by using equivalent stress range $\Delta\sigma_{E.2}$:

Calculate equivalent stress range corresponding to load cycles $N = 2 \cdot 10^6$. Since now the load type is variable-amplitude loading, formula (4.9) shall be applied. When calculating constant C related to the applied detail category, formula (4.2) needs to be supplemented with the partial safety factor for resistance γ_{Mf}. Stress ranges below the cut-off limit $\Delta\sigma_L$ are not taken into account:

$$C = 2 \cdot 10^6 \cdot (\Delta\sigma_C / \gamma_{Mf})^3 = 2 \cdot 10^6 \cdot (71/1,35)^3 = 2,91 \cdot 10^{11}$$

$$\sum (n_i \cdot \Delta\sigma_i^{\ 3}) = n_2 \cdot \Delta\sigma_2^{\ 3} = (1 \cdot 10^6) \cdot 59,1^3 = 2,06 \cdot 10^{11} \quad \text{stress ranges on steep part}$$

$$\sum (n_j \cdot \Delta\sigma_j^{\ 5}) = n_1 \cdot \Delta\sigma_1^{\ 5} = (1,5 \cdot 10^6) \cdot 37,1^5 = 1,05 \cdot 10^{14} \quad \text{stress ranges on less steep part}$$

$$\Delta\sigma_{E.2} = \sqrt[3]{\frac{\sum (n_i \cdot \Delta\sigma_i^{\ 3}) + \left(\dfrac{C}{5 \cdot 10^6}\right)^{-2/3} \cdot \sum (n_j \cdot \Delta\sigma_j^{\ 5})}{2 \cdot 10^6}}$$

$$= \sqrt[3]{\frac{2,06 \cdot 10^{11} + \left(\dfrac{2,91 \cdot 10^{11}}{5 \cdot 10^6}\right)^{-2/3} \cdot (1,05 \cdot 10^{14})}{2 \cdot 10^6}} = 51,7 \ N/mm^2$$

The verification shall be performed by design condition (4.19). The partial safety factor for load that is written in the formula shall not be applied here again, if it is already included in the original load values or stress ranges:

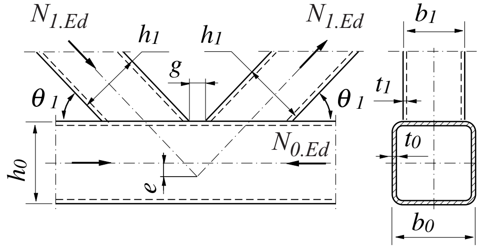
$$\frac{\gamma_{Ff} \Delta\sigma_{E.2}}{\Delta\sigma_C / \gamma_{Mf}} = \frac{1,0 \cdot 51,7}{71/1,35} = 0,983 \le 1,0 \quad OK$$

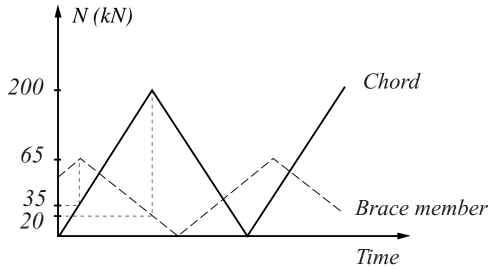
Hence it can be concluded, that the results obtained with both methods (i.e. assessment by equivalent stress range, as well as assessment by Palmgren-Miner damage sum) are, in practice, consistent with each other. By using equivalent stress range the calculations become however somewhat shorter.

Comparison S420 vs S355:
In respect to fatigue resistance only, the steel grade has no impact when designing according to Eurocode. However, even in fatigue design, too low steel grade can result in insufficient 'fatigue resistance':

In fatigue design Eurocode requires that the nominal, modified nominal or geometric stress ranges due to frequent loads $\psi_1 Q_k$ are for normal stresses at highest $\Delta\sigma \leq 1{,}5 f_y$ and for shear stresses at highest $\Delta\tau \leq 1{,}5 f_y / \sqrt{3}$.

Example 4.2





Check fatigue resistance of a gap K joint by using the nominal stress method. The chord is a hollow section 200×200×8 and brace members are 140×140×8. The steel grade of the hollow sections is SSAB Domex Tube Double Grade, which fulfills the EN 10219 requirements for both steel grades S420MH and S355J2H. Thereby the design calculations may be performed at designer's own choice either according to grade S420 or grade S355. Grade S420 is chosen in this Example as design basis.

The normal force range for the brace members is $\Delta N_{1.Ed}$ = 65 kN and for the chord $\Delta N_{0.Ed}$ = 200 kN. The normal forces in the chord and brace members have a phase shift according to adjacent figure, i.e. the chord normal force is 35 kN when the normal force in brace members is 65 kN. Similarly, the normal force in the brace members is 20 kN when the chord normal force is 200 kN. The number of stress cycles is $5 \cdot 10^4$.

The dimensions of the joint are as follows:

θ_1 = 37°
g = 35 mm
e = 0,8 mm (≈ 0 mm)

In this Example the fatigue assesment of the joint shall be performed by the nominal stress method.

The validity conditions are met (Table 4.6), but the checking of them is not presented herein.

The effects caused by the joint eccentricity can be neglected, since the eccentricity does not exceed the limits set for hollow section joints.

The secondary bending moments due to stiffness of the joints in lattice structures, however, must be taken into account if the member forces are calculated using the pin-ended structur-

al model for the brace members. This takes place by multiplying the nominal stress ranges by correction factor k_1 *given in Table 4.3:*

$$\Delta\sigma_1 = \frac{k_1 \cdot \Delta N_{1.Ed}}{A_1} = \frac{1,5 \cdot 65000}{4004} = 24,4\ N/mm^2 \qquad \text{brace member}$$

$$\Delta\sigma_0 = \frac{k_1 \cdot \Delta N_{0.Ed}}{A_0} = \frac{1,5 \cdot 200000}{5924} = 50,6\ N/mm^2 \qquad \text{chord}$$

Check whether the general conditions related to fatigue assessment are satisfied in respect to the highest stress fluctuation (condition (4.10)):

$\gamma_{Ff} = 1,0$ *partial safety factor for fatigue load*

$$\gamma_{Ff} \cdot \Delta\sigma_{max} = 1,0 \cdot 50,6 = 50,6\ N/mm^2 \le 1,5 f_y = 1,5 \cdot 420 = 630\ N/mm^2 \quad \text{OK}$$

The detail category of the joint depends on the ratio of the wall thickness of the chord versus the brace members (Table 4.6):

$t_0 / t_1 = 1,0 \Rightarrow$ *detail category 36*

Assume the Safe life method is chosen. Since the possible failure would lead to high consequences the partial safety factor for resistance shall be:
$\gamma_{Mf} = 1,35$ *(Table 4.1)*

The value of the fatigue strength in detail category 36 at load cycles $N = 5 \cdot 10^4$ *can be obtained from Figure 4.12. The exact value can be derived from formula (4.14). However, when doing so, some modifications are needed to fit the formula for the single-slope S-N curve of lattice joints having the slope 1:5 (m=5):*

$$\Delta\sigma_R = \sqrt[5]{\frac{\Delta\sigma_C^5 \cdot 2 \cdot 10^6}{N}} = \sqrt[5]{\frac{36^5 \cdot 2 \cdot 10^6}{5 \cdot 10^4}} = 75,3\ N/mm^2$$

Next, compare the fatigue strength to the stress ranges of the brace member and the chord:

$$\frac{\gamma_{Ff}\Delta\sigma_1}{\Delta\sigma_R / \gamma_{Mf}} = \frac{1,0 \cdot 24,4}{75,3 / 1,35} = 0,437 \le 1,0 \quad \text{OK} \quad \text{brace member}$$

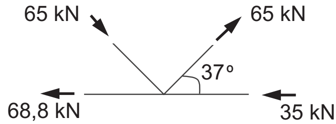
$$\frac{\gamma_{Ff}\Delta\sigma_0}{\Delta\sigma_R / \gamma_{Mf}} = \frac{1,0 \cdot 50,6}{75,3 / 1,35} = 0,907 \le 1,0 \quad \text{OK} \quad \text{chord}$$

Comparison S420 vs S355:
Similar to Example 4.1.

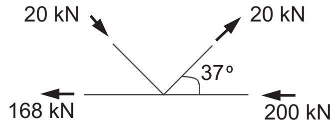
Example 4.3

Check the fatigue resistance of the same joint as in Example 4.2 by using the hot spot stress method. In Example 4.2 the brace member was checked by considering the load spectrum assigned for it alone, and correspondingly the chord was checked by considering the load spectrum assigned for it alone. In the hot spot stress method, however, also the interaction between the members has to be taken into account (i.e. the effect of stress fluctuation of the brace member to the chord). Hence, the assessment of the brace member and the chord shall be performed on the basis of the forces acting simultaneously in both. According to the data given in Example 4.2, the simultaneous forces are:

- for the maximum in brace member:
$N_{i.Ed}$ = 65 kN and simultaneously in chord $N_{0.Ed}$ = 35 kN



- for the maximum in chord:
$N_{0.Ed}$ = 200 kN and simultaneously in brace member $N_{i.Ed}$ = 20 kN



Assessment of the brace member:
First, calculate the values for the parameters applied in Table 4.12 (the validity conditions are met, but the checking of them is not presented herein):

$$\beta = \frac{b_i}{b_0} = \frac{140}{200} = 0,7$$

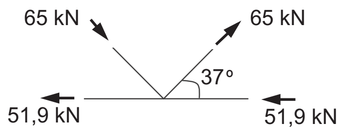
$$\gamma = \frac{b_0}{2t_0} = \frac{200}{2 \cdot 8} = 12,5$$

$$\tau = \frac{t_i}{t_0} = \frac{8}{8} = 1,0$$

$$g' = \frac{g}{t_0} = \frac{35}{8} = 4,38$$

Next, split the loads that are acting in the joint into two partial loads as presented in Table 4.12:

Load 1
(balanced member forces, Table 4.12):



The brace members are subjected to their original member forces, but the forces acting in the chord are determined by the equilibrium of the joint's forces:

$$N_i = 65\ kN \qquad \textit{brace members}$$

$$N_0 = N_i \cos\theta_i = 65 \cdot \cos 37 = 51,9\ kN \qquad \textit{chord}$$

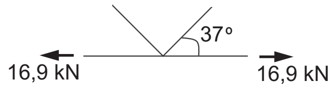
The effect of the brace member load N_i to the brace member's own hot spot stress is obtained from Table 4.12:

$$SCF_{N.i} = (-0,008 + 0,45 \cdot \beta - 0,34 \cdot \beta^2) \cdot (2\gamma)^{1,36} \cdot \tau^{-0,66} \cdot (\sin\theta_i)^{1,29}$$
$$= (-0,008 + 0,45 \cdot 0,7 - 0,34 \cdot 0,7^2) \cdot (2 \cdot 12,5)^{1,36} \cdot 1,0^{-0,66} \cdot (\sin 37)^{1,29}$$
$$= 5,81$$

Hence, Load 1 (= normal force in the brace member) causes to the brace member a hot spot stress range:

$$\Delta\sigma_{hs.i.1} = \frac{SCF_{N.i} \cdot (k_1 \cdot \Delta N_i)}{A_i} = \frac{5,81 \cdot (1,5 \cdot 65000)}{4004} = 142\ N/mm^2$$

Load 2
(chord forces only, Table 4.12):



The theoretical chord forces are determined according to the real loading and the partial Load 1 as follows:

$$N_{0.right} = 35 - 51,9 = -16,9\ kN \quad \text{right chord}$$
$$N_{0.left} = 68,8 - 51,9 = 16,9\ kN \quad \text{left chord}$$

The above derived results can be checked by superposing Load 1 and Load 2, resulting then in the original member forces.

The effect of the chord force N_0 to the brace member's hot spot stress is obtained from Table 4.12:

$$SCF_{N.i} = 0 \quad (\text{no effect}) \quad \Rightarrow \Delta\sigma_{hs.i.2} = 0$$

The total value of the hot spot stress range in the brace member is calculated by superposing the effects of Load 1 and Load 2:

$$\Delta\sigma_{hs.i} = \Delta\sigma_{hs.i.1} + \Delta\sigma_{hs.i.2} = 142 + 0 = 142\ N/mm^2$$

Check whether the general conditions related to fatigue assessment are satisfied in respect to the highest stress fluctuation (condition (4.10)):

$$\gamma_{Ff} = 1,0 \quad \text{partial safety factor for fatigue load}$$

$$\gamma_{Ff} \cdot \Delta\sigma_{max} = 1,0 \cdot 142 = 142\ N/mm^2 \le 1,5 f_y = 1,5 \cdot 420 = 630\ N/mm^2 \quad OK$$

The value of the fatigue strength at load cycles $N = 5 \cdot 10^4$ for hot spot stresses in hollow section lattice joints can be obtained from Figure 4.13. The exact value can be calculated from formula (4.30):

$$\Delta\sigma_{hs.R} = 10^{\frac{1}{3} \cdot \left[12,476 - log(N_R) + 0,18 \cdot log(N_R) \cdot log\left(\frac{16}{t}\right)\right]}$$

$$= 10^{\frac{1}{3} \cdot \left[12,476 - log(5 \cdot 10^4) + 0,18 \cdot log(5 \cdot 10^4) \cdot log\left(\frac{16}{8}\right)\right]}$$

$$= 476\ N/mm^2$$

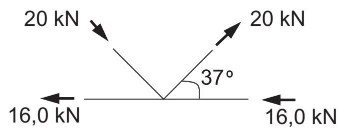
For constant-amplitude loading, the fatigue resistance shall be verified by design condition (4.36):

$$\frac{\gamma_{Ff}\Delta\sigma_{hs.i}}{\Delta\sigma_{hs.R}/\gamma_{Mf}} = \frac{1,0 \cdot 142}{476/1,35} = 0,403 \le 1,0 \quad OK$$

Assessment of the chord:

Split the loads that are acting in the joint into two partial loadings as presented in Table 4.12:

Load 1
(balanced member forces, Table 4.12):



The brace members are subjected to their original member forces, but the forces acting in the chord are determined by the equilibrium of the joint's forces:

$N_i = 20\ kN$ *brace members*

$N_0 = N_i cos\theta_i = 20 \cdot cos37 = 16,0\ kN$ *chord*

The effect of the brace member load N_i to the chord's hot spot stress is obtained from Table 4.12:

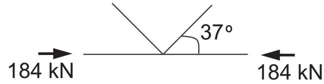
$$SCF_{N.0} = (0,48 \cdot \beta - 0,5 \cdot \beta^2 - 0,012/\beta + 0,012/g') \cdot (2\gamma)^{1,72} \cdot \tau^{0,78} \cdot (g')^{0,2} \cdot (sin\theta)^{2,09}$$

$$SCF_{N.0} = (0,48 \cdot 0,7 - 0,5 \cdot 0,7^2 - 0,012/0,7 + 0,012/4,38) \cdot (2 \cdot 12,5)^{1,72} \cdot$$
$$1,0^{0,78} \cdot (4,38)^{0,2} \cdot (sin37)^{2,09}$$
$$= 9,04$$

Hence, Load 1 (= normal force in the brace member) causes to the chord a hot spot stress range:

$$\Delta\sigma_{hs.0.1} = \frac{SCF_{N.0} \cdot (k_1 \cdot \Delta N_i)}{A_i} = \frac{9,04 \cdot (1,5 \cdot 65000)}{4004} = 220\ N/mm^2$$

Load 2
(chord forces only, Table 4.12):



The theoretical chord forces are determined according to the real loading and the partial Load 1 as follows:

$$N_{0.right} = 200 - 16,0 = 184\ kN \quad \text{right chord}$$

$$N_{0.left} = 168 - (-16) = 184\ kN \quad \text{left chord}$$

The effect of the chord force N_0 to the chord's own hot spot stress is obtained from Table 4.12:

$$SCF_{N.0} = (2,45 + 1,23 \cdot \beta) \cdot (g')^{-0,27} = (2,45 + 1,23 \cdot 0,7) \cdot (4,38)^{-0,27} = 2,22$$

Load 2 (= normal force in the chord) causes to the chord a hot spot stress range:

$$\Delta\sigma_{hs.0.2} = \frac{SCF_{N.0} \cdot (k_1 \cdot \Delta N_0)}{A_0} = \frac{2,22 \cdot (1,5 \cdot 200000)}{5924} = 112\ N/mm^2$$

The total value of the hot spot stress range in the chord is calculated by superposing the effects of Load 1 and Load 2:

$$\Delta\sigma_{hs.0} = \Delta\sigma_{hs.0.1} + \Delta\sigma_{hs.0.2} = 220 + 112 = 332\ N/mm^2$$

Check whether the general conditions related to fatigue assessment are satisfied in respect to the highest stress fluctuation (condition (4.10)):

$$\gamma_{Ff} = 1,0 \quad \text{partial safety factor for fatigue load}$$

$$\gamma_{Ff} \cdot \Delta\sigma_{max} = 1,0 \cdot 332 = 332\ N/mm^2 \le 1,5 f_y = 1,5 \cdot 420 = 630\ N/mm^2 \quad OK$$

Since the chord has same wall thickness as the brace member (t = 8 mm), the same S-N curve can be applied for the chord. Thereby the same fatigue strength value can be applied for the chord, as calculated earlier for the brace member at load cycles $N = 5 \cdot 10^4$:

$$\Delta\sigma_{hs.R} = 476\ N/mm^2$$

Next, compare the hot spot stress range to the fatigue strength of the chord:

$$\frac{\gamma_{Ff}\Delta\sigma_{hs.0}}{\Delta\sigma_{hs.R}/\gamma_{Mf}} = \frac{1,0 \cdot 332}{476/1,35} = 0,942 \le 1,0 \quad OK$$

Hence it can be concluded, that the results obtained by the hot spot stress method for the brace member and for the chord are, in practice, quite similar to the results obtained in the previous Example for the same joint by the nominal stress method. In general, the hot spot stress method enables better accuracy for the impact of different parameters of the joint.

Comparison S420 vs S355:
Similar to Example 4.1.

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5. PREVENTION OF BRITTLE FRACTURE AND LAMELLAR TEARING IN STRUCTURAL HOLLOW SECTIONS

5.1 Brittle fracture

Brittle fracture is a rapid, even at sonic speed propagating failure without any clearly distinguishable plastic deformation. Brittle fracture may be triggered by an initial crack (for example a weld defect) in a statically loaded member, or by a fatigue grown crack or weld defect, whereafter the fracture propagates rapidly also in a defect-free structure. Brittle fracture leads usually to more serious consequences than a ductile failure. During a ductile failure, plastic hinges are developed in the structure to change the distribution of the loading, and the failure can be limited to be local.

The probability for brittle fracture depends on the following factors:

- steel strength grade
- stress level and state
- thickness of the material
- loading rate
- operating temperature and
- steel toughness

A member with high strength and thick walls is more sensitive to brittle fracture than one with low strength and thin walls. A high loading rate increases the risk of brittle fracture, and so does a low operating temperature. A tougher steel grade is better in low temperatures. Characteristics indicating the susceptibility to brittle fracture are transition temperature and impact toughness.

The fracture mode of steel turns from ductile to brittle within the transition temperature region. The transition temperature is not an explicit material property, but it depends on the testing method and on the criteria set for the brittle behaviour. As a testing method the Charpy-V test is commonly used. In the test, an impact test bar having a 2 mm deep V-notch is broken using a swinging hammer at a specified temperature. The size of the standard test bar is 10×10 mm. The transition temperature is defined as the temperature at which the energy used to break the test bar is at least 27 J. On high strength fine grain steels also other criteria such as 30 J and 40 J are applied.

5.2 Prevention of brittle fracture according to Eurocode

Provisions to prevent brittle fracture are presented in Part EN 1993-1-10 of Eurocode. The provisions apply for members subject to tension, as well as for welded and fatigue loaded members where a part of the stress cycle is tension. The instructions apply to all fatigue loaded members and detail categories presented in EN 1993-1-9, when the stresses are calculated as nominal stresses [8,9,10].

For members not subject to tension, welding or fatigue load, the rules may be conservative [8,9,10].

In respect to brittle fracture, according to EN 1993-1-10 it is not needed to set requirements for fracture toughness for members which are subjected to compression only. Structural members may contain, however, residual stresses from fabrication (rolling, cold-forming, welding), which cause tension even in the compressed members. That is why EN 1993-1-1 requires that the compressed members should also have sufficient fracture toughness, the criteria of which can be determined in the National Annex [4,5,6]. The recommendation given in EN 1993-1-1 is to apply the method of EN 1993-1-10 for the compressed members by supposing a stress level $\sigma_{Ed} = 0{,}25 f_y(t)$ for the member, see Table 5.1.

> *Finnish National Annex to standard EN 1993-1-1 [7]:*
>
> *For compressed members the recommended value of Eurocode $\sigma_{Ed} = 0{,}25 f_y(t)$ is used.*

The method given in EN 1993-1-10 is based on assessment by fracture mechanics and the calculation model developed therefrom. The subject is presented in more details in [12].

To facilitate the practical design work, there is in EN 1993-1-10 a table (EN 1993-1-10: Table 2.1) that has been prepared by applying the aforementioned calculation model and presenting tabulated values for the maximum permitted material thicknesses of the member for different reference temperatures T_{Ed} and for different steel grades by their nominal yield strength and impact toughness values.

The herein presented Table 5.1, which is based on [13], is prepared by the corresponding assessment by fracture mechanics, having however some parameters slightly changed in [13] after the publication of EN 1993-1-10 [8,9,10]. Furthermore, the values of material thickness have been presented with 1 mm precison instead of 5 mm precision (on small thicknesses 5 mm scaling is too coarse). It is anticipated that the table presented in [13] will in future substitute the table currently (= when this text is being written) presented in Eurocode.

Cold-forming itself does not cause (nor has caused) the need to change the numerical values of the material thicknesses presented in the table. The effects of cold-forming shall be taken into account when calculating the reference temperature T_{Ed} that is applied as a parameter in the table, as presented later on.

The material thicknesses presented in the table are grouped for three different stress levels σ_{Ed} as follows [8,9,10]:

a) $\sigma_{Ed} = 0{,}75 f_y(t)$ *(5.1a)*
b) $\sigma_{Ed} = 0{,}50 f_y(t)$ *(5.1b)*
c) $\sigma_{Ed} = 0{,}25 f_y(t)$ *(5.1c)*

where σ_{Ed} is the nominal stress acting at the potential initial point of fracture, calculated according to the theory of elasticity using <u>serviceability limit state</u> loads according to the frequent load combination (see clause 7.1.4), which approximately corresponds to the load combination at accidental limit state (regarding the displacement calculations at serviceability limit state, Finnish National Annex to EN 1993-1-1 defines to apply the characteristic load combination which, if applied in brittle fracture assessment, leads to conservative value for σ_{Ed})

$f_y(t)$ is the nominal yield strength of the material, where $f_y(t)$ can be determined either from the formula:

$$f_y(t) = f_{y.nom} - 0,25\frac{t}{t_0} \quad [N/mm^2] \qquad (5.2)$$

where t is the plate thickness in millimetres and t_0 = 1 mm
or $f_y(t)$ can be adopted from the relevant material standards according to R_{eH} -values given therein (see Chapter 1)

When applying Table 5.1, linear interpolation can be used. Because the stresses are calculated at serviceability limit state, the σ_{Ed} -values are in practice normally between σ_{Ed} = (0,50 ... 0,75) x $f_y(t)$ and the level of σ_{Ed} = 0,75$f_y(t)$ can be used as a conservative value. Extrapolation beyond the extreme values of the table is not allowed.

When using Table 5.1 the reference temperature T_{Ed} of the member to be considered shall be known. The reference temperature is determined in Eurocode as follows [8,9,10]:

$$T_{Ed} = T_{md} + \Delta T_r + \Delta T_\sigma + \Delta T_R + \Delta T_{\dot{\varepsilon}} + \Delta T_{\varepsilon.cf} \qquad (5.3)$$

where T_{md} is the design value of the lowest operating temperature for the member, see EN 1991-1-5

ΔT_r is an adjustment for radiation loss, see EN 1991-1-5

ΔT_σ is the adjustment for stress and yield strength of the material, crack imperfection and member shape and dimensions, ΔT_σ = 0 when using Table 5.1

ΔT_R is a safety allowance, if required, to reflect different reliability levels for different applications, the value of which can be chosen in the National Annex:

Finnish National Annex to standard EN 1993-1-10 [11]:
The recommended value of Eurocode $\Delta T_R = 0$ is used.

$\Delta T_{\dot{\varepsilon}}$ is the adjustment for strain rate (loading rate) other than the reference strain rate (see formula 5.4)

$\Delta T_{\varepsilon.cf}$ is the adjustment for the degree of cold-forming ε_{cf} according to clause 5.2.1

As a reference value for strain rate the value $\dot{\varepsilon}_0 = 4 \times 10^{-4}/sek$ has been used, which covers the dynamic action effects for most transient and persistent design situations. For other strain rate values $\dot{\varepsilon}$ (e.g. impact loads) an adjustment $\Delta T_{\dot{\varepsilon}}$ is needed, that is calculated as follows [8,9,10]:

$$\Delta T_{\dot{\varepsilon}} = \frac{1440 - f_y(t)}{550} \cdot \left(ln\frac{\dot{\varepsilon}}{\dot{\varepsilon}_0}\right) \quad [°C] \qquad (5.4)$$

On cold-formed hollow sections, if the structure is not subject to impact loads, when using Table 5.1 according to that presented above, expression (5.3) simplifies in Finland to the following reduced form:

$$T_{Ed} = T_{md} + \Delta T_r + \Delta T_{\varepsilon.cf} \tag{5.5}$$

where the lowest operating temperature of the member T_{md} is determined according to Part EN 1991-1-5 of Eurocode and its National Annex [1,2,3]. In Finland the design value of the lowest outdoor operating temperature varies in the southwards-region from Oulu within the range of -35...-45 °C and correspondingly in the northwards-region within the range of -40...-50 °C [3].

Standard EN 1991-1-5 is, however, incomplete in respect that it doesn't give any value for the adjustment factor ΔT_r applied in expression (5.3). Factor ΔT_r relates to a radiation loss that takes place outdoors under a clear star sky (for example in case of a bridge deck). In [12] the value ΔT_r = -5 °C has been presented.

For the Charpy-V impact toughness values specified in different temperatures, the following correlation can be used if needed [8,9,10]:

$$T_{40J} = T_{27J} + 10\ [°C] \tag{5.6}$$

$$T_{30J} = T_{27J} + 0\ [°C] \tag{5.7}$$

Table 5.1 The maximum permissible wall thickness t (mm) for cold-formed hollow sections [13]

Steel grade	Subgrade	Charpy energy KV		Reference temperature T_{Ed} (ºC)													
EN 10219		T (ºC)	(J_{min})	10	0	-10	-20	-30	-40	-50	-60	-70	-80	-90	-100	-110	-120
				$\sigma_{Ed} = 0{,}75\, f_y(t)$													
S235	JRH	20	27	62	51	43	36	30	25	21	18	16	14	12	11	10	9
S275	J0H	0	27	78	65	54	45	37	31	26	22	18	15	13	11	10	9
	J2H	-20	27	112	94	78	65	54	45	37	31	26	22	18	15	13	11
	NH, MH	-20	40	132	111	93	77	64	53	44	37	31	25	21	18	15	13
	NLH, MLH	-50	27	185	158	134	112	94	78	65	54	45	37	31	26	22	18
S355	J0H	0	27	63	52	43	35	29	24	20	16	14	11	10	8	7	6
	J2H	-20	27	92	76	63	52	43	35	29	24	20	16	14	11	10	8
	K2H, NH, MH	-20	40	109	91	75	62	51	42	35	28	23	19	16	13	11	9
	NLH, MLH	-50	27	155	131	110	92	76	63	52	43	35	29	24	20	16	14
S420	MH	-20	40	95	79	65	53	44	36	29	24	19	16	13	11	9	7
	MLH	-50	27	137	115	96	80	66	54	44	36	29	24	20	16	13	11
S460	NH, MH	-20	40	88	73	60	49	40	32	26	21	17	14	12	9	8	6
	NLH, MLH	-50	27	128	107	89	73	60	49	40	33	27	22	18	14	12	10
				$\sigma_{Ed} = 0{,}50\, f_y(t)$													
S235	JRH	20	27	90	77	65	55	47	41	35	31	27	24	21	19	18	16
S275	J0H	0	27	113	96	81	69	58	49	42	36	31	27	24	21	18	17
	J2H	-20	27	156	133	113	96	81	69	58	49	42	36	31	27	24	21
	NH, MH	-20	40	179	154	132	112	95	81	68	58	49	42	36	31	27	23
	NLH, MLH	-50	27	199	199	181	156	133	113	96	81	69	58	49	42	36	31
S355	J0H	0	27	96	80	68	57	48	40	34	29	25	21	18	16	14	13
	J2H	-20	27	133	113	96	80	68	57	48	40	34	29	25	21	18	16
	K2H, NH, MH	-20	40	155	132	112	95	80	67	56	47	40	34	28	24	21	18
	NLH, MLH	-50	27	199	181	156	133	113	96	80	68	57	48	40	34	29	25
S420	MH	-20	40	139	118	99	83	70	58	49	41	34	29	24	20	17	15
	MLH	-50	27	190	164	140	119	100	84	71	59	49	41	34	29	24	21
S460	NH, MH	-20	40	130	110	93	78	65	54	45	37	31	26	22	18	16	14
	NLH, MLH	-50	27	180	155	132	111	94	78	65	54	45	38	31	26	22	19
				$\sigma_{Ed} = 0{,}25\, f_y(t)$													
S235	JRH	20	27	134	116	101	88	77	67	59	53	47	42	39	35	33	31
S275	J0H	0	27	167	145	126	109	95	82	71	62	55	49	43	39	35	32
	J2H	-20	27	199	191	167	145	126	109	95	82	71	62	55	49	43	39
	NH, MH	-20	40	199	199	189	165	144	125	108	94	81	71	62	55	48	43
	NLH, MLH	-50	27	229	202	199	199	191	167	145	126	109	95	82	71	62	55
S355	J0H	0	27	150	130	112	97	83	72	62	54	47	42	37	33	30	27
	J2H	-20	27	198	173	150	130	112	97	83	72	62	54	47	42	37	33
	K2H, NH, MH	-20	40	199	197	172	149	129	111	96	83	71	62	54	47	41	37
	NLH, MLH	-50	27	210	199	199	198	173	150	130	112	97	83	72	62	54	47
S420	MH	-20	40	199	183	159	138	118	102	87	75	64	55	48	42	37	33
	MLH	-50	27	199	199	199	185	160	139	119	103	88	75	65	56	48	42
S460	NH, MH	-20	40	199	176	152	131	113	96	82	71	61	52	45	39	34	30
	NLH, MLH	-50	27	199	199	199	177	153	132	114	97	83	71	61	53	45	39

Linear interpolation can be used when applying the table. Extrapolation beyond the extreme values is not allowed.

5.2.1 Effect of cold-forming to the brittle fracture, adjustment $\Delta T_{\varepsilon.cf}$ of the reference temperature for cold-formed hollow sections

Cold-forming causes in steel strain hardening and decrease of impact toughness.

Increase of strength due to cold-forming as well as decrease of impact toughness give rise to tendency of brittle fracture. According to EN 1993-1-10 this shall be taken into account when determining the reference temperature T_{Ed}, which shall be adjusted due to cold-forming by using the adjustment $\Delta T_{\varepsilon.cf}$. Eurocode is, however, incomplete in respect that it doesn't give provisions how to determine this adjustment.

The instructions presented herein are based on [13], which has been prepared especially to supplement the incomplete provisions of Eurocode.

Simplified method:

The following procedure can be applied as a conservative simplification. The method is addressed only for cold-formed structural hollow sections conforming to EN 10219 [13].

- circular hollow sections:

$$\Delta T_{\varepsilon.cf} = 0 \qquad \text{for } d_i/t > 30 \tag{5.8a}$$

$$\Delta T_{\varepsilon.cf} = -20\,°C \qquad \text{for } d_i/t \le 30 \tag{5.8b}$$

- square and rectangular hollow sections:

$$\Delta T_{\varepsilon.cf} = -35\,°C \qquad \text{for } t \le 16\ mm \tag{5.9a}$$

$$\Delta T_{\varepsilon.cf} = -45\,°C \qquad \text{for } 16\ mm < t \le 40\ mm \tag{5.9b}$$

where d_i is the <u>internal</u> diameter
t is the wall thickness

Rigorous method:

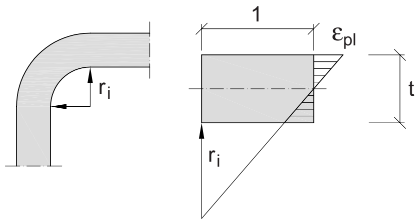
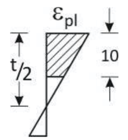
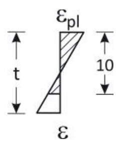
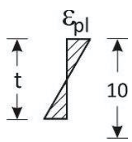
The method applies generally to all cold-formed sections [13].

$$\Delta T_{\varepsilon.cf} = 0 \qquad \text{for } \varepsilon_{cf} \le 2\ \% \qquad \text{but in other cases:} \tag{5.10a}$$

$$\Delta T_{\varepsilon.cf} = -3 \cdot \varepsilon_{cf}\ [°C] \qquad \text{however } \Delta T_{\varepsilon.cf} \ge -45\,°C \tag{5.10b}$$

where, instead of the degree of cold-forming ε_{cf}, effective strain ε_{eff} caused by cold-forming shall be applied. Effective strain ε_{eff} is obtained from Table 5.2.

Table 5.2 Cold-formed corner. Plastic strain ε_{pl} and effective strain ε_{eff} [13]

Plastic strain ε_{pl} :		
		$\varepsilon_{pl} = \dfrac{1}{2\left(\dfrac{r_i}{t}\right)+1}$
Effective strain ε_{eff} :		
t [mm]	ε_{pl} -distribution	ε_{eff}
≥ 20		$\varepsilon_{pl} \cdot \left[1 - \dfrac{10}{t}\right]$
< 20 ≥ 10		$\dfrac{\varepsilon_{pl}}{2} \cdot \left[\dfrac{t}{20} + \dfrac{(20-t)^2}{20t}\right]$
< 10		$\dfrac{\varepsilon_{pl}}{2} \cdot \dfrac{t}{10}$

5.2.2 The lowest operating temperature for cold-formed SSAB Domex Tube structural hollow sections

Table 5.1 defines the maximum permissible material thickness corresponding to each reference temperature T_{Ed} for different steel grades. In Table 5.3, which is based on Table 5.1, the similar evaluation is performed ‘conversely‘: the table defines for square and rectangular hollow sections, having the wall thickness t = max. 12,5 mm, the lowest permissible reference temperature T_{Ed} and the corresponding operating temperature T_{md} according to expression (5.3), when the effect of cold-forming is taken into account using the adjustment $\Delta T_{\varepsilon .cf}$ according to the Simplified method presented in clause 5.2.1, and when the other adjustment terms for the reference temperature in expression (5.3) need not be taken into account. In Table 5.4 the corresponding evaluation is applied for circular hollow sections.

Table 5.3 The lowest permissible reference temperature T_{Ed} and operating temperature T_{md}. Square and rectangular hollow sections

Square and rectangular hollow sections [a)]									
Steel grade		**EN 10219 structural hollow sections**				**SSAB Domex Tube structural hollow sections**			
		Impact toughness		The lowest design value of temperature [b)]		Impact toughness		The lowest design value of temperature [b)]	
		Test temperature	Charpy-V impact energy	T_{Ed} (ºC)	T_{md} (ºC)	Test temperature	Charpy-V impact energy	T_{Ed} (ºC)	T_{md} (ºC)
S235	JRH	20 ºC	27 J	-85	-50	-40 ºC	27 J	-145	-110
S355	**J2H**	**-20 ºC**	**27 J**	**-95**	**-60**	**-40 ºC [c)]**	**40 J [c)]**	**-125 [d)]**	**-90 [d)]**
S420	**MH**	**-20 ºC**	**40 J**	**-90**	**-55**	**-40 ºC [e)]**	**40 J [e)]**	**-110 [f)]**	**-75 [f)]**
S460	MH	-20 ºC	40 J	-85	-50	-40 ºC	27 J	-95	-60

a) Dimensional manufacturing range by SSAB:
- square and rectangular: t = max. 12,5 mm

b) $T_{md} = T_{Ed} - \Delta T_{\varepsilon.cf}$
where
T_{Ed} is the reference temperature according to Table 5.1
T_{md} is the operating temperature of the member after adjusting the reference temperature by the cold-forming factor $\Delta T_{\varepsilon.cf}$
$\Delta T_{\varepsilon.cf}$ is determined using the Simplified method presented in clause 5.2.1 [13]

c) SSAB Domex Tube Double Grade.
d) SSAB Domex Tube Double Grade, when designed as grade S355.

e) SSAB Domex Tube Double Grade.
f) SSAB Domex Tube Double Grade, when designed as grade S420.

- The values in the table have been calculated with wall thickness t = 12,5 mm. When having smaller wall thicknesses the values are on the safe side.
- The values in the table have been calculated at serviceability limit state assuming the stress level $\sigma_{Ed} = 0{,}75\, f_y$, hence the values of the table can be applied as conservative values also at lower stress levels.

Table 5.4 The lowest permissible reference temperature T_{Ed} and operating temperature T_{md}. Circular longitudinally welded hollow sections

Circular longitudinally welded hollow sections a)									
		EN 10219 structural hollow sections				**SSAB Domex Tube structural hollow sections**			
		Internal diameter $d_i / t \le 30$							
		Impact toughness		The lowest design value of temperature b)		Impact toughness		The lowest design value of temperature b)	
Steel grade		Test temperature	Charpy-V impact energy	T_{Ed} (ºC)	T_{md} (ºC)	Test temperature	Charpy-V impact energy	T_{Ed} (ºC)	T_{md} (ºC)
S235	JRH	20 ºC	27 J	-85	-65	-40 ºC	27 J	-145	-125
S355	**J2H**	**-20 ºC**	**27 J**	**-95**	**-75**	**-40 ºC c)**	**40 J c)**	**-125 d)**	**-105 d)**
S420	**MH**	**-20 ºC**	**40 J**	**-90**	**-70**	**-40 ºC e)**	**40 J e)**	**-110 f)**	**-90 f)**
S460	MH	-20 ºC	40 J	-85	-65	-40 ºC	27 J	-95	-75
		Internal diameter $d_i / t > 30$							
		Impact toughness		The lowest design value of temperature b)		Impact toughness		The lowest design value of temperature b)	
Steel grade		Test temperature	Charpy-V impact energy	T_{Ed} (ºC)	T_{md} (ºC)	Test temperature	Charpy-V impact energy	T_{Ed} (ºC)	T_{md} (ºC)
S235	JRH	20 ºC	27 J	-85	-85	-40 ºC	27 J	-145	-145
S355	**J2H**	**-20 ºC**	**27 J**	**-95**	**-95**	**-40 ºC c)**	**40 J c)**	**-125 d)**	**-125 d)**
S420	**MH**	**-20 ºC**	**40 J**	**-90**	**-90**	**-40 ºC e)**	**40 J e)**	**-110 f)**	**-110 f)**
S460	MH	-20 ºC	40 J	-85	-85	-40 ºC	27 J	-95	-95

a) Dimensional manufacturing range by SSAB:
- circular longitudinally welded: t = max. 12,5 mm
- circular spirally welded: t = max. 20 mm

b) $T_{md} = T_{Ed} - \Delta T_{\varepsilon.cf}$

where:

T_{Ed} is the reference temperature according to Table 5.1

T_{md} is the operating temperature of the member after adjusting the reference temperature by the cold-forming factor $\Delta T_{\varepsilon.cf}$

$\Delta T_{\varepsilon.cf}$ is determined using the Simplified method presented in clause 5.2.1 [13]

c) SSAB Domex Tube Double Grade.

d) SSAB Domex Tube Double Grade, when designed as grade S355.

e) SSAB Domex Tube Double Grade.

f) SSAB Domex Tube Double Grade, when designed as grade S420.

- The values in the table have been calculated with wall thickness t = 12,5 mm. When having smaller wall thicknesses the values are on the safe side.
- The values in the table are valid also for circular spirally welded structural hollow sections (when t = 12,5 mm), provided testing of impact toughness is agreed for them as presented in the table.
- The values in the table have been calculated at serviceability limit state assuming the stress level $\sigma_{Ed} = 0{,}75 f_y$, hence the values of the table can be applied as conservative values also at lower stress levels.

5.3 Lamellar tearing

Structures to be fabricated using structural hollow sections generally contain also various welded plate components (base plates of columns, end-plates of beams and trusses, etc.). Therefore lamellar tearing is in the following presented in respect to hollow sections and plate materials together.

Lamellar tearing can take place in a welded joint, if the plate is subject to tension perpendicular to its surface (Figure 5.1). Tension can be caused by:
- stresses due to welding shrinkage in fabrication
- tensile stresses caused by external loading to the joint

Susceptibility to lamellar tearing depends on the following factors:
- properties of the material in the through-thickness direction
- design and geometry of the joint
- stiffness of the structure surrounding the joint (prevention of welding shrinkage)
- execution of welding
- direction and magnitude of the loading transmitted to the joint (Figure 5.2)

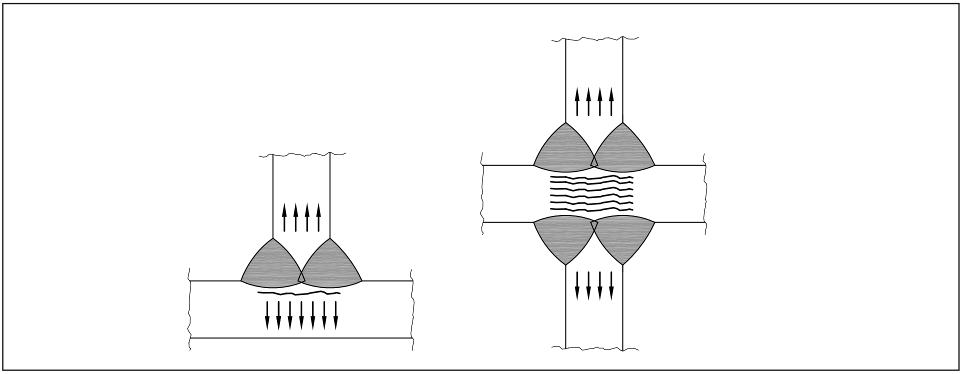


Figure 5.1 Lamellar tearing in the parent metal underneath the welded joint [8,9,10]

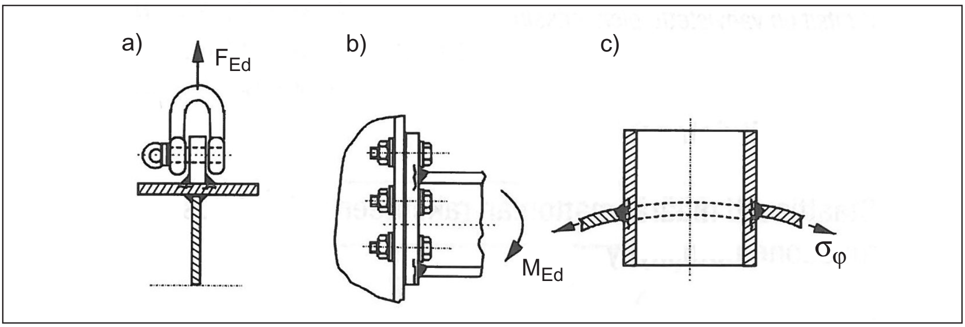


Figure 5.2 Examples of joints where lamellar tearing is critical:
a) lifting lugs on the plate's surface, b) end-plates of a beam or a column and c) joint of a tubular part to a plate or shell [14]

Through-thickness properties of the material

Ductility and deformation capability of hot rolled steels are smallest in the through-thickness direction of the plate. If the deformation capability of steel is not sufficient, especially when using high strength steel grades, lamellar tearing may occur in the welded joint. Lamellar tearing occurs underneath a welded joint as fractures parallel to the surface. Stiff fillet or butt welded L or T joints are especially sensitive for lamellar tearing, since they may become subject to high stresses in through-thickness direction [15].

The structures to be made of structural hollow sections typically contain joints that are welded to the surface of the hollow section (for example hollow section lattice structures), which themselves thereby create a potential situation for lamellar tearing.

During the manufacture of the steel, additional treatment can be used to improve the through-thickness properties of the product. High sulphur content particularly, even when below the limits set in general steel product standards, can increase the sensitivity to lamellar tearing. The sulphur content of SSAB Domex Tube structural hollow sections is decreased down to $S \leq 0{,}012$ %, which is less than half of the content allowed by EN 10219.

The through-thickness properties of the material and the risk for lamellar tearing can be assessed by measuring in a through-thickness tensile test the reduction of area at fracture i.e. the Z-value, see Table 5.5.

The plates tested in through-thickness direction are thereby called commonly as Z-plates. According to EN 10164 it is a supplement property that can be specified as an option in practice for any steel grades if agreed at the time of the order, provided that the material thickness is $t \geq 15$ mm. There exists three different quality classes for Z-plates in EN 10164, see Table 5.6.

Since EN 10164 does not cover materials with thickness less than 15 mm, the Z requirement can normally be neglected on SSAB Domex Tube structural hollow sections. On wall thicknesses *t* ≥ 15 mm of structural hollow sections (i.e. circular spirally welded structural hollow sections, when t = 16 / 18 / 20 mm) it is possible to agree, if needed, on measuring the reduction of area at fracture (Z-value) by through-thickness tensile test and classification as presented in EN 1993-1-10 (Z15 / Z25 / Z35). In this case the through-thickness tensile test will be performed according to EN 10164, but the Z classification shall not include the ultrasonic testing by EN 10164.

In regard to structural hollow sections, it has been observed by experience that lamellar tearing is a very rare phenomenon. This is also partly due to the fact that lamellar tearing risk generally decreases when having a smaller material thickness (see Table 5.9: factor Z_c). In design and execution it is, however, of course advisable to avoid solutions which themselves cause or increase the risk for lamellar tearing.

Table 5.5 Probability of lamellar tearing [16]

Reduction of area in through-thickness direction (%)	Probability of lamellar tearing
Z < 10	Possible in welded joints already when lightly loaded in through-thickness direction of the plate.
10 ≤ Z < 15	Possible in welded joints when moderately loaded in through-thickness direction of the plate.
15 ≤ Z < 20	Possible in welded joints when highly loaded in through-thickness direction of the plate.
20 ≤ Z < 35	Extremely rare.
Z ≥ 35	Extremely unlikely.

Table 5.6 Quality classes of Z-plates [17]

EN 10164:2004 Class for reduction of area at fracture	Requirements for reduction of area at fracture (Z-value)	
	Minimum mean value of three tests (%)	Minimum single test value (%)
Z15	15	10
Z25	25	15
Z35	35	25

Geometry and welding of the joint

In addition to material properties, the risk of lamellar tearing can be reduced by changing the geometry of the joint and execution of welding. When using higher energy in welding, the lamellar tearing tendency is generally slightly lower. This is due to a better penetration and wider weld, which distribute the deformations to a wider area in the through-thickness direction of the plate [15].

The structural geometry of the joints is advisable to design in such a way, that external loading does not cause perpendicular tension to the surface of the plate, example in Figure 5.3. Also by detail design of joints it is possible to reduce the risk to lamellar tearing, some examples in Figure 5.4.

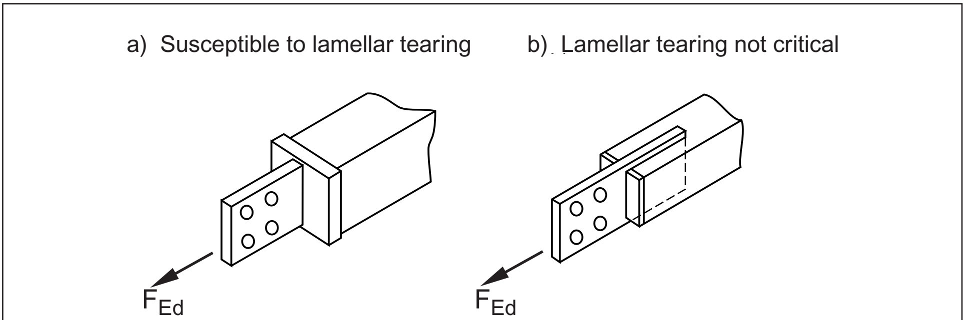


Figure 5.3 Splice joint. Design practices to reduce the risk for lamellar tearing

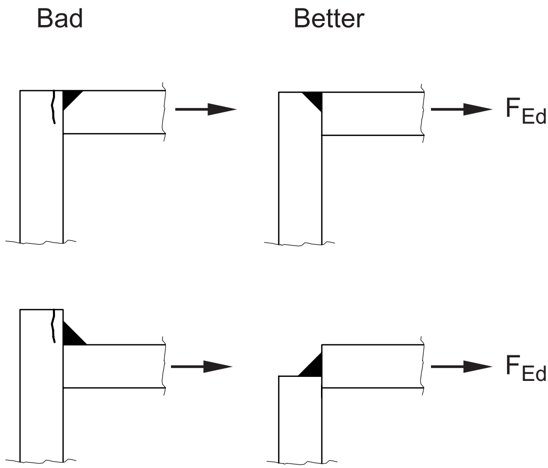


Figure 5.4 Corner joints in plated structures. Design practices to reduce the risk for lamellar tearing (does not concern 'corner joints' of structural hollow sections) [14]

The shrinkage stresses due to welding can be reduced by choosing correct welding sequence of the joints in the structural component, as well as by structural (geometrical) design and implementation of welding of the single joints. Regarding a single joint, the shrinkage stresses can be reduced for example as follows [18,19]:

- by reducing the amount of weld consumable needed
- by welding in as few passes as possible
- by using fusion face buttering in welding
- by using symmetrical welding sequence in double-sided weld grooves

The lamellar tearing due to shrinkage stresses caused by welding can be detected by non-destructive testing after welding, see Chapter 8.

5.4 Prevention of lamellar tearing according to Eurocode

Provisions to prevent lamellar tearing have been presented in Part EN 1993-1-10 of Eurocode. It is stated in EN 1993-1-10, however, that the provisions regarding lamellar tearing are valid only for grades S235-S460. The reason for this seems to be, that when preparing Part EN 1993-1-10, the reference standard EN 10164:1993 for the Z-quality classification applied only up to grade S500. Since the newer revision of EN 10164 (i.e. EN 10164: 2004) now covers steel grades up to S960, and since according to Part EN 1993-1-12 of Eurocode also the grades S500-S700 have been reached within the scope of Eurocode, it is appropriate to apply the lamellar tearing provisions of EN 1993-1-10 also for steels in grades S500-S700.

The steel material to be used shall be classified according to Table 5.7. The class to be applied in Table 5.7 can be determined in the National Annex. Eurocode recommends class 1 [8,9,10].

Finnish National Annex to standard EN 1993-1-10 [11]:
Class 1 is used.

Depending on the class in Table 5.7, the risk for lamellar tearing is prevented either:

- by requiring satisfactory Z-value for the material according to EN 10164 (Z15, Z25 or Z35) or
- by using testing after fabrication of the structure to verify whether lamellar tearing has occurred.

Hence, the idea of Eurocode is that (a) either Z-steel shall be used or (b) testing after fabrication (welding) of the structure shall be used to check whether lamellar tearing has occurred.

The Z-value can be determined, however, according to EN 10164 only for steels having material thickness $t \geq 15$ mm, why the Z requirement can normally be neglected on SSAB Domex Tube structural hollow sections. On wall thicknesses $t \geq 15$ mm of structural hollow sections (i.e. circular spirally welded structural hollow sections, when t = 16 / 18 / 20 mm) it is possible to agree, if needed, on measuring the reduction of area at fracture (Z-value) by through-thickness tensile test and classification as presented in EN 1993-1-10 (Z15 / Z25 / Z35). In this case the through-thickness tensile test will be performed according to EN 10164, but the Z classification shall not include the ultrasonic testing by EN 10164.

Table 5.7 Choice of quality class [8,9,10]

EN 1993-1-10: Class	Application guidance
1	All steel products and thicknesses listed in European standards for all applications.
2	Certain steel products and thicknesses listed in European standards and /or certain listed applications.

Lamellar tearing may be neglected if the following condition is satisfied [8,9,10]:

$$Z_{Ed} \leq Z_{Rd} \tag{5.11}$$

where Z_{Ed} is the required design Z-value, which depends on the magnitude of the prevented shrinkage of metal during welding

Z_{Rd} is the available design Z-value for the material according to EN 10164 (Z15, Z25, or Z35)

The design value for Z_{Ed} can be determined from the formula [8,9,10]:

$$Z_{Ed} = Z_a + Z_b + Z_c + Z_d + Z_e \qquad (5.12)$$

where Z_a , Z_b , Z_c , Z_d and Z_e are determined from Table 5.9.

In Part EN 1993-1-1 of Eurocode the condition (5.11) is relieved so that for buildings the choice of Z-value is recommended according to Table 5.8. The final choice can be determined in the National Annex [4,5,6].

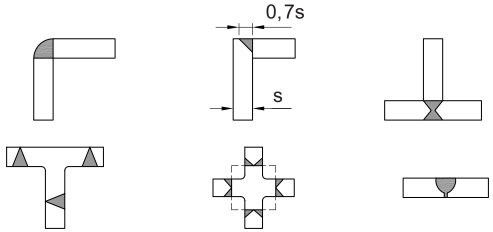
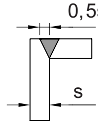
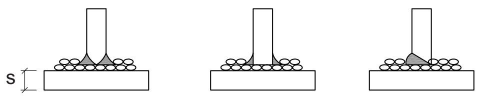
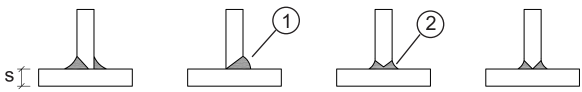
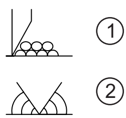
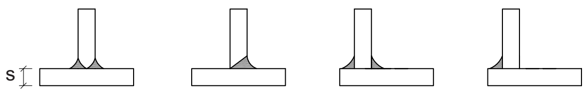
Finnish National Annex to standard EN 1993-1-1 [7]:

For buildings the values according to Table 5.8 recommended in Eurocode are used.

Table 5.8 Choice of quality class for buildings according to EN 10164 [4,5,6]

Design value according to expression (5.12) for Z_{Ed}	Required design value for Z_{Rd} expressed in the terms of Z-value according to EN 10164
$Z_{Ed} \leq 10$	–
$10 < Z_{Ed} \leq 20$	Z 15
$20 < Z_{Ed} \leq 30$	Z 25
$Z_{Ed} > 30$	Z 35

Table 5.9 Factors needed to determine Z_{Ed} [8,9,10]

a)	Weld depth relevant for straining from metal shrinkage	Effective weld depth a_{eff} (see Figure 5.5)	Throat thickness of the fillet weld	Z_i
		$a_{eff} \leq 7$ mm	a = 5 mm	$Z_a = 0$
		$7 < a_{eff} \leq 10$ mm	a = 7 mm	$Z_a = 3$
		$10 < a_{eff} \leq 20$ mm	a = 14 mm	$Z_a = 6$
		$20 < a_{eff} \leq 30$ mm	a = 21 mm	$Z_a = 9$
		$30 < a_{eff} \leq 40$ mm	a = 28 mm	$Z_a = 12$
		$40 < a_{eff} \leq 50$ mm	a = 35 mm	$Z_a = 15$
		$50 < a_{eff}$	a > 35 mm	$Z_a = 15$
b)	Shape and position of welds in T- and cruciform- and corner-connections			$Z_b = -25$
		Corner joints 		$Z_b = -10$
		Single run fillet welds $Z_a = 0$ or fillet welds with $Z_a > 1$ when using buttering with low strength weld material 		$Z_b = -5$
		Multi-run fillet welds 		$Z_b = 0$
		Partial and full penetration welds  with appropriate welding sequence to reduce the shrinkage effects 		$Z_b = 3$
		Partial and full penetration welds 		$Z_b = 5$
		Corner joints 		$Z_b = 8$

(continues)

Table 5.9 Factors needed to determine Z_{Ed} [8,9,10]
(continued)

c)	Effect of material thickness s on the restraint to shrinkage.	$s \le 10$ mm		$Z_c = 2$ *
		$10 < s \le 20$ mm		$Z_c = 4$ *
		$20 < s \le 30$ mm		$Z_c = 6$ *
		$30 < s \le 40$ mm		$Z_c = 8$ *
		$40 < s \le 50$ mm		$Z_c = 10$ *
		$50 < s \le 60$ mm		$Z_c = 12$ *
		$60 < s \le 70$ mm		$Z_c = 15$ *
		$70 < s$		$Z_c = 15$ *
d)	Remote restraint of shrinkage after welding by other portions of the structure.	Low restraint:	Free shrinkage possible (for example T joints)	$Z_d = 0$
		Medium restraint:	Free shrinkage restricted (for example diaphragms in box girders)	$Z_d = 3$
		High restraint:	Free shrinkage not possible (for example stringers in orthotropic deck plates)	$Z_d = 5$
e)	Influence of preheating.	Without preheating		$Z_e = 0$
		Preheating $\ge 100\ °C$		$Z_e = -8$

* May be reduced by 50 % for material stressed, in the through-thickness direction, by compression due to predominantly static loads.

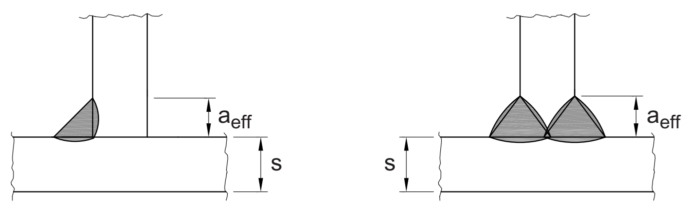


Figure 5.5 Effective weld depth a_{eff} for shrinkage [8,9,10]

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6. FIRE DESIGN OF STRUCTURAL HOLLOW SECTIONS

In fire situation the temperature of steel rises together with the temperature of the gases in fire compartment. As the temperature of steel rises, its strength and deformation properties change. According to their purpose of use, structures have different fire resistance requirements (for example load carrying and compartmentation requirements).

Often it is necessary to protect steel components in order to delay the increase of temperature during fire. The fire protection of structural hollow sections can be executed in many ways, for example using mineral wool or fire protection painting (also called as intumescent coating). The heat retention capacity of the hollow section can be improved for example by fillling it with concrete. Hollow sections are profitable regarding fire protection, since their section factor (the ratio between the surface area exposed to fire and the volume of the steel) is smaller than the section factor of open sections. In addition, hollow sections with their rounded corners are especially well-suited for fire protection painting. Various fire protection methods are presented in clause 6.8.

The resistance of a hollow section can be calculated in a fire situation in two ways: either by the properties of the material (yield strength and modulus of elasticity) in a fire situation or by determining the critical temperature of the steel structure as a function of utilisation ratio. The non-uniform temperature distribution of the steel component can be taken into account when fire design is carried out by the material properties. When applying the critical temperature method, the temperature distribution of the steel component is supposed to be uniform. The methods are illustrated in Figures 6.1 and 6.2.

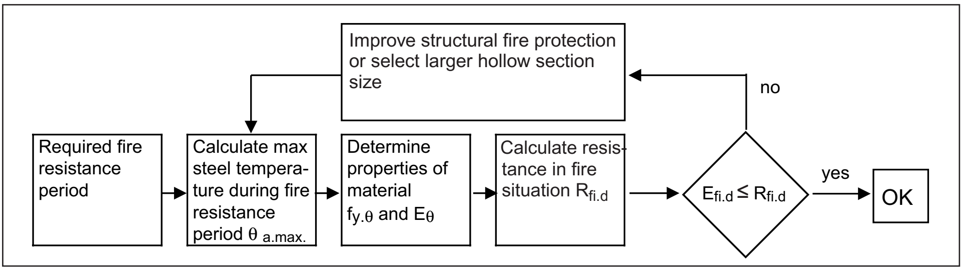


Figure 6.1 Fire design based on properties of the material

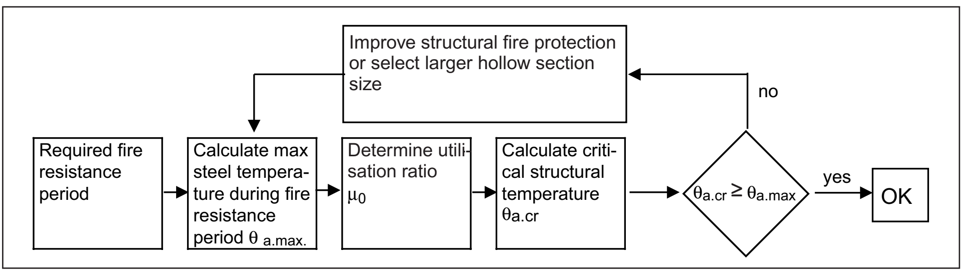


Figure 6.2 Fire design based on critical temperature

6.1 Temperature development in fire compartment

The temperature development in real fire can be presented according to Figure 6.3, when the fire is allowed to evolve freely. The duration times of the fire phases are not in correct scale to each other, but they may vary considerably. The combustibles in the fire compartment catch fire at the flashover phase, when the temperature of the fire compartment is ca. 350 - 550 ºC. During the full fire the temperature of the fire compartment can be over 1000 ºC [1].

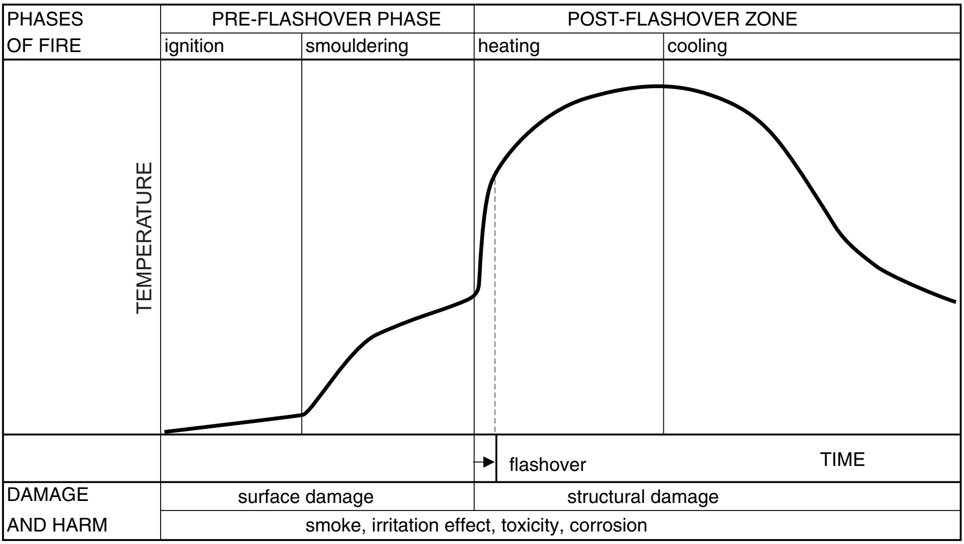


Figure 6.3 Phases during a real fire and temperature development in fire compartment [2,3]

The development of the temperature in the fire compartment can be modelled either with nominal temperature-time curves (for example standard fire) having been defined for a certain period of time, or with parametric temperature-time curves taking into account e.g. magnitude of the fire load, size of the fire compartment and amount of ventilation openings [6,7].

6.1.1 Standard fire

The commonly used standard temperature-time curve is presented in Figure 6.4. The temperature develops according to standard ISO 834 as follows [6,7]:

$$\theta_g = 20 + 345 \log(8t + 1) \tag{6.1}$$

where θ_g is the gas temperature in the fire compartment [ºC]
t is the time [min]

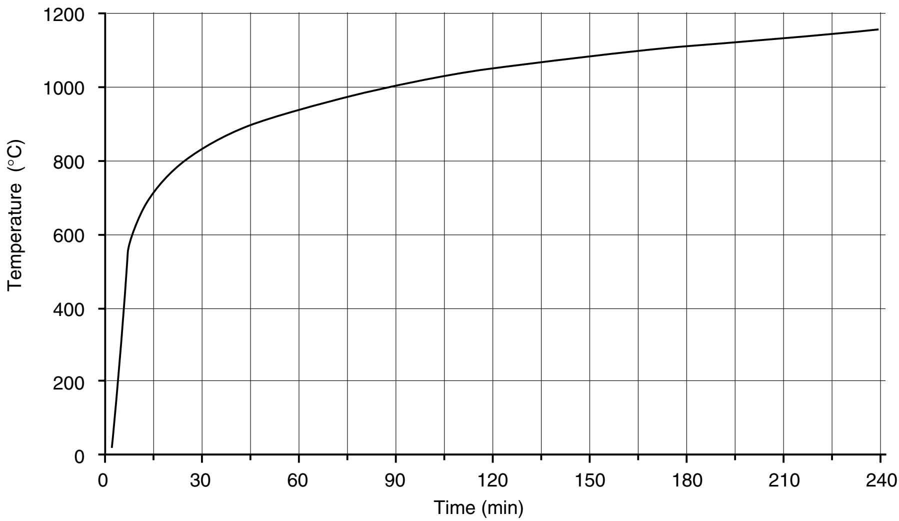


Figure 6.4 Temperature-time curve according to standard ISO 834

6.1.2 Parametric fire model

According to Finnish National Annex the parametric fire model may be used as presented in Part EN 1991-1-2 of Eurocode [8]. It is applicable for fire compartments not larger than 500 m^2, with no openings in the roof and with a height not over 4 m [6,7]. Apart from time, the parametric temperature-time curve is also influenced by fire load, openings during fire and the thermal properties of the surrounding structures (e.g. the thermal conductivity of the walls) [2,3]. The temperature decrease in the cooling phase is also taken into account, if the required fire resistance time is longer than the calculated combustion time. The difference between the standard fire curve and the parametric curve is presented in Figure 6.5.

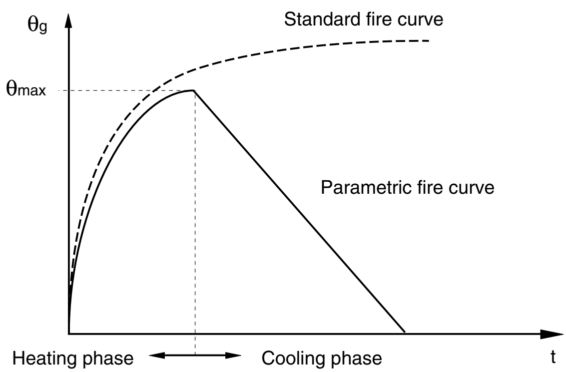


Figure 6.5 Temperature-time curves of standard fire and parametric fire

6.2 Strength and modulus of elasticity of steel in fire situation

The strength and modulus of elasticity of steel decrease at elevated temperatures. Moreover, the whole steel cross-section heats up during fire. Normally the steel structures have to be protected against fire, or it has to be proved by calculations that the structures are capable of withstanding the fire in an unprotected state for the required time.

Material models and rules applied for the change of material properties of cold-formed structural hollow sections in fire situations are presented herein, being the same as for the corresponding flat steels.

The stress-strain relationship for steel at elevated temperatures is presented in Figure 6.6. The change of strength and deformation properties of steel at elevated temperatures are defined using the following reduction factors [9,10,11]:

- effective yield strength, relative to the yield strength at 20 ºC: $k_{y.\theta} = f_{y.\theta} / f_y$
- proportional limit, relative to the yield strength at 20 ºC:
    - cross-section Classes 1-3: $k_{p.\theta} = f_{p.\theta} / f_y$
    - cross-section Class 4: $k_{p0,2.\theta} = f_{p0,2.\theta} / f_y$
- slope of linear elastic range, relative to slope at 20 ºC: $k_{E.\theta} = E_{a.\theta} / E_a$

The modulus of elasticity during fire is calculated using reduction factor $k_{E.\theta}$. The proportional limit can be determined using reduction factor $k_{p.\theta}$. At stress levels beyond the proportional limit a certain amount of permanent strain will remain in the steel. For the strength reduction factor basing on the proportional limit, a different value is used in Class 4 than in Classes 1-3, see Table 6.1. The effective yield strength is calculated using reduction factor $k_{y.\theta}$. The effective yield strength corresponds to the strain $\varepsilon_{y.\theta}$ = 0,02. In fire design, the resistances of the member are calculated in Classes 1-3 using reduction factor $k_{y.\theta}$, and in Class 4 using reduction factor $k_{p0,2.\theta}$ as a consequence of its smaller deformation capability (for more details see clause 6.6.3).

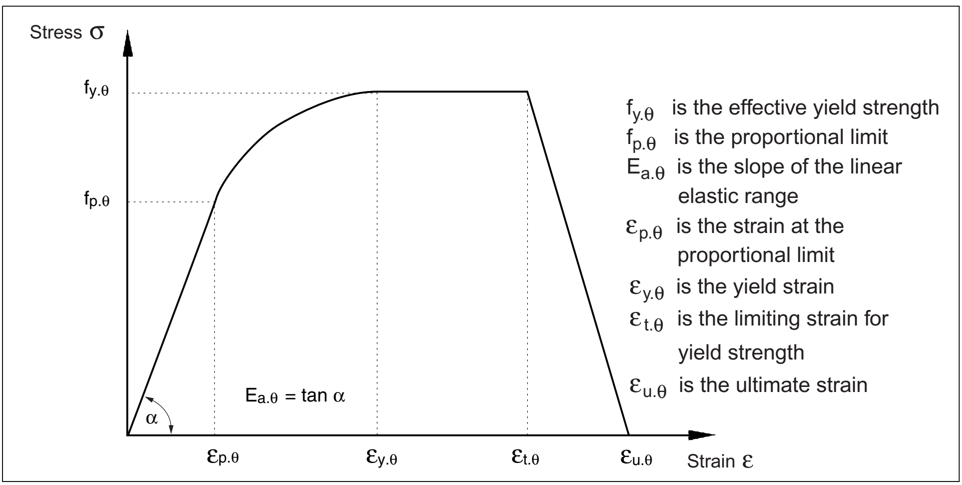


Figure 6.6 Carbon steel. Stress-strain relationship at elevated temperatures [9,10,11].

The effect of temperature to the reduction factors is presented in Table 6.1 and in Figure 6.7. The intermediate values can be calculated using linear interpolation. On Figure 6.7 it can be seen that the effective yield strength of steel decreases when temperature exceeds 400 ºC. However, the modulus of elasticity of steel decreases already when temperature exceeds 100 ºC.

Table 6.1 Carbon steel. The effect of temperature to the strength and modulus of elasticity [9,10,11]

Steel temperature	Reduction factors at temperature θ_a in relation to the values at 20 ºC			
	Reduction factor for effective yield strength (relative to f_y)	Reduction factor for proportional limit (relative to f_y)		Reduction factor for the slope of the linear elastic range (relative to E_a)
		Classes 1-3	Class 4	
θ_a	$k_{y.\theta} = f_{y.\theta} / f_y$	$k_{p.\theta} = f_{p.\theta} / f_y$	$k_{p0,2.\theta} = f_{p0,2.\theta} / f_y$	$k_{E.\theta} = E_{y.\theta} / E_a$
20 ºC	1,000	1,000	1,00	1,000
100 ºC	1,000	1,000	1,00	1,000
200 ºC	1,000	0,807	0,89	0,900
300 ºC	1,000	0,613	0,78	0,800
400 ºC	1,000	0,420	0,65	0,700
500 ºC	0,780	0,360	0,53	0,600
600 ºC	0,470	0,180	0,30	0,310
700 ºC	0,230	0,075	0,13	0,130
800 ºC	0,110	0,050	0,07	0,090
900 ºC	0,060	0,0375	0,05	0,0675
1000 ºC	0,040	0,0250	0,03	0,0450
1100 ºC	0,020	0,0125	0,02	0,0225
1200 ºC	0,000	0,0000	0,00	0,0000
For intermediate values of the steel temperature, linear interpolation may be used.				

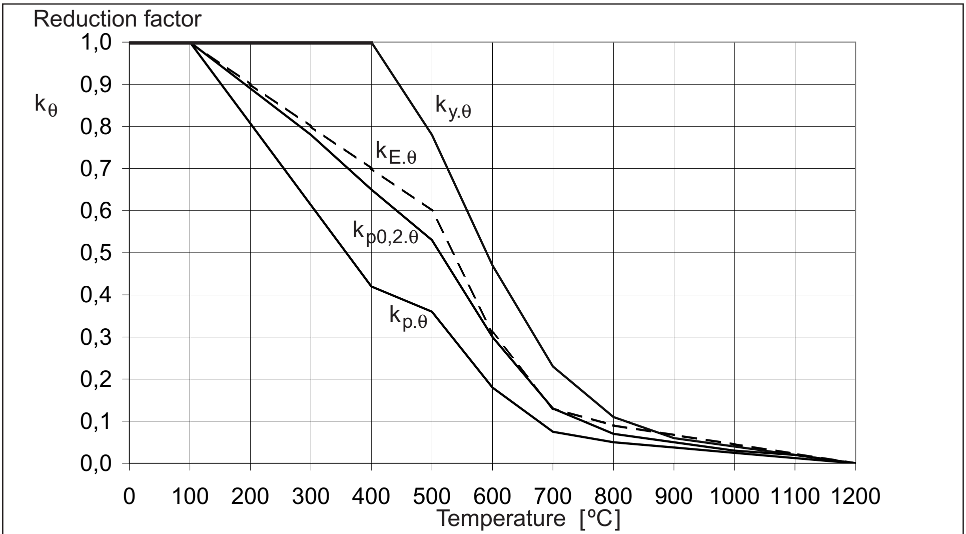


Figure 6.7 Carbon steel. Reduction factors for strength and modulus of elasticity as a function of temperature

6.3 Steel temperature development

During fire heat may transfer from the fire compartment to the steel structure through radiation, conduction or convection. Among others, the factors that affect the temperature rise are, temperature development in the fire compartment, the section factor of the steel member (see clause 6.3.4) and the fire protection [2,3]. Also the form of the cross-section has an essential influence on the temperature development of the cross-section. The fire design of steel structures in Part EN 1993-1-2 of Eurocode is based on so-called simple calculation models, whereby e.g. the temperature is supposed to be uniformly distributed in the whole cross-section. According to Eurocode it is also allowed to use more advanced calculating models; however, the decision on their use is to be given in the National Annex [9,10,11].

Finnish National Annex to standard EN 1993-1-2 [12]:

Advanced calculation methods may be used.
Detailed guidance is not presented.

6.3.1 Indoor unprotected steel structure

The increase of temperature in an unprotected steel structure is calculated as follows [9,10,11]:

$$\Delta\theta_{a.t} = k_{sh} \cdot \frac{A_m/V}{c_a \rho_a} \cdot \dot{h}_{net.d} \cdot \Delta t \qquad (6.2)$$

where
- k_{sh} is correction factor for the shadow effect
- A_m/V is the section factor for unprotected steel member [1/m], but not less than 10 m^{-1}
- A_m is the exposed-to-fire surface area of the member per unit length [m^2/m]
- V is the volume of the member per unit length [m^3/m]
- c_a is the specific heat of steel [J / (kgK)]
- $\dot{h}_{net.d}$ is the design value of the net heat flux per unit area [W/m^2], defined for the exposed-to-fire surface area of the member
- Δt is the time interval [s], but not more than 5 s
- ρ_a is the unit mass of steel, ρ_a = 7850 kg/m^3

For cross-sections having a convex shape (such as a rectangular or circular hollow section) and exposed to fire on all directions, the shadow effect does not play any role and consequently the value k_{sh} = 1,0 can be chosen [9,10,11].

The design value of the net heat flux is calculated as follows [6,7]:

$$\dot{h}_{net.d} = \dot{h}_{net.c} + \dot{h}_{net.r} \qquad (6.3)$$

where
- $\dot{h}_{net.c}$ is the net heat flux due to convection [W/m^2]
- $\dot{h}_{net.r}$ is the net heat flux due to radiation [W/m^2]

The net heat flux due to convection [6,7]:

$$\dot{h}_{net.c} = \alpha_c(\theta_g - \theta_m) \tag{6.4}$$

where α_c is the coefficient of heat transfer by convection [W / (m^{2}K)] ,
α_c = 25 W / (m^{2}K) in standard fire
θ_g is the gas temperature near the fire exposed member [ºC]
θ_m is the surface temperature of the member [ºC]

The net heat flux due to radiation [6,7]:

$$\dot{h}_{net.r} = \Phi \cdot \varepsilon_m \cdot \varepsilon_f \cdot \sigma \cdot [(\theta_r + 273)^4 - (\theta_m + 273)^4] \tag{6.5}$$

where Φ is the configuration factor, Φ = 1,0
ε_m is the surface emissivity of the member,
for carbon steel ε_m = 0,7 [9,10,11]
ε_f is the emissivity of the fire, ε_f = 1,0 [9,10,11]
σ is the Stefan-Boltzmann constant, σ = 5,67·10^{-8} W/(m^{2}K^4)
θ_r is the effective radiation temperature of the fire environment [ºC]
θ_m is the surface temperature of the member [ºC]

For the specific heat of steel, the value c_a = 600 J/(kgK) may be taken, or it may be calculated more accurately as follows [9,10,11]:

$$c_a = 425 + 7,73 \cdot 10^{-1} \cdot \theta_a - 1,69 \cdot 10^{-3} \cdot \theta_a^2 + 2,22 \cdot 10^{-6} \cdot \theta_a^3 \tag{6.6a}$$

$$for\ 20°C \le \theta_a < 600°C$$

$$c_a = 666 + \frac{13002}{738 - \theta_a} \qquad for\ 600\,°C \le \theta_a < 735\,°C \tag{6.6b}$$

$$c_a = 545 + \frac{17820}{\theta_a - 731} \qquad for\ 735\,°C \le \theta_a < 900\,°C \tag{6.6c}$$

$$c_a = 650\ J/(kgK) \qquad for\ 900°C \le \theta_a \le 1200°C \tag{6.6d}$$

where θ_a is the steel temperature [ºC].

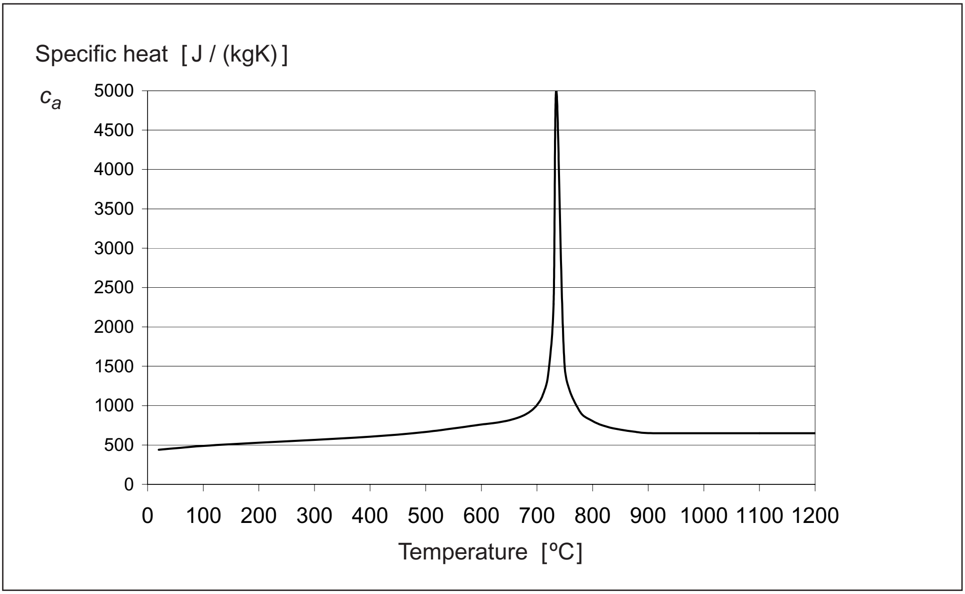


Figure 6.8 Specific heat of carbon steel (the peak at 750 ºC is due to phase transformations in carbon steel)

6.3.2 Indoor steel structure insulated by fire protection material

The properties and performance of fire protection materials used in design shall be assessed using the test procedures given in prestandards ENV 13381-1, ENV 13381-2 or ENV 13381-4. These standards include a requirement that the fire protection material should remain coherent and cohesive to the member they are protecting throughout the relevant fire exposure [9,10,11]:

The temperature increase in an insulated steel member is calculated as follows [9,10,11]:

$$\Delta\theta_{a.t} = \frac{\lambda_p / d_p}{c_a \rho_a} \cdot \frac{A_p}{V} \cdot \frac{\theta_{g.t} - \theta_{a.t}}{1 + \phi/3} \cdot \Delta t - (e^{\phi/10} - 1) \cdot \Delta\theta_{g.t} \qquad \text{but } \Delta\theta_{a.t} \geq 0 \text{ if } \Delta\theta_{g.t} > 0 \tag{6.7}$$

$$\phi = \frac{c_p \rho_p}{c_a \rho_a} \cdot d_p \cdot \frac{A_p}{V} \tag{6.8}$$

where A_p/V is the section factor for steel member insulated by fire protection material
A_p is the appropriate surface area of fire protection material per unit length of the member [m^2/m]
V is the volume of the member per unit length [m^3/m]
c_a is the temperature dependent specific heat of steel [J / (kgK)]
c_p is the temperature independent specific heat of the fire protection material [J / (kgK)]

d_p is the thickness of the fire protection material [m]
Δt is the time interval [s], but not more than 30 s
$\theta_{a.t}$ is the steel temperature [ºC] at time t
$\theta_{g.t}$ is the ambient gas temperature [ºC] at time t
$\Delta\theta_{g.t}$ is the increase of the ambient gas temperature [K] during the time interval Δt
λ_p is the thermal conductivity of the fire protection system [W / (mK)]
ρ_a is the unit mass of steel, ρ_a = 7850 kg/m^3
ρ_p is the unit mass of the fire protection material [kg/m^3]

The area of the fire protection material A_p should generally be taken as the area of its inner surface area, but for hollow encasement protections with a clearance around the steel member, the same value may be used as for hollow encasement protection without a clearance, see Table 6.2 [9,10,11].

The moisture in fire protection material delays the temperature rise of steel until the moisture has evaporated. During this delay time, the temperature of steel can be supposed to stay at 100 ºC, because the moisture in the fire protection material gets out through the surface opposite to the fire [2,3]. The delay time is determined using the procedure given in prestandard ENV 13381-4 [9,10,11].

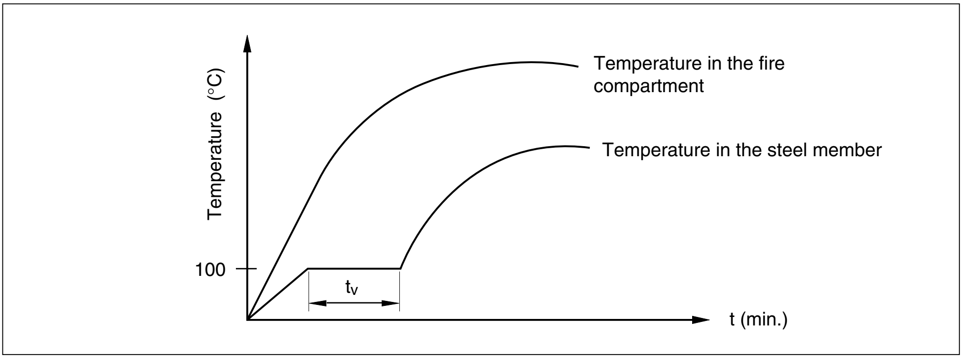


Figure 6.9 Schematic picture of the influence of the moisture content in the fire protection material on the steel temperature development

6.3.3 Joints

The temperature of a joint may be assessed using the local section factor A/V of the parts forming that joint. As a simplification a uniformly distributed temperature may be supposed within the joint. The temperature may be calculated using the maximum A/V value of the connected steel members in the vicinity of the joint [9,10,11].

For beam-to-column and beam-to-beam joints, where the beams are supporting any type of concrete floor, the temperature for the joint may be obtained using the temperature determined for the bottom flange at mid span [9,10,11].

The temperature of the joint components may be calculated as follows [9,10,11]:

- when the depth of the beam is $D \leq 400$ mm:

$$\theta_h = 0{,}88\,\theta_0\,[1 - 0{,}3\,(h/D)] \tag{6.9a}$$

- when the depth of the beam is $D > 400$ mm:

$$\theta_h = 0{,}88\,\theta_0 \qquad \text{for } h \leq D/2 \tag{6.9b}$$

$$\theta_h = 0{,}88\,\theta_0\,[1 + 0{,}2\,(1 - 2h/D)] \qquad \text{for } h > D/2 \tag{6.9c}$$

where θ_h is the temperature at the height h [mm] of the steel beam, see Figure 6.10
θ_0 is the bottom flange temperature of the steel beam remote from the joint
h is the height of the component being considered above the bottom flange [mm]
D is the depth of the beam [mm]

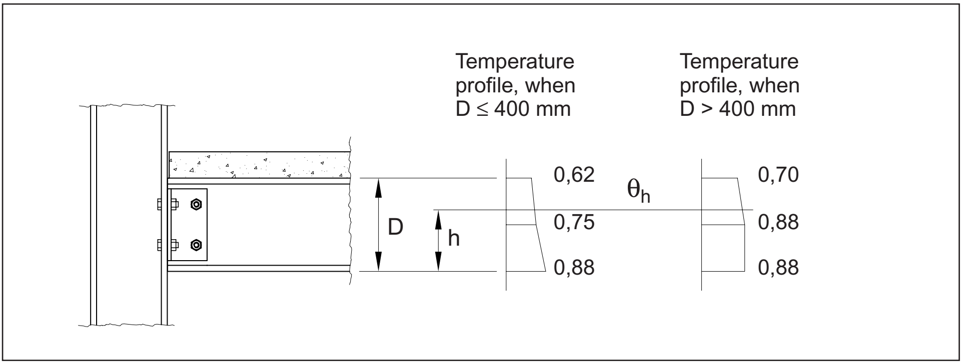


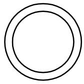
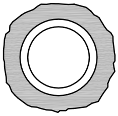
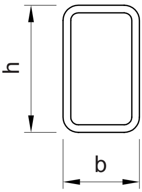
Figure 6.10 Distribution of temperature as a function of height in a composite joint [9,10,11]

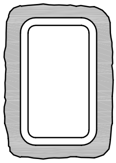
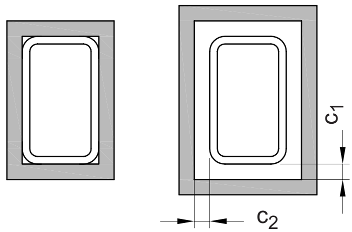
6.3.4 Section factor of a steel member

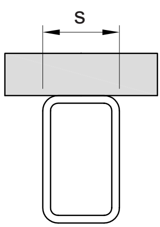
The section factor of an unprotected steel member is defined as the ratio of the exposed surface area of the member to the volume of the member. In the case of a fire-protected steel member, the area of the protection-surface opposite to the fire is used (i.e. inner area of the fire protection). If the section factor is constant in different cross-sections of the member, it is possible to use the perimeter instead of the surface area and the cross-section area instead of the volume.

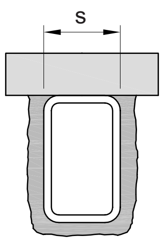
The calculation of the section factor for unprotected and for fire-protected hollow sections is presented in Table 6.2 applying the guidelines of Part EN 1993-1-2 of Eurocode.

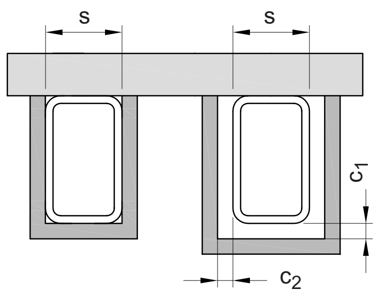
Table 6.2 Section factor for structural hollow sections in fire design

<table>
<tr><th>Unprotected</th><th>Fire protective paint or spray coating</th><th>Hollow encasement protection</th></tr>
<tr><td colspan="3">Cross-section exposed to fire on all sides</td></tr>
<tr><td>
$$\frac{A_m}{V} = \frac{A_u}{A \cdot 1m} \approx \frac{1}{t}$$</td><td>
$$\frac{A_p}{V} = \frac{A_u}{A \cdot 1m} \approx \frac{1}{t}$$</td><td></td></tr>
<tr><td>

$$\frac{A_m}{V} = \frac{A_u}{A \cdot 1m} \approx \frac{1}{t}$$</td><td>
$$\frac{A_p}{V} = \frac{A_u}{A \cdot 1m} \approx \frac{1}{t}$$</td><td>

$$\frac{A_p}{V} = \frac{2(b+h) \cdot 1m}{A \cdot 1m}$$
$c_1 \leq h/4$, $c_2 \leq h/4$ a)</td></tr>
<tr><td colspan="3">Cross-section exposed to fire on three sides</td></tr>
<tr><td>

$$\frac{A_m}{V} = \frac{A_u - (s \cdot 1m)}{A \cdot 1m}$$</td><td>

$$\frac{A_p}{V} = \frac{A_u - (s \cdot 1m)}{A \cdot 1m}$$</td><td>

$$\frac{A_p}{V} = \frac{[2(b+h) - s] \cdot 1m}{A \cdot 1m}$$
$c_1 \leq h/4$, $c_2 \leq h/4$ a)</td></tr>
<tr><td colspan="3">A_u is the external surface area of the hollow section [m^2/m] (Annex 11.1)
A is the cross-section area of the hollow section [m^2] (Annex 11.1)</td></tr>
<tr><td colspan="3">a) In case of a hollow encasement protection with a clearance around the steel member, the same section factor value may be used as for a hollow encasement protection without a clearance, provided the clearance (c_1 and c_2) fulfills the conditions presented.</td></tr>
</table>

6.4 Critical temperature of a structural hollow section

The critical temperature of steel is defined as the temperature where the yield strength of the steel has decreased to the stress level caused by the loading [2,3].

If the temperature distribution in the cross-section is uniform and stability of the member need not to be taken into account, the fire design of the member can be carried out basing on temperature assessment instead of resistance assessment. For temperature based assessment, the design condition of the member is [9,10,11]:

$$\theta_a \leq \theta_{cr} \tag{6.10}$$

where θ_a is the temperature of steel at time t to be considered
θ_{cr} is the critical temperature of steel

The critical temperature depends on the utilisation ratio of the cross-section, and it can be calculated as follows [9,10,11]:

$$\theta_{a.cr} = 39,19\,ln\left[\frac{1}{0,9674 \cdot \mu_0^{3,833}} - 1\right] + 482 \qquad for\ \ \mu_0 \geq 0,013 \tag{6.11}$$

where μ_0 is the utilisation ratio, which is calculated as follows [9,10,11]:

$$\mu_0 = \frac{E_{fi.d}}{R_{fi.d.0}} \tag{6.12}$$

where $E_{fi.d}$ is the design value of force or moment in the fire situation
$R_{fi.d.0}$ is the design value of the corresponding resistance of the steel member in the fire situation at time $t = 0$

On Class 4 members other than members in tension, the resistance in fire situation is sufficient, if at the time to be considered the temperature θ_a in all points of the cross-section is not more than θ_{crit}, the value of which can be chosen in the National Annex. The recommended value in Eurocode is θ_{crit} = 350 ºC [9,10,11].

Finnish National Annex to standard EN 1993-1-2 [12]:
The value θ_{crit} = 450 ºC *may be used.*

Table 6.3 Influence of the utilisation ratio of the cross-section to the critical temperature of steel [9,10,11]

μ_0	θ_{cr}	μ_0	θ_{cr}	μ_0	θ_{cr}	μ_0	θ_{cr}
0,10	829	0,34	645	0,58	560	0,82	490
0,12	802	0,36	636	0,60	554	0,84	483
0,14	779	0,38	628	0,62	549	0,86	475
0,16	759	0,40	620	0,64	543	0,88	467
0,18	741	0,42	612	0,66	537	0,90	458
0,20	725	0,44	605	0,68	531	0,92	448
0,22	711	0,46	598	0,70	526	0,94	436
0,24	698	0,48	591	0,72	520	0,96	421
0,26	685	0,50	585	0,74	514	0,98	398
0,28	674	0,52	578	0,76	508	1,00	349
0,30	664	0,54	572	0,78	502		
0,32	654	0,56	566	0,80	496		

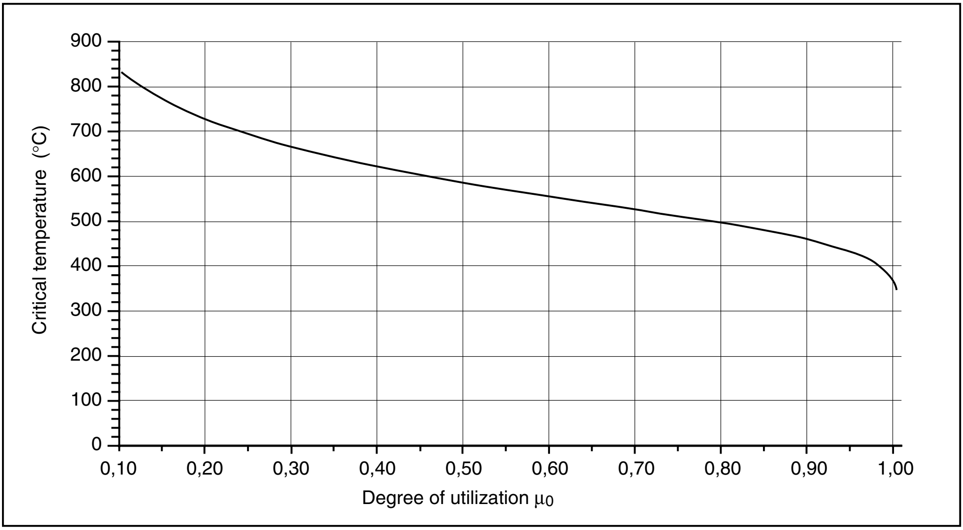


Figure 6.11 The critical temperature of steel as a function of utilisation ratio of the cross-section

6.5 Loads in fire situation

The fire situation is considered to be as one of the accidental design situations, for which the partial safety factors for loads are normally taken as $\gamma_F = 1{,}0$, and for which the load combination to be used is determined according to Part EN 1990 of Eurocode using the following expression [4,4a]:

$$\sum_{j \geq 1} G_{k.j} \text{ "+" } A_d \text{ "+" } (\psi_{1.1} \text{ or } \psi_{2.1}) Q_{k.1} \text{ "+" } \sum_{i > 1} \psi_{2.i} Q_{k.i} \qquad (6.13)$$

where
- "+" means "to be combined with" (i.e. simultaneous action of the loads)
- j is the index for permanent load
- i is the index for the variable load
- $G_{k.j}$ is the characteristic value of permanent load
- A_d is the design value of accidental load (if defined)
- $Q_{k.1}$ is the characteristic value of the leading variable load
- $Q_{k.i}$ is the characteristic value of other variable load
- $\psi_{1.1}$ is the combination factor ψ_1 for the leading variable load (Table 2.4)
- $\psi_{2.1}$ is the combination factor ψ_2 for the leading variable load (Table 2.4)
- $\psi_{2.i}$ is the combination factor ψ_2 for other variable load (Table 2.4)

The choice between the combination factors $\psi_{1.1}$ and $\psi_{2.1}$ is made in the National Annex.

Finnish National Annex to standard EN 1990 [5]:

The combination factor $\psi_{1.1}$ is used. However, when the leading variable load is other than snow, ice or wind load, the combination factor $\psi_{2.1}$ is used.

The structural analysis for the fire situation may be carried out in one of the following ways [9,10,11]:

- global analysis of the structural system
- analysis of part of the structure
- analysis of a structural member

When verifying the requirements in standard fire, a member analysis is sufficient [9,10,11].

Instead of applying a unique load combination according to expression (6.13), a structural member can be designed for fire situation by using a simple method, where the effects of loads at normal temperature are reduced. Consequently the effect of thermal expansions and deformations will be omitted. The value of force and moment in fire situation is obtained in a simple way from the following formula [9,10,11]:

$$E_{fi.d} = \eta_{fi} E_d \qquad (6.14)$$

where
- E_d is the design value of the force or moment at normal temperature
- η_{fi} is the reduction factor for fire situation

Depending on the normal temperature load combination to be chosen in Part EN 1990 of Eurocode, the reduction factor η_{fi} is calculated as follows [9,10,11]:

- when using at normal temperature the load combination according to expression (2.3) in Chapter 2, the reduction factor η_{fi} is calculated from the formula:

$$\eta_{fi} = \frac{G_k + \psi_{fi} Q_{k.1}}{\gamma_G G_k + \gamma_{Q.1} Q_{k.1}} \tag{6.15}$$

- when using at normal temperature the load combination according to expressions (2.4a) and (2.4b) of Chapter 2, the reduction factor η_{fi} is smaller of the following:

$$\eta_{fi} = \frac{G_k + \psi_{fi} Q_{k.1}}{\gamma_G G_k + \gamma_{Q.1} \psi_{0.1} Q_{k.1}} \tag{6.16a}$$

$$\eta_{fi} = \frac{G_k + \psi_{fi} Q_{k.1}}{\xi \gamma_G G_k + \gamma_{Q.1} Q_{k.1}} \tag{6.16b}$$

where G_k is the characteristic value of permanent load
$Q_{k.1}$ is the characteristic value of the leading variable load
γ_G is the partial safety factor for permanent loads (Table 2.1)
$\gamma_{Q.1}$ is the partial safety factor for the leading variable load (Table 2.1)
ψ_{fi} is the combination factor $\psi_{1.1}$ or $\psi_{2.1}$ for loads (Table 2.4), which is chosen in the National Annex
ξ is the reduction factor for unfavourable permanent load (Table 2.1)

Finnish National Annex to standard EN 1990 [5]:

- Expressions (6.16a) and (6.16b) are used, and in expression (6.16a) only the permanent loads are taken into account.
- The combination factor $\psi_{1.1}$ is used. However, when the leading variable load is other than snow, ice or wind load, the combination factor $\psi_{2.1}$ is used.

In Figure 6.12 the reduction factor η_{fi} is presented according to the expression (6.15) using the recommended values in Eurocode γ_G = 1,35 and $\gamma_{Q.1}$ = 1,5. **The national values must be checked from the National Annex of the relevant country.**

In Figure 6.13 the reduction factor η_{fi} is presented on the basis of the expressions (6.16a) and (6.16b) using the values in the Finnish National Annex.

For the reduction factor η_{fi} the value η_{fi} = 0,65 may be used as a simplification, except for the loads in load category E in Table 2.4 (areas susceptible to accumulation of goods, including access areas), for which the value η_{fi} = 0,7 is used [9,10,11].

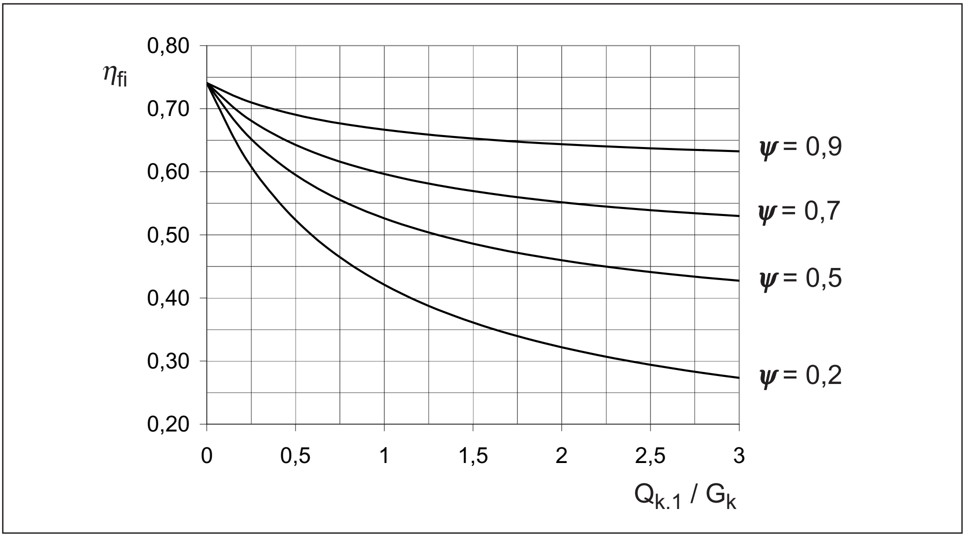


Figure 6.12 Load reduction factor for fire situation as a function of the ratio between variable load and permanent load, when using the general load combination according to Eurocode and the values of partial safety factors given therein [9,10,11]

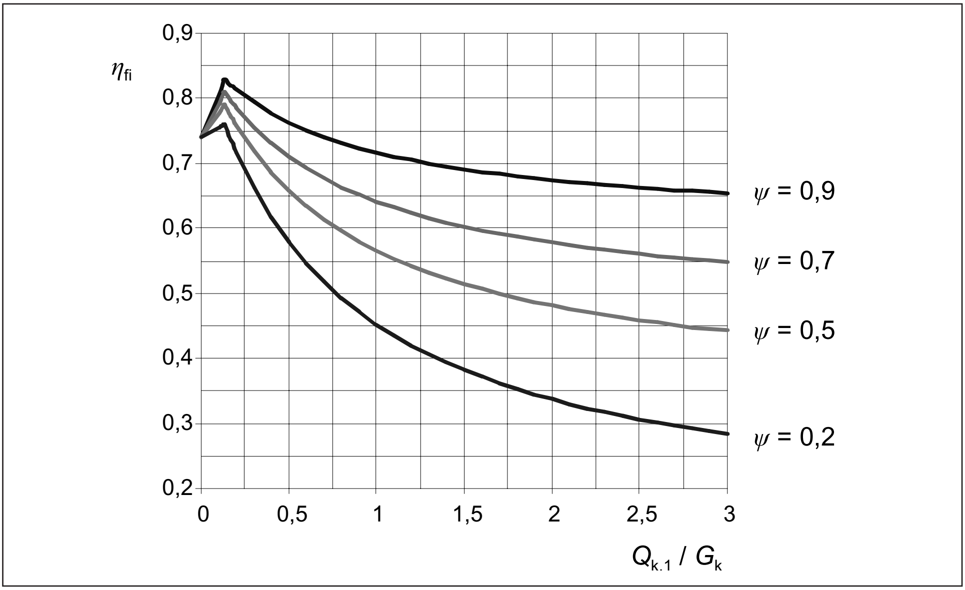


Figure 6.13 Load reduction factor for fire situation as a function of the ratio between variable load and permanent load, when using the load combination according to the Finnish National Annex and the values of partial safety factors given therein [12]

6.6 Resistance of a structural hollow section in fire situation

The design condition in fire situation is [9,10,11]:

$$E_{fi.d} \leq R_{fi.d.t} \qquad (6.17)$$

where $E_{fi.d}$ is the design value of force or moment for the fire design situation
$R_{fi.d.t}$ is the corresponding design resistance of the steel member for the fire design situation at time t

The partial safety factor for resistance in the fire situation is chosen in the National Annex. The recommended value in Eurocode is $\gamma_{M.fi}$ = 1,0 [9,10,11].

Finnish National Annex to standard EN 1993-1-2 [12]:
The recommended value of Eurocode $\gamma_{M.fi}$ = 1,0 *is used.*

EN 1993-1-2 gives different provisions to calculate the resistance in fire situation when the temperature distribution in the cross-section is:

a) uniformly distributed, or
b) non-uniformly distributed

Because there are, however, no instructions presented in Eurocode how to determine the temperature distribution (only a reference to so-called advanced calculation models), the provisions presented in the following apply the assessment of a member and calculation of resistance when using the simple calculation model presented in Eurocode, whereby the temperature may usually be supposed as uniformly distributed in the cross-section, unless specificly otherwise stated [9,10,11].

6.6.1 Cross-section classification in fire situation

The cross-section classification is carried out for the fire situation according to the same principles as for normal temperature, but using a reduced value of factor ε.

Factor ε represents the relative deformation capability of the cross-section, that is used in various contexts. At normal temperature its definition is according to Table 2.7:

$$\varepsilon = \sqrt{\frac{235}{f_y}} \qquad (6.18)$$

Expression (6.18) implicitly supposes that the elastic modulus of steel is constant. In fire situation the elastic modulus and strength, however, do vary according to Table 6.1, why expression (6.18) needs to be corrected as follows [13]:

$$\varepsilon_{fi} = \sqrt{\frac{k_{E.\theta}}{k_{y.\theta}}} \cdot \varepsilon = \sqrt{\frac{E_{fi}}{E} \cdot \frac{f_y}{f_{y.fi}}} \cdot \varepsilon \qquad (6.19)$$

Because the elastic modulus and strength change with different rate when the temperature is rising, factor ε_{fi} varies in the fire situation according to Figure 6.14. Consequently it might happen, that the cross-section classification would need to be changed in the calculations in the middle of the fire design, when the temperature changes. **Therefore, for simplicity, it has been defined in EN 1993-1-2 that the cross-section Class shall be determined for fire situation by reducing factor ε with a fixed coefficient as follows [9,10,11]:**

$$\varepsilon = 0,85\sqrt{235/f_y} \tag{6.20}$$

where f_y is the yield strength of steel at normal temperature.

For the reduced value of ε there is, however, in Eurocode no specific designation ε_{fi} assigned corresponding to the fire situation. The aforementioned notation praxis of Eurocode can be interpreted so, that the reduced ε value from expression (6.20) is intended to substitute factor ε that is applied in various normal temperature formulae, when applying those formulae in fire design. This way the calculation work is simplified, and the ‘wandering’ of other calculatory values (due to variation of the reduction factor presented in Figure 6.14, when the temperature of steel is rising) will be avoided.

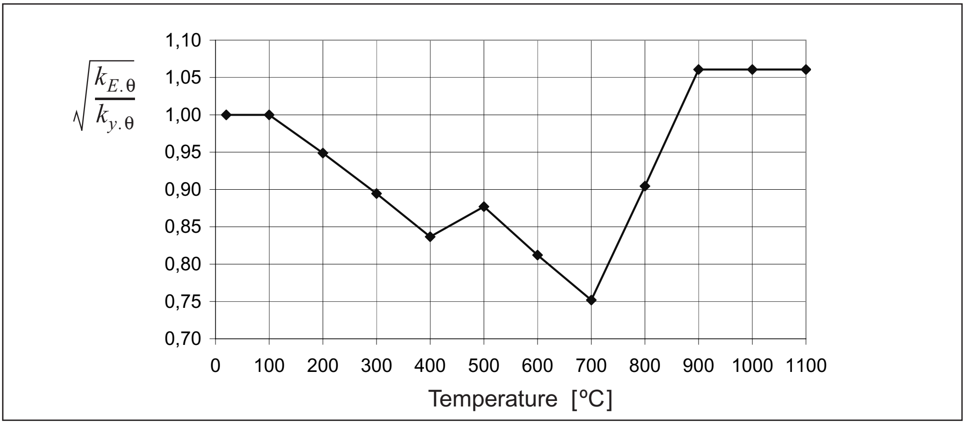


Figure 6.14 Carbon steel. Reduction coefficient for factor ε as a function of temperature [13]

6.6.2 Resistance, when cross-section is classified in fire situation into Classes 1-3

6.6.2.1 Tension resistance

Tension resistance of a member which is classified in fire situation into Classes 1-3 is calculated from the following formula, when the temperature θ_a of the cross-section is uniformly distributed [9,10,11]:

$$N_{fi.t.Rd} = k_{y.\theta} N_{Rd} \frac{\gamma_{M0}}{\gamma_{M.fi}} \tag{6.21}$$

where $k_{y.\theta}$ is the reduction factor for the yield strength of steel at temperature θ_a, which is reached at time t (Table 6.1)
N_{Rd} is the plastic resistance of the cross-section $N_{pl.Rd}$ at normal temperature
γ_{M0} is the partial safety factor for resistance at normal temperature (Table 2.5)
$\gamma_{M.fi}$ is the partial safety factor for resistance in fire design (Table 2.5)

6.6.2.2 Compression resistance

Compression resistance of a member which is classified in fire situation into Classes 1-3 is calculated from the following formula, when the temperature θ_a of the cross-section is uniformly distributed [9,10,11]:

$$N_{b.fi.t.Rd} = \chi_{fi} A\, k_{y.\theta} f_y / \gamma_{M.fi} \tag{6.22}$$

where χ_{fi} is the reduction factor for flexural buckling in fire design
A is the cross-section area
$k_{y.\theta}$ is the reduction factor for the yield strength of steel at temperature θ_a, which is reached at time t (Table 6.1)
f_y is the nominal yield strength of steel at normal temperature
$\gamma_{M.fi}$ is the partial safety factor for resistance in fire design (Table 2.5)

For the value of reduction factor χ_{fi} the lesser of the values $\chi_{y.fi}$ and $\chi_{z.fi}$ is chosen, which are calculated as follows [9,10,11]:

$$\chi_{fi} = \frac{1}{\varphi_\theta + \sqrt{\varphi_\theta^2 - \bar{\lambda}_\theta^2}} \tag{6.23}$$

$$\varphi_0 = \frac{1}{2}[1 + \alpha\bar{\lambda}_\theta + \bar{\lambda}_\theta^2] \tag{6.24}$$

$$\alpha = 0,65\sqrt{235/f_y} \tag{6.25}$$

$$\bar{\lambda}_\theta = \bar{\lambda} \cdot \sqrt{k_{y.\theta}/k_{E.\theta}} \tag{6.26}$$

where $\bar{\lambda}_\theta$ is the non-dimensional slenderness of the member at temperature θ_a
$\bar{\lambda}$ is the non-dimensional slenderness of the member at normal temperature
$k_{y.\theta}$ is the reduction factor for the yield strength of steel at temperature θ_a, which is reached at time t (Table 6.1)
$k_{E.\theta}$ is the reduction factor for the elastic modulus of steel at temperature θ_a, which is reached at time t (Table 6.1)
f_y is the nominal yield strength of steel at normal temperature
$\gamma_{M.fi}$ is the partial safety factor for resistance in fire design (Table 2.5)

The buckling length l_{fi} of a column for the fire design situation is generally determined as for normal temperature design. However, in a braced frame the buckling length l_{fi} of a column length may be determined by considering it as fixed in direction at continuous or semi-continuous joints to the column lengths in the fire compartments above and below, provided that the fire resistance of the building components that separate these fire compartments is not less than the fire resistance of the column [9,10,11].

In the case of a braced frame in which each storey comprises a separate fire compartment with sufficient fire resistance, in an intermediate storey the buckling length l_{fi} of a continuous column may be taken as l_{fi} = 0,5L and in the top storey the buckling length may be taken as l_{fi} = 0,7L, where L is the system length in the relevant storey, see Figure 6.15 [9,10,11].

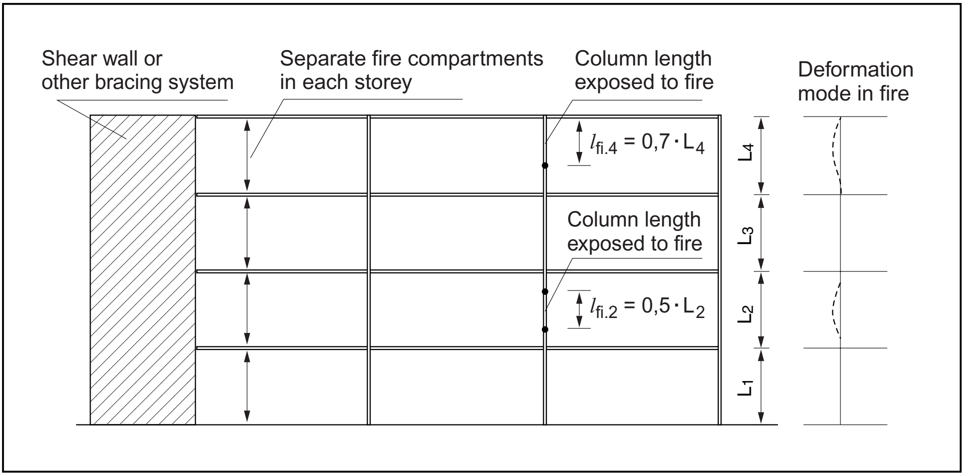


Figure 6.15 Buckling lengths of columns in braced frames [9,10,11]

6.6.2.3 Bending resistance

Bending resistance of a member which is classified in fire situation into Classes 1-3 is calculated as follows [9,10,11]:

$$M_{fi.t.Rd} = k_{y.\theta}\frac{M_{Rd}}{\kappa_1\kappa_2}\cdot\frac{\gamma_{M0}}{\gamma_{M.fi}} \quad \text{but} \quad M_{fi.t.Rd} \le M_{Rd} \tag{6.27}$$

where $k_{y.\theta}$ is the reduction factor for the yield strength of steel at temperature θ_a, which is reached at time t (Table 6.1)

M_{Rd} is the plastic bending resistance $M_{pl.Rd}$ of the cross-section at normal temperature (or reduced bending resistance, when effects of shear are taken into account when necessary), when having a cross-section that is classified in fire situation into Class 1 or 2

M_{Rd} is the elastic bending resistance $M_{el.Rd}$ of the cross-section at normal temperature (or reduced bending resistance, when effects of shear are taken into account when necessary), when having a cross-section that is classified in fire situation into Class 3

κ_1 is an adaptation factor for non-uniform temperature distribution across the cross-section:

$\kappa_1 = 1,0$, for a beam exposed to fire on four sides

$\kappa_1 = 0,7$, for an unprotected beam exposed to fire on three sides, and with a composite or concrete slab on the fourth side

$\kappa_1 = 0,85$, for a fire-protected beam exposed to fire on three sides, and with a composite or concrete slab on the fourth side

κ_2 is an adaptation factor for non-uniform temperature distribution along the beam:

$\kappa_2 = 0,85$, at the supports of statically indeterminate beam

$\kappa_2 = 1,0$, in all other cases

γ_{M0} is the partial safety factor for resistance at normal temperature (Table 2.5)

$\gamma_{M.fi}$ is the partial safety factor for resistance in fire design (Table 2.5)

The lateral-torsional buckling resistance of a laterally unrestraint member is calculated as follows [9,10,11]:

$$M_{b.fi.t.Rd} = \chi_{LT.fi} W_y k_{y.\theta} f_y / \gamma_{M.fi} \tag{6.28}$$

where $\chi_{LT.fi}$ is the reduction factor for lateral-torsional buckling in fire design

W_y is the plastic section modulus $W_{pl.y}$ of the cross-section at normal temperature, when having a cross-section that is classified in fire situation into Class 1 or 2

W_y is the elastic section modulus $W_{el.y}$ of the cross-section at normal temperature, when having a cross-section that is classified in fire situation into Class 3

$k_{y.\theta}$ is the reduction factor for the yield strength of steel at temperature θ_a, which is reached at time t (Table 6.1)

f_y is the nominal yield strength of steel at normal temperature

$\gamma_{M.fi}$ is the partial safety factor for resistance in fire design (Table 2.5)

The reduction factor for lateral-torsional buckling $\chi_{LT.fi}$ is calculated as follows [9,10,11]:

$$\chi_{LT.fi} = \frac{1}{\phi_{LT.\theta} + \sqrt{\phi_{LT.\theta}^2 - \bar{\lambda}_{LT.\theta}^2}} \tag{6.29}$$

$$\phi_{LT.\theta} = \frac{1}{2}[1 + \alpha\bar{\lambda}_{LT.\theta} + \bar{\lambda}_{LT.\theta}^2] \tag{6.30}$$

$$\alpha = 0,65\sqrt{235/f_y} \tag{6.31}$$

$$\bar{\lambda}_{LT.\theta} = \bar{\lambda}_{LT} \cdot \sqrt{k_{y.\theta}/k_{E.\theta}} \tag{6.32}$$

where $\bar{\lambda}_{LT.\theta}$ is the non-dimensional slenderness of the member for lateral-torsional buckling, when the temperature of the member is θ_a

$\bar{\lambda}_{LT}$ is the non-dimensional slenderness of the member for lateral-torsional buckling at normal temperature

$k_{y.\theta}$ is the reduction factor for the yield strength of steel at temperature θ_a, which is reached at time t (Table 6.1)

$k_{E.\theta}$ is the reduction factor for the elastic modulus of steel at temperature θ_a, which is reached at time t (Table 6.1)

f_y is the nominal yield strength of steel at normal temperature

$\gamma_{M.fi}$ is the partial safety factor for resistance in fire design (Table 2.5)

6.6.2.4 Shear resistance

Shear resistance of a member which is classified in fire situation into Classes 1 - 3 is calculated as follows [9,10,11]:

$$V_{fi.t.Rd} = k_{y.\theta.web} V_{Rd} \frac{\gamma_{M0}}{\gamma_{M.fi}} \qquad (6.33)$$

where $k_{y.\theta.web}$ is the reduction factor for the yield strength of steel, when the temperature of the web is θ_{web}, which is reached at time t (Table 6.1)

V_{Rd} is the shear resistance of the cross-section at normal temperature (Note: In Finland however, the appropriate value of the η factor in respect to fire situation shall be applied. For structural hollow sections η = 1,0 at normal temperature and in fire design.)

γ_{M0} is the partial safety factor for resistance at normal temperature (Table 2.5)

6.6.2.5 Combined effect of bending moment and normal force when buckling or lateral-torsional buckling can occur

The formulae presented in fire-part EN 1993-1-2 of Eurocode for a member which is subject to combined effect of (M+N) resemble those presented in ENV-Eurocode, and they differ from the corresponding formulae presented for the normal temperature in Part EN 1993-1-1 of EN-Eurocode. The reason for this is, that when preparing EN 1993-1-2 for fire design, there was not enough time to verify the applicability of the (M+N) interaction formulae presented in EN 1993-1-1 [13].

The resistance of a member which is classified in fire situation into Classes 1-3 is checked as follows [9,10,11]:

$$\frac{N_{fi.Ed}}{N_{b.fi.t.Rd}} + \frac{k_y M_{y.fi.Ed}}{M_{y.fi.t.Rd}} + \frac{k_z M_{z.fi.Ed}}{M_{z.fi.t.Rd}} \le 1,0 \qquad (6.34)$$

$$\frac{N_{fi.Ed}}{N_{z.b.fi.t.Rd}} + \frac{k_{LT} M_{y.fi.Ed}}{M_{b.fi.t.Rd}} + \frac{k_z M_{z.fi.Ed}}{M_{z.fi.t.Rd}} \le 1,0 \qquad (6.35)$$

where $N_{fi.Ed}$ is the compressive normal force acting in the cross-section in fire situation

$M_{y.fi.Ed}$ is the bending moment acting in the cross-section in fire situation about y-axis

$M_{z.fi.Ed}$ is the bending moment acting in the cross-section in fire situation about z-axis

$N_{b.fi.t.Rd}$ is the buckling resistance of the member in fire situation according to clause 6.6.2.2

$N_{z.b.fi.t.Rd}$ is the buckling resistance of the member in fire situation about z-axis

$M_{y.fi.t.Rd}$ is the bending resistance of the cross-section in fire situation about y-axis according to clause 6.6.2.3

$M_{z.fi.t.Rd}$ is the bending resistance of the cross-section in fire situation about z-axis

$M_{b.fi.t.Rd}$ is the lateral-torsional buckling resistance of the member in fire situation according to clause 6.6.2.3

The factors k_y, k_z and k_{LT} needed in design conditions (6.34) and (6.35) are calculated as follows [9,10,11]:

$$k_{LT} = 1 - \frac{\mu_{LT} N_{fi.Ed}}{N_{z.b.fi.t.Rd}} \le 1 \qquad (6.36)$$

$$\mu_{LT} = 0,15\,\bar{\lambda}_{z.\theta}\,\beta_{M.LT} - 0,15 \le 0,9 \qquad (6.37)$$

and:

$$k_y = 1 - \frac{\mu_y N_{fi.Ed}}{N_{y.b.fi.t.Rd}} \le 3 \qquad (6.38)$$

$$\mu_y = (2\beta_{M.y} - 5)\bar{\lambda}_{y.\theta} + 0,44\beta_{M.y} + 0,29 \le 1,1 \qquad with \quad \bar{\lambda}_{y.20°C} \le 1,1 \qquad (6.39)$$

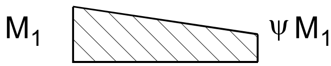
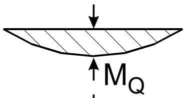
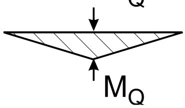
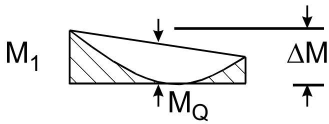
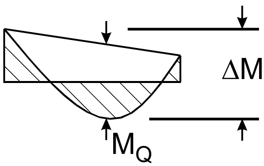
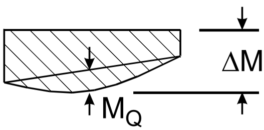
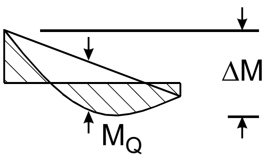
and:

$$k_z = 1 - \frac{\mu_z N_{fi.Ed}}{N_{z.b.fi.t.Rd}} \le 3 \qquad (6.40)$$

$$\mu_z = (1,2\beta_{M.z} - 3)\bar{\lambda}_{z.\theta} + 0,71\beta_{M.z} - 0,29 \le 0,8 \qquad (6.41)$$

The equivalent uniform moment factors β_M needed in expressions (6.36) - (6.41) are obtained from Table 6.4.

Table 6.4 Equivalent uniform moment factors [9,10,11]

Moment diagram	Equivalent uniform moment factor β_M
End moments  $-1 \leq \psi \leq 1$	$\beta_{M.\psi} = 1,8 - 0,7\psi$
Moments due to in-plane lateral loads   	$\beta_{M.Q} = 1,3$ $\beta_{M.Q} = 1,4$
Moments due to in-plane lateral loads plus end moments     	$\beta_M = \beta_{M.\psi} + \frac{M_Q}{\Delta M}(\beta_{M.Q} - \beta_{M.\psi})$ $M_Q = \lvert max\ M \rvert$ *due to lateral load only* $\Delta M = \lvert max\ M \rvert$ *for moment diagram without change of sign* $\Delta M = \lvert max\ M \rvert + \lvert min\ M \rvert$ *for moment diagram with change of sign*

6.6.3 Resistance, when cross-section is classified in fire situation into Class 4

The provisions of EN 1993-1-2 for Class 4 apply as such for resistances of square and rectangular hollow sections. For circular hollow sections the Eurocode's instructions cannot be applied as such (it is not possible to determine an effective cross-section for Class 4 circular hollow sections). Therefore, in respect to Class 4 circular hollow sections, the provisions of Eurocode need to be adapted according to the following.

In fire situation the members in Class 4 are designed applying the instructions presented for Class 3 in clause 6.6.2, but with the following changes:

- **square and rectangular hollow sections:**
  - cross-section area is substituted by an effective area (in case of member in tension, the whole area is effective)
  - section modulus of the cross-section is substituted by the effective section modulus
  - in fire design, reduced strength $k_{p0,2.\theta} \cdot f_y$ is used as steel strength (Table 6.1) where f_y is the steel strength at normal temperature 20 °C

The effective cross-section area and the effective section modulus are calculated according to Chapter 2 by using strength and modulus of elasticity at normal temperature.

- **circular hollow sections:**
  - resistances at normal temperature are calculated according to the provisions in Chapter 2 for Class 4 circular hollow sections
  - in fire design, reduced strength $k_{p0,2.\theta} \cdot f_y$ is used as steel strength (Table 6.1) where f_y is the steel strength at normal temperature 20 °C
  - in fire design, the bending resistance and shear resistance are calculated using formulae (6.27) and (6.33), wherein partial safety factor γ_{M0} shall be substituted by the partial safety factor γ_{M1} given in EN 1993-1-6 for Class 4 circular hollow sections (Table 2.5). The design resistance in fire situation, however, is not allowed to exceed the design resistance at normal temperature.
  - the formulae (6.28) and (6.35) are neglected (circular hollow section is not subject to lateral-torsional buckling)

6.7 Resistance of joints in fire situation

The fire resistance of a bolted or welded joint may be assumed to be sufficient provided that the following conditions are satisfied [9,10,11]:

1) The thermal resistance $(d_f / \lambda_f)_c$ of the joint's fire protection should be equal or greater than the minimum value of thermal resistance $(d_f / \lambda_f)_m$ of fire protection applied to any of the jointed members,

   where d_f is the thickness of the fire protection material (d_f = 0 for unprotected members)

   λ_f is the effective thermal conductivity of the fire protection material.

2) The utilisation ratio of the joint should be equal or less than the maximum utilisation ratio of any of the connected members. As a simplification the comparison of the utilisation ratio may be performed for normal temperature.

3) At normal temperature the joints are designed according to EN 1993-1-8

If the conditions above are not fulfilled, the resistance of the joints in a fire situation shall be checked according to the following provisions.

6.7.1 Resistance of bolted joints

Net section failure at fastener holes need not be considered, provided that there is a fastener in each hole, because the steel temperature is lower at joints due to presence of additional material.

6.7.1.1 Shear resistance and bearing resistance

The resistances in different bolted joint categories A...C are calculated as follows:

Bolted joint category A: Bearing Type

The shear resistance of a bolt in fire situation is [9,10,11]:

$$F_{v.t.Rd} = k_{b.\theta} F_{v.Rd} \frac{\gamma_{M2}}{\gamma_{M.fi}} \tag{6.42}$$

where $k_{b.\theta}$ is temperature dependent reduction factor for the bolt strength according to Table 6.5
$F_{v.Rd}$ is the shear resistance of the bolt per shear plane at normal temperature
γ_{M2} is the partial safety factor for resistance at normal temperature (Table 2.5)
$\gamma_{M.fi}$ is the partial safety factor for resistance in fire design (Table 2.5)

The bearing resistance of a bolt in fire situation is [9,10,11]:

$$F_{b.t.Rd} = k_{b.\theta} F_{b.Rd} \frac{\gamma_{M2}}{\gamma_{M.fi}} \tag{6.43}$$

where $k_{b.\theta}$ is temperature dependent reduction factor for the bolt strength according to Table 6.5
$F_{b.Rd}$ is the bearing resistance of the bolt at normal temperature
γ_{M2} is the partial safety factor for resistance at normal temperature (Table 2.5)
$\gamma_{M.fi}$ is the partial safety factor for resistance in fire design (Table 2.5)

Bolted joint category B: Slip resistant at serviceability limit state and
Bolted joint category C: Slip resistant at ultimate limit state:

Slip resistant joints are supposed to slip in fire situation and the resistance of a single bolt is calculated as in joint Category A [9,10,11].

6.7.1.2 Tension resistance

Bolted joint categories D and E: Non-preloaded and preloaded bolts:

The tension resistance of a bolt in fire situation is [9,10,11]:

$$F_{ten.t.Rd} = k_{b.\theta} F_{t.Rd} \frac{\gamma_{M2}}{\gamma_{M.fi}} \tag{6.44}$$

where $k_{b.\theta}$ is temperature dependent reduction factor for the bolt strength according to Table 6.5
$F_{t.Rd}$ is the tension resistance of the bolt at normal temperature
γ_{M2} is the partial safety factor for resistance at normal temperature (Table 2.5)
$\gamma_{M.fi}$ is the partial safety factor for resistance in fire design (Table 2.5)

Table 6.5 Reduction factor for strength of bolts depending on the temperature [9,10,11]

Temperature θ_a	Reduction factor for bolts $k_{b.\theta}$
20 °C	1,000
100 °C	0,968
150 °C	0,952
200 °C	0,935
300 °C	0,903
400 °C	0,775
500 °C	0,550
600 °C	0,220
700 °C	0,100
800 °C	0,067
900 °C	0,033
1000 °C	0,000

6.7.2 Resistance of welded joints

6.7.2.1 Butt welds

The design strength of a full penetration butt weld, for temperatures up to 700 °C, is taken as equal to the strength of the weaker part joined using the appropriate reduction factor of the parent metal. For temperatures > 700 °C, the reduction factor given in the following for fillet welds can be applied also for butt welds [9,10,11].

6.7.2.2 Fillet welds

The resistance of a fillet weld per unit length in fire situation is [9,10,11]:

$$F_{w.t.Rd} = k_{w.\theta} F_{w.Rd} \frac{\gamma_{M2}}{\gamma_{M.fi}} \tag{6.45}$$

where $k_{w.\theta}$ is temperature dependent reduction factor for the weld strength according to Table 6.6

$F_{w.Rd}$ is the resistance of the fillet weld per unit length at normal temperature according to clause 3.3.3

γ_{M2} is the partial safety factor for resistance at normal temperature (Table 2.5)

$\gamma_{M.fi}$ is the partial safety factor for resistance in fire design (Table 2.5)

The reduction factor for fillet weld is presented in Table 6.6. From the table it can be seen that when the temperature rises, the strength of the fillet weld begins to decrease already after 300 °C, and it decreases faster than the strength of the parent metal (Table 6.1). If the fillet weld is designed at normal temperature to have equal strength with the parent metal (for example the welds of the brace members in a hollow section lattice), and if the weld is wanted to have equal strength also in fire situation, the throat thickness of the weld must be increased according to Figure 6.16 by the ratio of the reduction factors $k_{y.\theta}/k_{w.\theta}$. When the temperature of the steel component and the weld (for example due to sufficient fire protection) is not more than 600 °C, an enlargement by factor $k_{y.\theta}/k_{w.\theta}$ = 1,25 is sufficient. At higher temperatures the enlargement factor begins to grow fast. If the temperature of the weld is not more than 300 °C, enlargement of the throat thickness is not needed.

Table 6.6 Reduction factor for strength of the welds depending on the temperature [9,10,11]

Temperature θ_a	Reduction factor for the welds $k_{w.\theta}$
20 °C	1,000
100 °C	1,000
150 °C	1,000
200 °C	1,000
300 °C	1,000
400 °C	0,876
500 °C	0,627
600 °C	0,378
700 °C	0,130
800 °C	0,074
900 °C	0,018
1000 °C	0,000

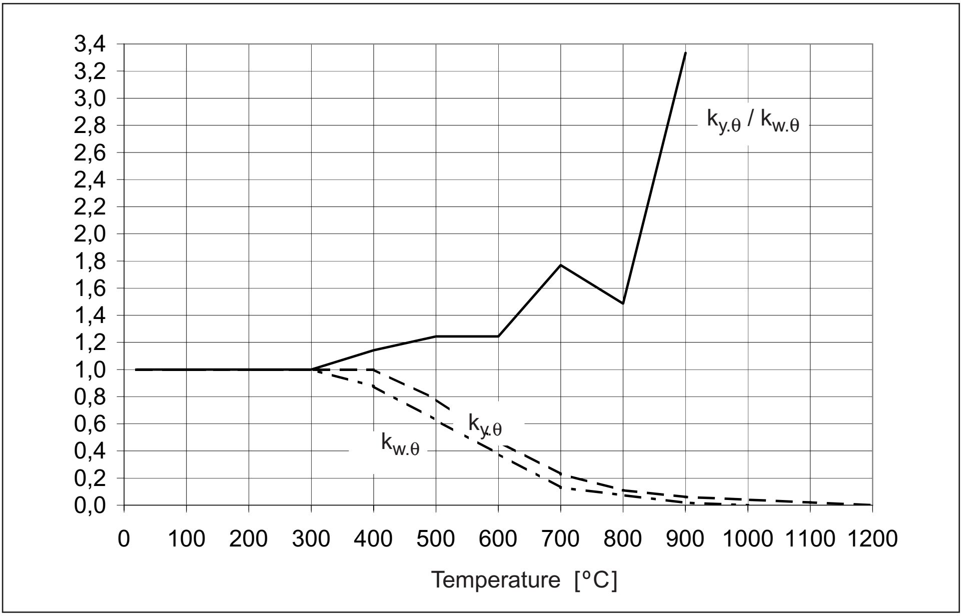
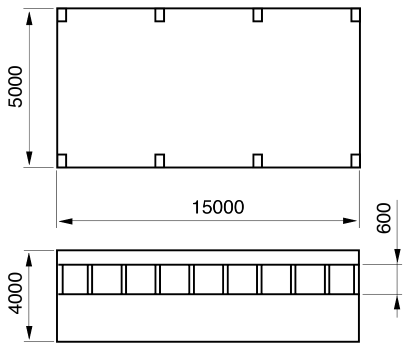


Figure 6.16 Variation of the strength of the parent metal and the fillet weld as a function of the temperature

Example 6.1

Structural hollow sections of size 180×180×6 are chosen as columns at the sides and corners of a building presented in the adjacent figure. In fire situation the compressive load is $E_{fi.d} = N_{fi.Ed} = 550$ *kN.*



The fire resistance period required for the building is 15 min (fire resistance class R15). The buckling length of the columns is $L_{fi} = 4{,}0$ *m. There is a clearance between the columns and the walls (due to wall purlins). Consequently, in respect to fire design, the columns are assumed to be exposed to fire on all four sides. The temperature development in the fire compartment is determined using the standard temperature-time curve (formula (6.1)).*

The steel grade of the hollow sections is SSAB Domex Tube Double Grade, which fulfills the EN 10219 requirements for both steel grades S420MH and S355J2H. Thereby the design calculations may be performed at designer's own choice either according to grade S420 or grade S355. Grade S420 is chosen in this Example as design basis.

Temperature development of an unprotected hollow section:

The temperature increase of unprotected steel is obtained from formula (6.2):

$$\Delta\theta_{a.t} = k_{sh} \cdot \frac{A_m/V}{c_a \rho_a} \cdot \dot{h}_{net.d} \cdot \Delta t$$

The net heat flux per area consists of convection and radiation:

$$\dot{h}_{net.c} = \alpha_c(\theta_g - \theta_m) = 25(\theta_g - \theta_m)$$

$$\begin{aligned}\dot{h}_{net.r} &= \Phi \cdot \varepsilon_m \cdot \varepsilon_f \cdot \sigma \cdot [(\theta_r + 273)^4 - (\theta_m + 273)^4] \\ &= 1,0 \cdot 0,7 \cdot 1,0 \cdot 5,67 \cdot 10^{-8}[(\theta_r + 273)^4 - (\theta_m + 273)^4] \\ &= 3,969 \cdot 10^{-8}[(\theta_r + 273)^4 - (\theta_m + 273)^4]\end{aligned}$$

$$\begin{aligned}\dot{h}_{net.d} &= \dot{h}_{net.c} + \dot{h}_{net.r} \\ &= 25(\theta_g - \theta_m) + 3,969 \cdot 10^{-8}[(\theta_r + 273)^4 - (\theta_m + 273)^4]\end{aligned}$$

By substituting the material characteristics of steel and the section factor in formula (6.2), we obtain:

$k_{sh} = 1,0$	*correction factor for shadow effect for hollow sections*
$A_m/V = 171\ m^{-1}$	*section factor (when exposed to fire on all sides, Annex 11.1)*
$c_a = 600\ J/(kgK)$	*specific heat is assumed to stay constant (simplification)*
$\rho_a = 7850\ kg/m^3$	*unit mass of steel*
$\theta_g = \theta_r$	*it is assumed that in case of a member that is exposed to fire on all sides, the gas temperature in the fire compartment is equal to the effective radiation temperature of the fire environment (simplification)*
$\Delta t = 5\ s$	*time interval in calculations (permitted maximum)*

$$\begin{aligned}\Delta\theta_{a.t} &= k_{sh} \cdot \frac{A_m/V}{c_a \rho_a} \cdot \dot{h}_{net.d} \cdot \Delta t \\ &= 9,076 \cdot 10^{-4}(\theta_r - \theta_m)\Delta t + 1,441 \cdot 10^{-12}[(\theta_r + 273)^4 - (\theta_m + 273)^4]\Delta t\end{aligned}$$

$$\Delta\theta_{a.t} = 4,538 \cdot 10^{-3}(\theta_r - \theta_m) + 7,205 \cdot 10^{-12}[(\theta_r + 273)^4 - (\theta_m + 273)^4]$$

The enclosed figure presents the temperature development of an unprotected 180×180×6 hollow section in standard fire. The curve is calculated using the formula above with time steps of 5 seconds. The maximum temperature conforming to the required fire resistance period (15 min) is:

$$\theta_{a.max} = 683\ °C$$

Temperature development of a fire-protected hollow section:

The column $180 \times 180 \times 6$ is protected with 15 mm thick mineral wool boards. The temperature increase of the fire-protected steel structure conforms with the formula (6.7):

$$\Delta\theta_{a.t} = \frac{\lambda_p / d_p}{c_a \rho_a} \cdot \frac{A_p}{V} \cdot \frac{\theta_{g.t} - \theta_{a.t}}{1 + \phi / 3} \cdot \Delta t - (e^{\phi / 10} - 1) \cdot \Delta\theta_{g.t} \quad \text{but } \Delta\theta_{a.t} \geq 0 \text{ if } \Delta\theta_{g.t} > 0$$

The material properties needed in above formula for the fire protection material and for the steel are:

$$\frac{A_p}{V} = \frac{2(b+h) \cdot 1m}{A \cdot 1m} = \frac{2 \cdot (0,18m + 0,18m) \cdot 1m}{4083 \cdot 10^{-6} m^2 \cdot 1m} = 176,3\ m^{-1}$$

section factor for a hollow section having hollow encasement protection

$c_a = 600$ J/(kgK) *specific heat is assumed to stay constant (simplification)*

$d_p = 15$ mm *thickness of the fire protection material*

$\Delta t = 5$ s *time interval in calculations (for fire-protected max. 30 seconds)*

$\lambda_p = 0,25$ W/(mK) *thermal conductivity is assumed to stay constant (simplification)*

$\rho_a = 7850$ kg/m^3 *unit mass of steel*

The unit mass and specific heat of the fire protection material are:

$c_p = 1000$ J/(kgK) *specific heat of the fire protection material*

$\rho_p = 150$ kg/m^3 *unit mass of the fire protection material*

Factor ϕ is determined by formula (6.8):

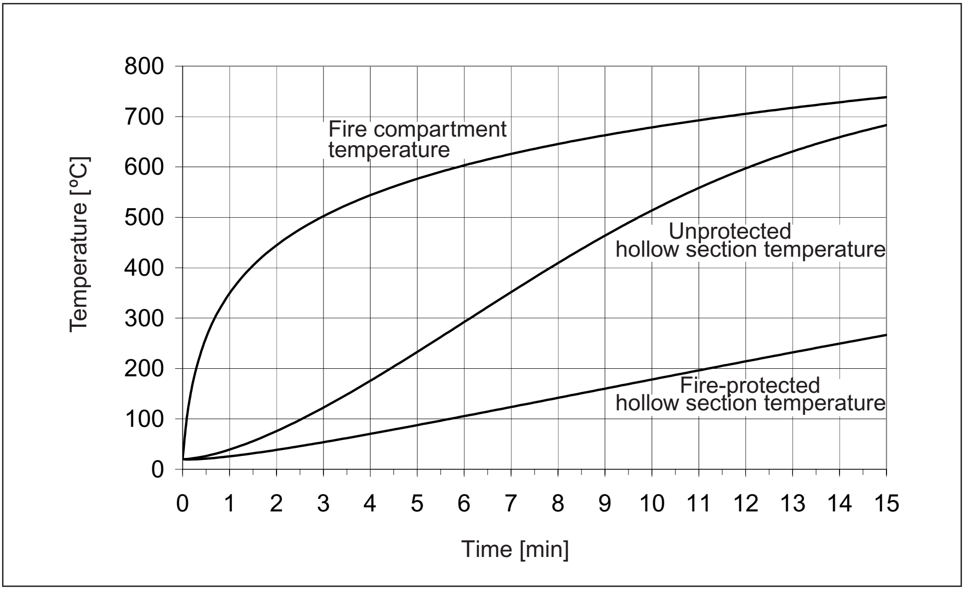
$$\phi = \frac{c_p \rho_p}{c_a \rho_a} \cdot d_p \cdot \frac{A_p}{V} = \frac{1000 \cdot 150}{600 \cdot 7850} \cdot 0,015 \cdot 176,3 = 0,0842$$

By substituting the material properties and factor ϕ in formula (6.7), we obtain the following result:

$$\Delta\theta_{a.t} = 3,034 \cdot 10^{-3} \cdot (\theta_{g.t} - \theta_{a.t}) - 8,460 \cdot 10^{-3} \cdot \Delta\theta_{g.t}$$

The enclosed figure presents the temperature development of a $180 \times 180 \times 6$ hollow section protected with 15 mm mineral wool boards in a standard fire. The curve is calculated using the formula above with time steps of 5 seconds. The maximum temperature conforming to the required fire resistance period (15 min) is:

$\theta_{a.max} = 267$ °C



Resistance of the structure:

In fire design, the resistance is calculated using the cross-section Class determined for fire situation, which may differ from the cross-section Class at normal temperature. Therefore, check the cross-section classification of the hollow section separately for normal temperature (Table 2.7) and for fire situation:

Web and flange (normal temperature):

$$\frac{\bar{b}}{t} = \frac{(b-3t)}{t} = \frac{(180-3\cdot 6)}{6} = 27{,}0 \le 38\varepsilon = 38\cdot\sqrt{235/f_y} = 38\cdot\sqrt{235/420} = 28{,}4$$

$\Rightarrow$ *for normal temperature, cross-section Class 2 (thereby at normal temperature the cross-section is fully effective)*

Web and flange (fire situation):

the reduced value for ε *to be applied in fire situation is calculated from formula (6.20):*

$$\varepsilon = 0{,}85\sqrt{235/f_y} = 0{,}85\sqrt{235/420} = 0{,}6358$$

$$\frac{\bar{b}}{t} = \frac{(180-3\cdot 6)}{6} = 27{,}0 > 42\varepsilon = 42\cdot 0{,}6358 = 26{,}7$$

$\Rightarrow$ *for fire situation, cross-section Class 4*

Effective cross-section in fire situation:

In fire situation the effective cross-section is determined by using strength and modulus of elasticity at normal temperature
⇒ effective cross-section in fire situation = effective cross-section at normal temperature

The cross-section of the here applied hollow section is fully effective at normal temperature (Class 2). Thereby it can be easily concluded, even without any calculations, that the effective cross-section in fire situation is here the same as gross cross-section:

$$A_{fi.eff} = A_{eff} = A = 4083\ mm^2 \qquad (Annex\ 11.1)$$

Compression resistance:

When calculating the resistance, the changes in the steel strength and modulus of elasticity caused by temperature shall be taken into account. The resistance is determined by the maximum temperature during the fire resistance period. In case of cross-section Class 4, reduction factors $k_{p0,2.\theta}$ and $k_{E.\theta}$ are applied to determine the strength and modulus of elasticity respectively. The values for these factors are obtained from Table 6.1 by linear interpolation. The results are presented in the table below.

Fire situation	$\theta_{a.max}$ (°C)	$k_{p0,2.\theta}$	$k_{E.\theta}$	$\bar{\lambda}_\theta$ formula (6.26)	χ_{fi} formula (6.23)	$N_{b.fi.t.Rd}$ (kN) formula (6.22)
Unprotected hollow section	683	0,1589	0,1606	0,8018	0,6094	166,1
Fire-protected hollow section	267	0,8163	0,8330	0,7980	0,6115	856,0

The unprotected column does not meet the fire resistance requirement because:

$$N_{b.fi.t.Rd} = 166,1\ kN < N_{fi.Ed} = 550\ kN \qquad not\ \ OK$$

The fire-protected column can resist a 15 minute fire if a 15 mm layer of mineral wool is applied as fire protection, because:

$$N_{b.fi.t.Rd} = \chi_{fi} A_{eff} k_{p0,2.\theta} f_y / \gamma_{M.fi}$$
$$= 0,6115 \cdot 4083 \cdot 0,8163 \cdot 420/1,0 = 856,0\ kN \geq N_{fi.Ed} = 550\ kN \qquad OK$$

Critical temperature as determined by the utilisation ratio:

The above presented assessment for fire resistance can also be determined by the utilisation ratio. Substituting values in formulae (6.22) and (6.12) we obtain:

$$N_{b.fi.t.Rd} = \chi_{fi} A_{eff} k_{p0,2.\theta} f_y / \gamma_{M.fi}$$
$$= 0,6070 \cdot 4083 \cdot 1,0 \cdot 420/1,0 = 1041\ kN \qquad (fire\ situation\ at\ time\ t = 0)$$

$$\mu_0 = \frac{E_{fi.d}}{R_{fi.d.0}} = \frac{550}{1041} = 0,5283$$

The critical temperature is obtained from formula (6.11):

$$\theta_{a.cr} = 39,19\ ln\left[\frac{1}{0,9674 \cdot \mu_0^{3,833}} - 1\right] + 482$$

$$= 39,19\ ln\left[\frac{1}{0,9674 \cdot 0,5283^{3,833}} - 1\right] + 482 = 576°C$$

Hence, using the critical temperature we obtain the corresponding result as above:

Unprotected hollow section: $\theta_{a.max} = 683\,°C > \theta_{a.cr} = 576\,°C$ *not OK*
Fire-protected hollow section: $\theta_{a.max} = 267\,°C \leq \theta_{a.cr} = 576\,°C$ *OK*

Comparison S420 vs S355:
In case the design calculations would be performed according to grade S355, the compression resistance for the unprotected hollow section would be 195,1 kN and for the fire-protected hollow section 852,6 kN. Hence we can notice, that increase of the material strength S355 → S420 improves the compression resistance of the fire-protected hollow section (though only slightly), while the compression resistance of the unprotected hollow section decreases. This illogicality in the results is due to the fact, that in fire situation the hollow section chosen in this Example is classified as S355 into cross-section Class 3, while as S420 the same hollow section is classified into Class 4, wherein the reduction of resistance must be determined by using a more unfavourable factor $k_{p0,2.\theta}$. *The ratio of factor* $k_{p0,2.\theta}$ *to* $k_{y.\theta}$ *varies largely depending on the applied temperatures (here 683 °C for the unprotected and 267 °C for the fire-protected).*

Thus, the hereby derived results can not be generalized. In a common case, the hollow section which is designed by higher steel grade will have a higher resistance also in fire situation than a hollow section of lower steel grade, provided they both belong to the same cross-section Class.

6.8 Fire protection methods

A steel structure can be fire-protected either by insulating or by improving its heat retention capacity. Structural solutions can also be used to increase the fire resistance period. In addition to the costs of materials, also installation and maintenance costs should be considered when selecting the fire protection method.

Table 6.7 Fire protection methods

Principle	Methods
Heat insulation	- insulation boards - fire protection paints (intumescent coatings) - sprayed insulations
Improvement of heat retention capacity	- concrete infill - water infill - sprinkler systems
Structural fire protection	- suspended ceilings - placement of the columns outside the fire compartment - placement of the columns inside the wall

The methods basing on the heat insulation in Table 6.7 can further be divided on the basis of their execution into dry or wet systems. The products belonging to the dry systems are normally fixed with mechanical fasteners to encase the structures to be protected. Dry systems include for example [2,3]:

- mineral fibre batts
- vermiculite boards
- calcium silicate boards
- gypsum boards and elements
- wood fibre plaster boards and
- cement sellulose boards.

Wet fire protection systems include for example [2,3]:

- sprayed mineral fibres
- sprayed vermiculite
- intumescent coating (fire protection paint)
- concrete and
- water.

6.8.1 Fire protection by insulation

As compared to unprotected structure, insulated structure is slower to heat and slower to reach the critical temperature. The insulation can be executed either by using boards or by sprayed materials. The thermal conductivity λ_p of the fire protection material depends on the temperature of the material, which shall be taken into account when calculating the temperature of the steel member. The thermal conductivity characteristics of the fire protection material are usually presented in manufacturers' brochures. The following is a description of the properties and use of the most common fire protection materials.

Mineral fibre batts

Mineral fibre batts are efficient thermal insulation material. The batts are suitable for fire protection, if their sintering temperature is high enough, ca. 800-1100 ºC. The unit mass of the mineral fibre batts used for fire protection is 100-400 kg/m^3 and their thickness varies between 10-120 mm.

The batts are fixed mechanically or glued in place. Mechanical fasteners include steel pins welded onto the hollow section to be protected. Optionally cartrige-fired pins may be used. The batts are pressed through the pins onto the surface of the hollow section and they are secured using retaining washers. When carrying out the installation, attention should be paid to the tightness of the seams. If the batts are glued in place, the steel surface should be dry and free from dirt and oil.

Vermiculite boards

The base material in the vermiculite boards is exfoliated mica, and the binding agent is siliceous material, e.g. cement. The fire resistance of the boards is based on the great amount of water inherently contained in them as well as on their good thermal insulation capacity even at elevated temperatures. In the initial phase of the fire, thermal energy is consumed to evaporate the water inherently contained in the boards in the same way as with gypsum boards.

The unit mass of the vermiculite boards is 350-500 kg/m^3 and the thickness is 16-80 mm. The boards are fixed to encase the hollow section to be protected using heat-resistant plaster and pins or screws. In mechanical fastening, a gap of ca. 3 mm is left between the encasement and the hollow section. The boards may also be glued in place. In this case, the substrate should be dry and clean. The temperature should be over 0 ºC.

Calcium cilicate boards

Calcium silicate boards are fibre-reinforcced, and their thickness is 6-65 mm. The unit mass of the boards is 430-950 kg/m^3, and they may be cut with normal woodworking tools. The calcium silicate boards are normally fixed with screws as a casing around the hollow section.

Gypsum boards and elements

The fire protection properties of gypsum board are based on the high amount of crystalline water it inherently contains. The temperature of steel remains at 100 ºC during evaporation of the crystalline water. After the crystalline water has evaporated, a non-reinforced gypsum board will break down. The boards can be reinforced by adding glass fiber as a binding agent.

The unit mass of the gypsum boards is 770-980 kg/m^3. Boards with a thickness of 13 mm or 15 mm are normally used for protection, possibly installed in several layers.

The boards are fixed with screws to the hollow section. There can be layers from one to four.

Gypsum elements are manufactured using a mixture of gypsum, perlite and glass fibres. The elements may be manufactured, for example, in half-circle form, which gives the possibility to use them as fire protection for circular columns. The elements are glued together on site.

Wood fibre plaster boards

The wood fibre plaster boards are manufactured from a mixture of wood fibres and gypsum. The unit mass of the board is ca. 1200 kg/m^3 and its thickness is 15 or 22 mm. The boards are fixed with screws or pins.

Cement cellulose boards

In the manufacture of cement cellulose board, different mineral constituents are used in addition to cement and cellulose. The unit mass of the board is ca. 1100 kg/m^3. These boards are used in light-weight partitioning walls. They are fixed directly onto the steelwork using screws or, optionally, using fixing strips.

Sprayed mineral fibres

Mineral fibres and cement are sprayed with water onto the surface of the hollow section. A zinc-coated steel mesh, which at the same time also works as reinforcement, can also be used as a fixing base. The thickness of the completed layer is normally 10-60 mm.

The unit mass of the mineral fibre spray system is 220-500 kg/m^3. For example, spray painting or plastering may be used as a coating. Because the surface of the fire protection is porous, it needs to be protected from mechanical abrasion. Hard plastering, woven glass fabric or a suitable board cladding may be used as a protection.

Sprayed vermiculite

In the vermiculite spraying, vermiculite is used as an aggregate, and cement, calcium or gypsum as a binder, in addition to water. The slurry is sprayed directly onto the steel surface or to the reinforcement mesh. The thickness of the completed layer is normally 10-60 mm. Some slurries may also be plastered manually. The unit mass of the sprayed layer is 300-800 kg/m^3. The surface may be protected against mechanical abrasion using the same methods as with sprayed mineral fibres.

Intumescent coatings (fire protection paints)

Intumescent coatings are generally suited to dry indoor conditions (corrosivity category C1) [14].

Intumescent coatings foam and and swell when the temperature rises beyond ca. 250-300 ºC. The thick foam layer generated in the reaction becomes charred during the fire and thereby protects the steel structure. At normal operating temperature the intumescent coating behaves like a normal paint. After the foam formation, all the paint types do not withstand a sustained fire, but the charred paint layer may begin to peel off from the steel surface already before half an hour. Intumescent coatings can be used where the required fire resistance period is R15-R120 [2,3,14].

The intumescent coating is applied onto the surface of the steel by paint brush, painting roller or by spraying. The thickness of the dry film is normally 0,2-3 mm. The painting can be done either at the workshop or on site. The members to be protected are easier to paint in the workshop, but they should be well protected for transportation, because the intumescent coating is easily damaged [3].

Intumescent coatings are used as a painting system consisting of surface preparation of the steel structure, priming, intumescent coating and top painting. The total thickness of the painting system is normally 0,2-5,0 mm. The top paint shall be compatible with the intumescent coating, and the film thickness of the top paint should conform to the recommendations given by the manufacturer of the intumescent coating. The top painting can be renewed four times at highest. The total thickness of the top paint layers must not be more than 300 µm [14].

The benefit of intumescent coatings is in the small thickness needed for protection. Moreover, the appearance of the structure is similar to that achieved with conventional anti-corrosive painting. Intumescent coating is, however, considerably more expensive than the normal anti-corrosive painting.

6.8.2 Fire protection by improving the heat retention capacity of the steel structure

Concrete infill of hollow sections

Concrete infill of a hollow section column is a simple and efficient fire protection method which retains the appearance and the dimensions of the hollow section unchanged. The use of reinforcing steel significantly improves the fire resistance period of the hollow section. The amount of reinforcing steel can be adjusted to regulate the resistance of the column at normal temperature and in fire situation. This way the same column size can be used in a multi-storey building from the ground floor to the top.

Since the concrete infill is usually carried out on-site, the light weight and quick installability of the hollow sections can be fully utilised during erection. At normal temperature a concrete-filled hollow section functions as a composite structure, and in fire situation the loads are mostly transferred through the concrete filling and reinforcing steel.

For fire situations, the hollow section must be provided with steam exhaust holes. During the fire, the steam pressure is then dissipated through the holes without damaging the hollow section. When placing the concrete infill, sufficiently thin layers must be applied and compaction of the cast should be done with great care.

The design provisions for concrete-filled composite column at normal temperature and at fire situation are presented in Parts EN 1994-1-1 and EN 1994-1-2 of Eurocode, which are the Parts covering composite structures. In a handbook-form, the instructions to design a concrete-filled column are presented in [15]. There exists also special design software like **ColGraph**, which is addressed especially for design of concrete-filled hollow section columns. The software calculates the capacity curves in parametric form for concentrically loaded concrete-filled steel composite columns, as defined in publication [15] by Finnish Constructional Steelwork Association. More details about the software are presented in Annex 11.7.

Water infill of hollow sections

Water filling in a hollow section functions as a cooling agent. The thermal energy generated by the fire is consumed to heat and steam the water inside the hollow section. The effect of water cooling can be enhanced by connecting the hollow sections to an overhead water tank. In a fire situation the steamed water rises into the reservoar, and the cooled water returns from the reservoar back to the hollow sections. To prevent the water from freezing, an agent such as calcium carbonate or calcium nitrate may be added to the water.

Water cooling is an effective fire protection method. By arranging a water circulation, the temperature of hollow sections normally stays at 200-250 ºC during the whole fire. Water cooling can be applied only for fire protection of columns. To prevent any leakage, special attention must be paid to sealing of the joints in the water pipes through which the water circulates. The water cooling does not change the appearance of the hollow section, if the circulation pipes are installed inside the hollow section.

Sprinkler systems

A sprinkler is an automatic fire-fighting system, which starts operating when the temperature rises in a fire situation. The temperature of the fire compartment does not rise, after the sprinkler system has started operating. National regulations include instructions on allowing for sprinkler systems in the fire design. The profitability of installing a sprinkler system depends on the ratio of its installation costs to the costs of the fire protection.

In Finland, the fire protection of steel structures can be executed by using sprinkler systems up to the fire resistance class R90 according to the certificate by VTT [16].

6.8.3 Fire protection by structural means

The fire protection of hollow section structures may be reduced or it is not even needed, if suitable structural solutions are used. The use of structural solutions to improve the fire resistance of structural members and joints reduces the need for fire protection materials which increase material and installation costs. Structural fire protection must be applied individually for each case, and it should be taken into account already at the design stage.

Suspended ceiling

To obtain the space required for HVAC-installations (Heating, Ventilation and Air conditioning), the room height can be lowered by using a suspended ceiling, which then hides the pipe installations and other services. Suspended ceiling can be used as the fire protection of the structural components in the intermediate space (for example floor beams). In such a case, the suspended ceiling shall be designed and dimensioned accordingly. Also the fixing of the suspended ceiling to the floor or roof above shall then resist the actions during the fire. In practice, the fixing structure is often the critical structural component in the fire situation.

Placement of the columns outside the fire compartment

When placing the columns outside the external walls, the increase of fire compartment temperature need not be taken into account in column design. A prequisite for this is that the column is placed sufficiently far from the window openings. In a fire situation, the hot combustion gases and flames exiting trough the window openings increase also the temperature of the nearby steel columns. Window openings are usually placed so close to one another, that the columns need to be protected by using a flame protection. As the material of the flame protection, for example steel sheet can be used.

Placement of the columns inside the wall

The fire-exposed-area of the column is reduced if the column can be placed partially or completely inside the fire protection material used in the wall structure. The materials to be used in the wall structure at the column locations must be fireproof in order to utilise their protective effect in fire design. A problem may be the connection of the stiffening bracing members to the column which is placed inside the wall.

6.9 References

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7. DESIGN OF HOLLOW SECTION STRUCTURES

A structural hollow section is a versatile structural component that can be used in many places of the building. Hollow sections show their best characterics in columns and lattice structures. The design of hollow section structures is simple and easy since, due to the good torsional stiffness, lateral-torsional buckling and torsional buckling are seldom governing factors in the design. This also enables the efficient utilisation of design software. In this Chapter, the design process of a hollow section structure is considered as a whole. First, the essential issues affecting the design are presented in general, after which a design solution for an Example building project is presented.

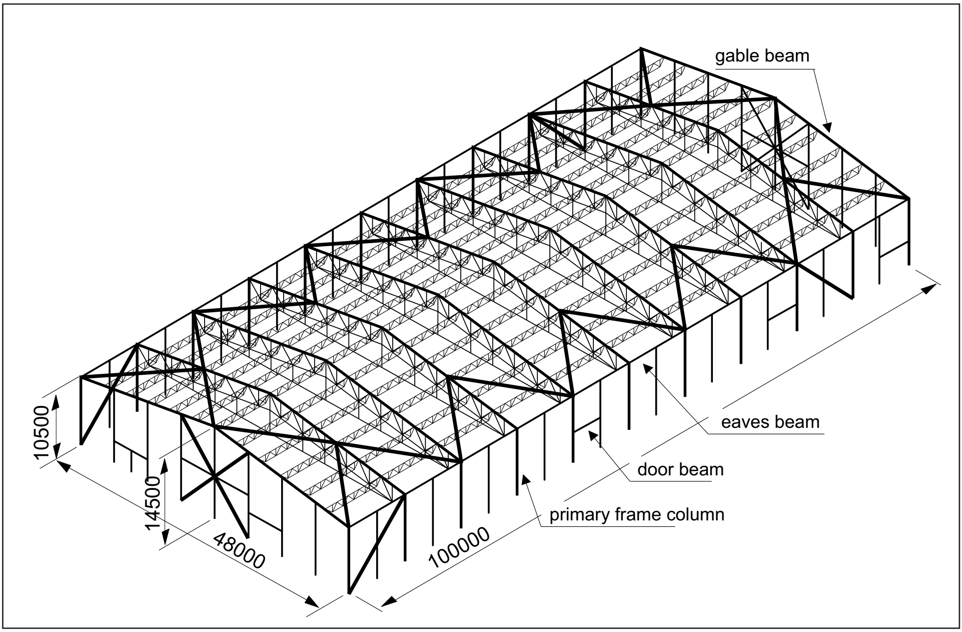


Figure 7.1 Example building project

The Example building is an exhibition hall shown in Figure 7.1. The building is used for arranging fairs and meetings.

The framework of the building consists of hollow section columns and of primary and secondary trusses. The bracing system of the building comprises a horizontal truss in the plane of the roof and wind bracings in the walls.

Hollow sections were chosen for columns, because the wall structure is designed using lightweight wool elements which do not support the colums about the minor axis. Hollow sections have high torsional stiffness and sufficient bending stiffness about the minor axis, which makes them a good solution in this case.

As a roof girder, it is usually advantageous to choose a normal K, N or KT type hollow section truss (clause 7.4). In these trusses the chords and the brace members are structural hollow sections and the joints of the brace members are simple. However, for the Example building project presented herein, a truss structure with a tension rod has been chosen because the building is located at difficult transport conditions. Due to transport requirements, in this Example the maximum depth of the prefabricated elements must not exceed 2,5 m. With the tension rod solution, the entire depth of the structure can be taken into account in the design to get the weight of the structure lighter. The truss is divided into erecting blocks so that the assembly joint is at the roof ridge.

A frame spacing of 10 m is chosen, as it produces an economic solution for the Example building. Between the main frames, there are longitudinal purlin trusses with a spacing of 4 m, which enables the use of a low load bearing profile. In addition, the purlin trusses are an easy way to provide lateral restraint to the top and bottom chord of the main truss.

The building is braced in the roof plane by horizontal bracing trusses (Figure 7.2). This solution produces smaller external dimensions for the foundation and the columns. Hollow sections are used as bracing members due to their excellent compression resistance.

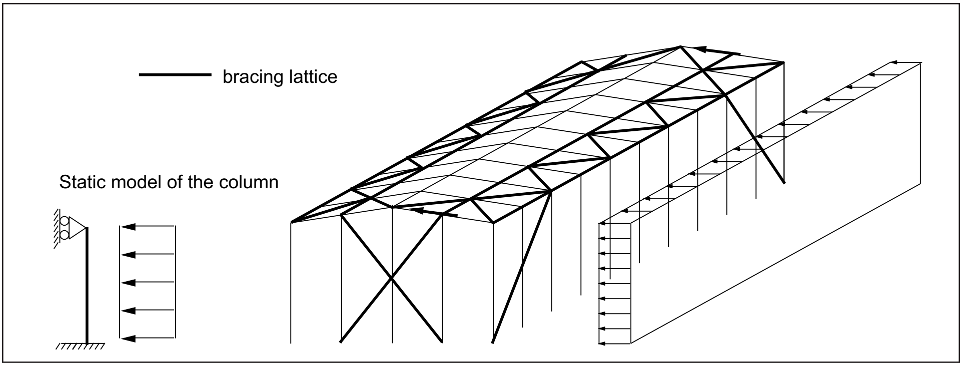


Figure 7.2 Horizontal bracing of the Example building (all columns are not shown)

On the external walls, there are bracing elements constructed from hollow sections, functioning as tension members. Here, too, hollow sections are an advantageous solution, as their stiffness makes the erection easier and the rigid elements retain their shape well. With respect to the foundations, the bracings would be, however, better to design both as tension and compression members, as the load transmitted to the foundation could then be divided into two (Figure 7.3).

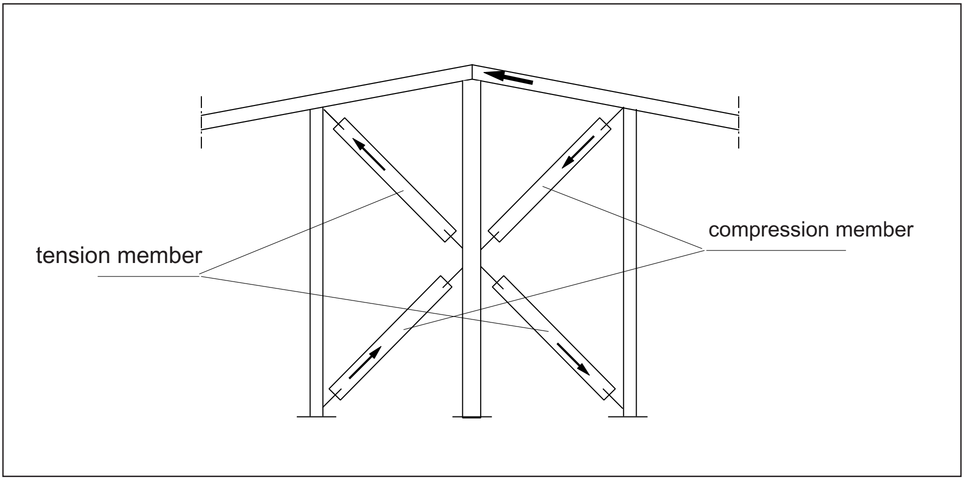


Figure 7.3 Bracings on the short side (the load at the ridge is transferred to the tension brace)

The wind columns on the long side are restrained at their top by the roof bracing to make them non-sway. The support force at the upper end of the wind columns is transferred to the main frames through the roof profile.

7.1 Loads on the structure

Structural loads are covered in Parts EN 1990 and EN 1991 of Eurocode as follows:

EN 1990	Basis of structural design
EN 1991-1-1	General actions: densities, self-weight, imposed loads for buildings
EN 1991-1-2	Actions on structures exposed to fire
EN 1991-1-3	Snow loads
EN 1991-1-4	Wind actions
EN 1991-1-5	Thermal actions
EN 1991-1-6	Actions during execution
EN 1991-1-7	Accidental actions
EN 1991-2	Traffic loads on bridges
EN 1991-3	Actions induced by cranes and machinery
EN 1991-4	Actions on silos and tanks

Because of the wide extent of the subject, this handbook does not present a comprehensive study on the determination of the loads, but the subject is considered only briefly as necessary in regard to the Example building.

The actions on the structures are divided in [1,1a] into permanent actions (G), variable actions (Q) and accidental actions (A). Permanent actions include the self-weight of the structure and the weight of the equipment fixed to the building. Variable actions include imposed loads, snow loads and wind actions. Accidental actions include for example actions in fire as well as seismic actions (i.e. earthquake actions).

This handbook does not consider accidental situations, except fire situation (see Chapter 6). On export, projects may arise where the seismic actions need to be considered. Seismic design is covered in Part EN 1998 of Eurocode, and EN 1998 related additional guidance is placed on web at the sites [www.eurocodes.fi] and [http://eurocodes.jrc.ec.europa.eu].

7.1.1 Self-weight and imposed loads

The self-weight and imposed loads of the buildings are presented mainly in Part EN 1991-1-1 of Eurocode [3,4].

The actions caused by the self-weight of the structures are normally calculated using the nominal unit masses and nominal dimensions (i.e. the dimensions presented in the drawings). For example, the unit mass of steel is taken as 7850 kg/m^3 and the unit mass of normally reinforced concrete as 2500 kg/m^3.

Loading caused by people, machines and movable objects is regarded as imposed loads. The self-weight of industrial equipment is also normally regarded as an imposed load. In [3,4], the areas of buildings are classified into five different categories (Categories A...E), which may further be divided into subcategories. The imposed loads to be applied in structural design for the loaded floor areas have been defined for each of these categories. In some cases, loads may be reduced in the way presented in [3,4] and its National Annex [5]. The reduction factor for the loads on beams is dependent on the loading area, and the reduction factor for the loads on columns is dependent on the number of storeys. The imposed load in an industrial buildings is determined by the weight of the machinery and equipment to be used in the building, as well as by the movable goods in the building.

7.1.2 Snow load

Snow loads on the roofs of the buildings are covered by Part EN 1991-1-3 of Eurocode [6,7].

The snow load on the roof is determined as vertical load to a horizontal projection of the roof as follows [6,7]:

$$s = \mu_i C_e C_t s_k \tag{7.1}$$

where μ_i is the shape coefficient of the snow load
C_e is the exposure coefficient
C_t is the thermal coefficient
s_k is the characteristic value of snow load on the ground

The recommended values of Eurocode for the exposure coefficient related to different topography types are presented in Table 7.1. The final values are determined in the National Annex.

Finnish National Annex to standard EN 1991-1-3 [8]:

In Finland, the values given in Table 7.1 are used for the exposure coefficient C_e.

For the thermal coefficient of the roof, usually the value C_t = 1,0 is used. The final value is determined in the National Annex.

Finnish National Annex to standard EN 1991-1-3 [8]:

If the thermal insulation of the roof structure is insignificant, the thermal coefficient C_t may be reduced on the basis of a more accurate study. The snow load s_k shall, however, always be at least 0,5 kN/m^2.

Table 7.1 Exposure coefficient C_e [6,7,8]

Topography	Eurocode	Finland
	C_e	C_e
Windswept a)	0,8	0,8 *)
Normal b)	1,0	1,0
Sheltered c)	1,2	1,0
a) **Windswept topography:** flat, unobstructed areas exposed on all sides without, or little shelter afforded by terrain, higher construction works or trees b) **Normal topography:** areas where there is no significant removal of snow by wind on construction work, because of terrain, other construction works or trees c) **Sheltered topography:** areas in which the construction work being considered is considerably lower than the surrounding terrain or surrounded by high trees and/or surrounded by higher construction works		
*) However, for roofs with the smaller horizontal dimension more than 50 m, the coefficient C_e is 1,0		
In this table the values according to Part EN 1991-1-3 of Eurocode and the values according to Finnish National Annex are presented. **The requirements valid in other countries must be checked from the National Annex of the relevant country.**		

In regard with EN 1991-1-3, the characteristic values of snow load on the ground s_k are determined in the National Annex of each country. The snow maps of different countries presented in Annex C of EN 1991-1-3 are only suggestive.

Finnish National Annex to standard EN 1991-1-3 [8]:

The characteristic values of snow load on the ground s_k are determined from the snow map presented in Figure 7.4. The values presented in the figure are minimum values. Greater values can be agreed upon for individual projects.

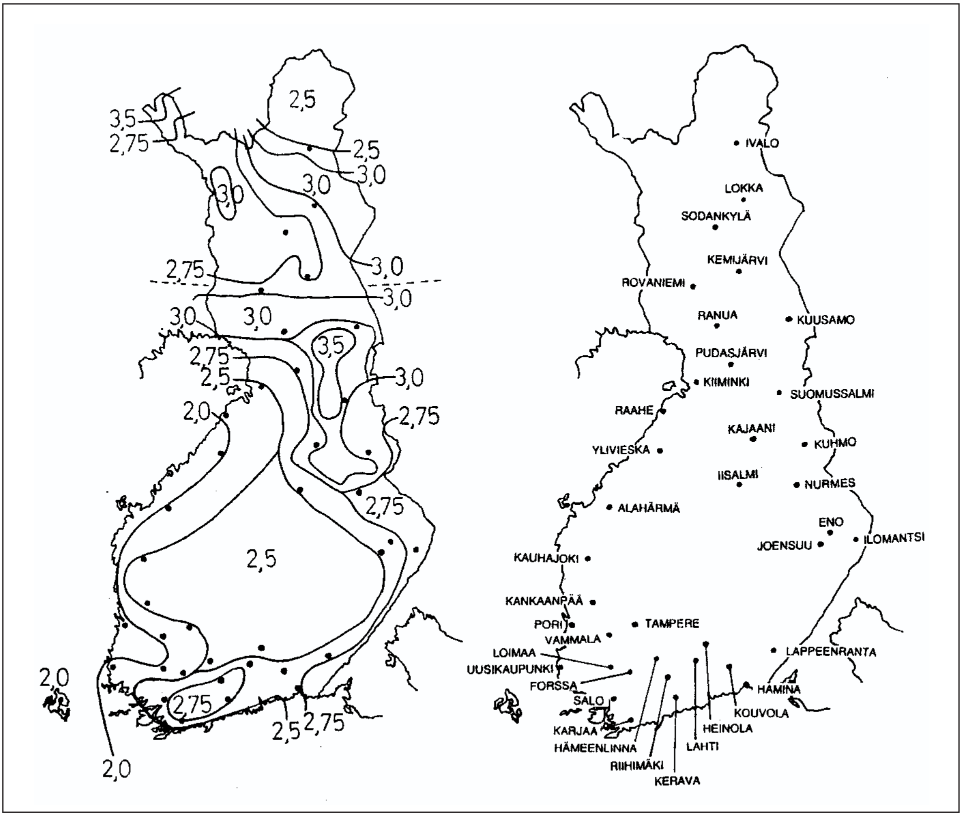


Figure 7.4 Characteristic values of snow load on the ground in Finland (kN/m^2). The intermediate values are obtained by linear interpolation in proportion to distances from the nearest curves [8]

The value of the shape coefficient μ_i depends on the shape of the roof and the load arrangement to be considered. For monopitch and duopitch roofs, the values of the shape coefficient μ_1 are presented in Figure 7.5 and Table 7.2. The load arrangements to be considered for monopitch and duopitch roofs are presented in Figure 7.6. Other roof shapes, see EN 1991-1-3.

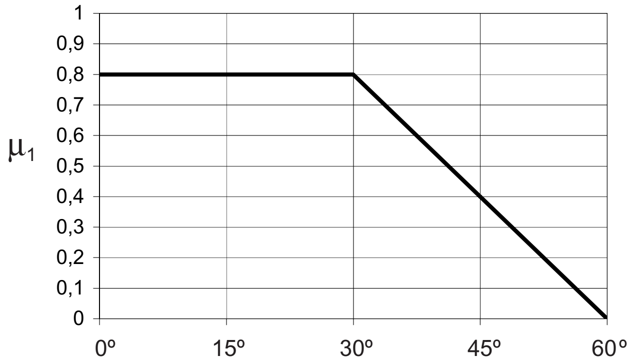


Figure 7.5 Snow load shape coefficient for monopitch and duopitch roofs [6,7]

Table 7.2 Snow load shape coefficient for monopitch and duopitch roofs [6,7]

Angle of pitch of roof α	$0° \leq \alpha \leq 30°$	$30° < \alpha < 60°$	$\alpha \geq 60°$
μ_1	0,8	$0{,}8(60 - \alpha)/30$	0,0
The values of the table apply when the snow is not prevented from sliding off the roof. Where snow fences or other obstructions exist or where the lower edge of the roof is terminated with a parapet, then the snow load shape coefficient should not be reduced below 0,8.			

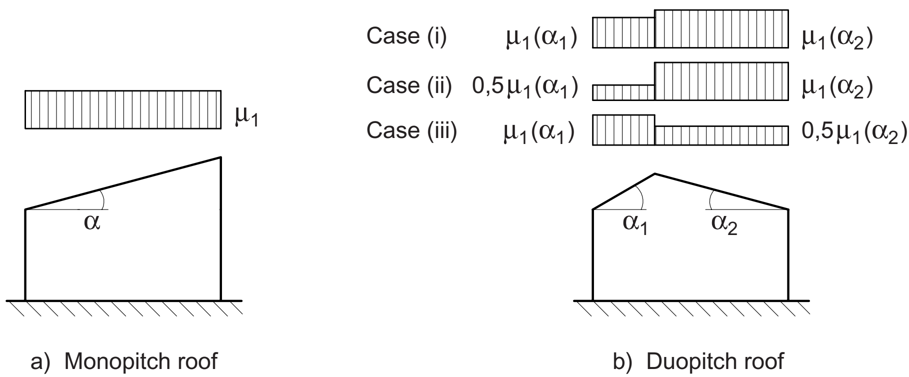


Figure 7.6 The load arrangements to be considered for monopitch and duopitch roofs [6,7]

7.1.3 Wind load

Wind actions are covered by Part EN 1991-1-4 of Eurocode [9...12].

The wind actions are applied to the surface of the structure as forces perpendicular to the surface. The wind actions act directly to the external surfaces of the enclosed structures, but may act directly or indirectly also to the inner surfaces of the structures (Figure 7.7). Additionally, when large areas of structures are swept by the wind, friction forces acting tangentially to the surface may be significant [9...12].

The net pressure on a wall, roof or element is the difference between the pressures on the opposite surfaces taking due account of their sign. According to Figure 7.7, the pressure directed towards the surface is taken as positive and suction directed away from the surface as negative [9...12].

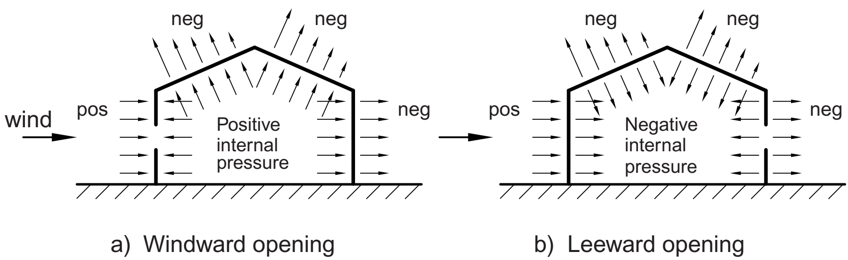


Figure 7.7 Pressures on surfaces [9...12]

The pressures acting on surfaces are calculated as follows [9...12]:

$$w_e = q_p(z_e) \cdot c_{pe} \qquad \textit{wind pressure acting on external surfaces} \qquad (7.2)$$

$$w_i = q_p(z_i) \cdot c_{pi} \qquad \textit{wind pressure acting on internal surfaces} \qquad (7.3)$$

where $q_p(z_e)$ is the peak velocity pressure (external)
z_e is the reference height for the external pressure
c_{pe} is the pressure coefficient for the external pressure (Table 7.4)
$q_p(z_i)$ is the peak velocity pressure (internal)
z_i is the reference height for the internal pressure
c_{pi} is the pressure coefficient for the internal pressure

The peak velocity pressure which is needed in formulae (7.2) and (7.3) is calculated as follows [9...12]:

$$q_p(z) = c_e(z) \cdot q_b \tag{7.4}$$

$$q_b = \frac{1}{2} \cdot \rho \cdot v_b^2 \tag{7.5}$$

$$v_b = c_{dir} \cdot c_{season} \cdot v_{b.0} \tag{7.6}$$

where $c_e(z)$ is the exposure factor which depends on the height of the considered surface measured from the ground, and on the surrounding terrain orography and roughness (terrain category), see Table 7.3 and Figure 7.8

q_b is the basic velocity pressure

ρ is the air density, ρ = 1,25 kg/m^3

v_b is the basic wind velocity

c_{dir} is the directional factor corresponding to different wind directions, the value of which may be given in the National Annex. The recommended value in Eurocode is c_{dir} = 1,0.

c_{season} is the season factor, the value of which may be given in the National Annex. The recommended value in Eurocode is c_{season} = 1,0.

$v_{b.0}$ is the fundamental value of the basic wind velocity, a characteristic value which is to be determined as the 10 minutes mean wind velocity, irrespective of wind direction and time of year, at 10 m above ground level in terrain category II (open country terrain) and for which the annual exceedence probability is 2 % (= mean return period is 50 years).

Finnish National Annex to standard EN 1991-1-4 [13]:

The fundamental value of the basic wind velocity is in Finland $v_{b.0}$ = 21 m/s. This value is applied in the whole country including the sea areas and mountain areas.

The values for the directional factor and season factor are not presented in the National Annex, thus the recommended values of Eurocode may be used: c_{dir} = 1,0 *and* c_{season} = 1,0.

The peak velocity pressure $q_p(z)$ and the pressure coefficients c_p needed in formulae (7.2) and (7.3) vary in different places and heights of the same building, why in practice the surfaces of the building are obliged to be assessed by dividing them into different zones and height strips as presented later on.

The wind force F_w acting on a structure can be calculated by vectorial summation of the forces $F_{w.e}$ and $F_{w.i}$ as well as friction forces F_{fr}, calculated on the basis of the external and internal pressures [9...12]:

$$F_{w.e} = c_s c_d \cdot \sum_{surfaces} (w_e \cdot A_{ref}) \quad \textit{external forces} \tag{7.7}$$

$$F_{w.i} = \sum_{surfaces} (w_i \cdot A_{ref}) \quad \textit{internal forces} \tag{7.8}$$

$$F_{fr} = c_{fr} \cdot q_p(z_e) \cdot A_{fr} \quad \textit{friction forces} \tag{7.9}$$

where $c_s c_d$ is the structural factor, for which a simplified value $c_s c_d$ = 1,0 may be used. More accurate values, see EN 1991-1-4

A_{ref} is the reference area of the individual surface perpendicular to the wind

c_{fr} is the friction coefficient, see EN 1991-1-4

A_{fr} is the area of external surface parallel to the wind

w_e, w_i and $q_p(z_e)$ are calculated according to formulae (7.2) - (7.6)

The effects of wind friction (formula (7.9)) on the surface can be disregarded when the total area of all surfaces parallel with (or at a small angle to) the wind is equal to or less than 4 times the total area of all external surfaces perpendicular to the wind (windward and leeward) [9...12].

Table 7.3 Terrain categories [9...12]

Terrain category	Description of the terrain category
0	Sea or coastal area exposed to the open sea
I	Lakes or flat area with negligible vegetation and without obstacles
II	Area with low vegetation such as grass and isolated obstacles (trees, buildings) with separations of at least 20 obstacle heights
III	Area with regular cover of vegetation or buildings or with isolated obstacles with separations of maximum 20 obstacle heights (such as villages, suburban terrain, permanent forest)
IV	Area in which at least 15 % of the surface is covered with buildings and their average height exceeds 15 m

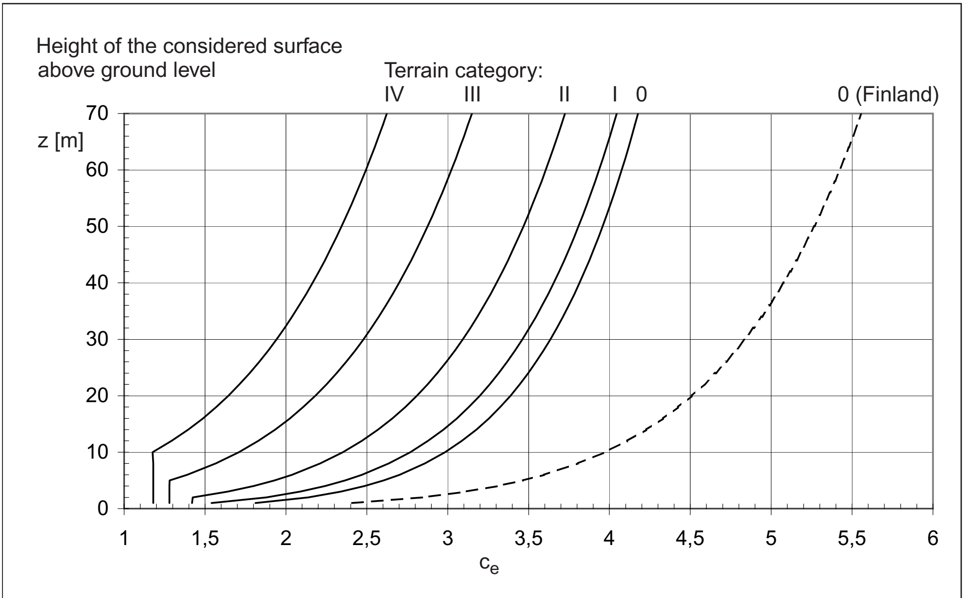


Figure 7.8 Exposure factor c_e, when the building is located on flat terrain where c_0 **= 1,0** (orography factor) and k_l **= 1,0** (turbulence factor) [9...13]

The values for the external pressure coefficient c_{pe} may be presented in the National Annex. In Table 7.4, the recommended values of Eurocode are presented, when the building plan is rectangular, and when the area of the wall A_{ref} is greater than 10 m^2. The values for the pressure coefficient c_{pe} when A_{ref} is less than 10 m^2 are given in Part EN 1991-1-4 of Eurocode. For determination of the pressure coefficient c_{pe} the different sides of the building are divided into zones A...E according to Figure 7.9 [9...12].

Finnish National Annex to standard EN 1991-1-4 [13]:

The values for the pressure coefficient c_{pe} are not presented in the National Annex, thus the recommended values of Eurocode presented in Table 7.4 are used.

The pressure coefficients of roofs for different roof shapes, see EN 1991-1-4.

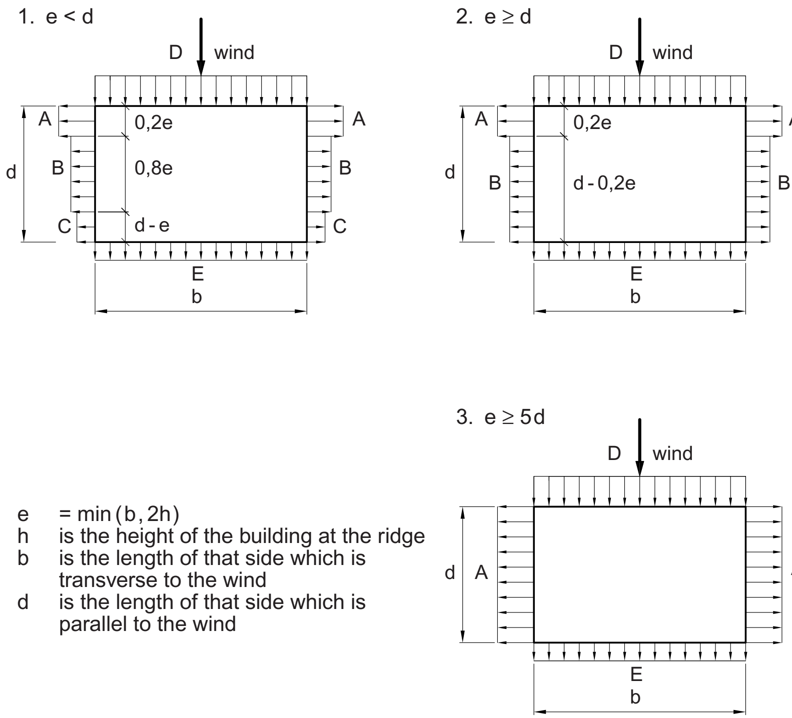
Table 7.4 Recommended values of external pressure coefficient c_{pe} for vertical walls of rectangular plan buildings, when A_{ref} ≥10 m^2 [9...13]

h/d a)	Zone				
	A	B	C	D	E
5	-1,2	-0,8	-0,5	+0,8	-0,7
1	-1,2	-0,8	-0,5	+0,8	-0,5
≤ 0,25	-1,2	-0,8	-0,5	+0,7	-0,3

a) For intermediate values of h/d, linear interpolation may be applied. Dimensions h and d, see Figure 7.9.

The values of the table also apply to walls of buildings with inclined roofs, such as duopitch and monopitch roofs.

In this table the recommended values according to Part EN 1991-1-4 of Eurocode that also apply in Finland, are presented. **National requirements must be checked from the National Annex of the relevant country.**



e = min (b, 2h)
h is the height of the building at the ridge
b is the length of that side which is transverse to the wind
d is the length of that side which is parallel to the wind

Figure 7.9 Zones A...E for vertical walls, plan view [9...12]

The windward wall (zone D, see Figure 7.9) of rectangular plan buildings is divided in vertical direction additionally into different wind load strips as follows (Figure 7.10):

- **when h ≤ b:**
  the windward wall of the building is considered in the vertical direction as one strip

- **when b < h ≤ 2b:**
  the windward wall of the building may be considered as two strips, wherein
  - the lower strip extends from the ground up to the height b and
  - the upper strip up from this

- **when h > 2b:**
  the windward wall of the building may be considered as consisting of several strips, wherein
  - the lowest strip extends from the ground up to the height b
  - the uppermost strip extends from the eaves downwards by amount of b, and
  - the region between the uppermost and lowest strip can be divided into horizontal strips, each having the height of h_{strip} according to Figure 7.10

The rules for the velocity pressure distribution for the leeward wall and sidewalls (zones A, B, C and E in Figure 7.9) may be given in the National Annex, or be defined for the individual project. In Eurocode it is recommended to use the height of the building as the reference height for the external pressure [9...12].

Finnish National Annex to standard EN 1991-1-4 [13]:
There are no instructions presented in the National Annex regarding this.

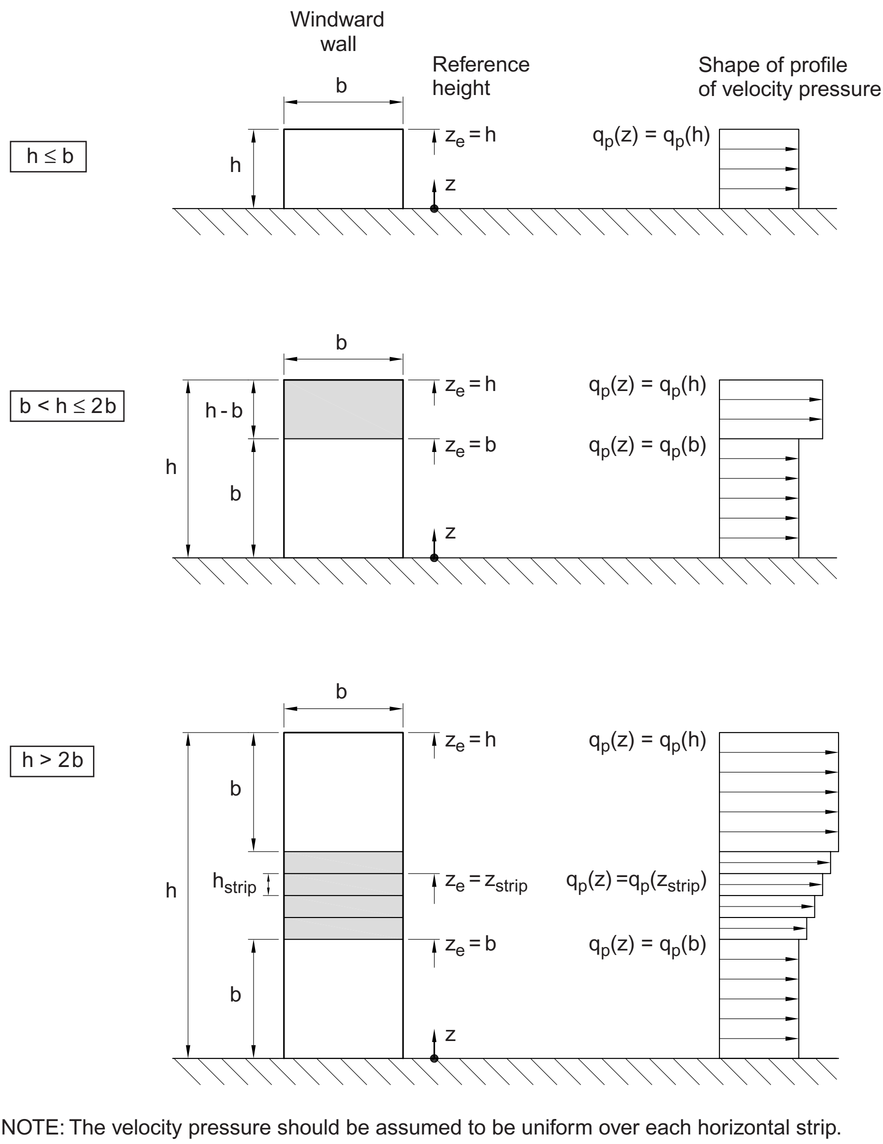


Figure 7.10 Division of the windward wall (zone D) to the horizontal strips to be considered, and determination of their reference height z_e [9...12]

7.1.4 Load combinations

The load combinations applied at ultimate limit state have been presented in Chapter 2. At serviceability limit state the following load combinations are defined [1,1a]:

$$\sum_{j \geq 1} G_{k.j} \text{"+"} Q_{k.1} \text{"+"} \sum_{i > 1} \psi_{0.i} Q_{k.i} \qquad \textit{characteristic combination} \qquad (7.10)$$

$$\sum_{j \geq 1} G_{k.j} \text{"+"} \psi_{1.1} Q_{k.1} \text{"+"} \sum_{i > 1} \psi_{2.i} Q_{k.i} \qquad \textit{frequent combination} \qquad (7.11)$$

$$\sum_{j \geq 1} G_{k.j} \text{"+"} \sum_{i \geq 1} \psi_{2.i} Q_{k.i} \qquad \textit{quasi-permanent combination} \qquad (7.12)$$

where "+" means "to be combined with"
(i.e. simultaneous action of the loads)
j is the index for permanent load
i is the index for variable load
$G_{k.j}$ is the characteristic value of permanent load
$Q_{k.1}$ is the characteristic value of the leading variable load
$Q_{k.i}$ is the characteristic value of other variable load
$\psi_{0.1}$ is the combination factor for the leading variable load (Table 2.4)
$\psi_{0.i}$ is the combination factor for other variable load (Table 2.4)

7.1.5 Requirements at serviceability limit state

In addition to the provisions at ultimate limit state, also the serviceability of the structure shall be checked using the loads at serviceability limit state, respectively. The deflections and displacements shall not deteriorate the functioning or appearance of the structure. It is not, however, clearly defined in Eurocode which load combination should be applied when verifying serviceability limit state (see expressions (7.10) - (7.12)). Neither are the quantitative limits for the deflections presented in Eurocode.

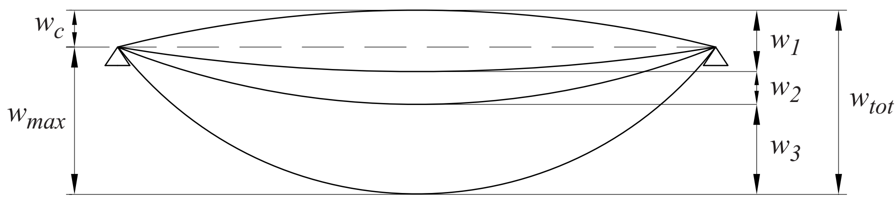
In Finland, in regard to steel structures, the National Annex to EN 1993-1-1 gives the following provisions:

Finnish National Annex to standard EN 1993-1-1 [17]:

At serviceability limit state, the load combination to be applied is the characteristic combination, expression (7.10).

The final vertical and horizontal deflections (w_{max}, see Figure 7.11) calculated with a static load, should not exceed the values in Table 7.5 if some harm is caused by it, unless due to type of structure, use or the nature of activity other values are determined to be more suitable.

Precamber (w_c, see Figure 7.11) may be used for compensation of the deflection due to permanent load, provided harm is not caused by it.



w_c Precamber in the unloaded structural member.

w_1 Initial part of the deflection under permanent loads of the relevant load combination according to expressions (7.10) - (7.12).

w_2 Long-term part of the deflection under permanent loads.

w_3 Additional part of the deflection due to the variable loads of the relevant load combination according to expressions (7.10) - (7.12).

w_{tot} Total deflection as sum of w_1 , w_2 , w_3.

w_{max} Remaining total deflection taking into account the precamber.

Figure 7.11 Vertical deflection

Table 7.5 Serviceability limit states for deflections and displacements according to the Finnish National Annex to EN 1993-1-1 [17]

Structure	Serviceability limit state for deflection
Main girders: - roofs - floors Cantilevers	 L / 300 L / 400 L / 150
Roof purlins	L / 200
Wall purlins	L / 150
Sheetings: - in roofs, with no risk for accumulation of water or other risk for failure of the roof - in roofs, with risk for accumulation of water or other risk for failure of the roof - when L ≤ 4,5 m - when 4,5 m < L ≤ 6,0 m - when L > 6,0 m - in floors - in walls - cantilevers	 L / 100 L / 150 30 mm L / 200 L / 300 L / 100 L / 100
Horizontal deflection of the structure: - 1- and 2-storey high buildings: - other buildings	 H / 150 H / 400
L = span H = the height of the building at the point to be checked	
Buildings supporting crane gantry girders, see EN 1993-6 and its National Annex.	

When plastic global analysis is used for the ultimate limit state, plastic redistribution of forces and moments at the serviceability limit state may occur. If so, the effects of it should be considered [14,15,16].

7.1.6 Additional horizontal forces due to initial imperfections

Part EN 1993-1-1 of Eurocode assigns various structural imperfections to be used in design calculations. These are not the same thing as the tolerances of the structures, presented in EN 1090-2. The imperfections presented in Eurocode correspond to characteristics that are related to the unloaded structure, i.e. they represent ‘initial input characteristics’ for the design calculations. By them the residual stresses and the geometrical imperfections (such as initial inclination, initial curvature etc.) of the unloaded structure are taken into account. In Eurocode they are called equivalent geometric imperfections. Sometimes they may be called also as additional horizontal forces, although such term does not appear in Eurocode [22].

Horizontal forces due to imperfections of the frame:

A frame may be classified as sway or non-sway. A sway frame is sensitive to global second-order effects (so-called P-Δ -effect). When the second-order effects are minor, the structure can be classified as non-sway.

The imperfections of the frame need be taken into account in the structural analysis of the frame only in the case of a sway frame [14,15,16].

The subject is presented in more details in Part EN 1993-1-1 of Eurocode and in [22,23].

Sway of the frames of the Example building has been prevented with bracing trusses and diagonal braces.

Fictitious horizontal forces to the bracing system due to imperfections of members to be restrained:

The effects of the initial imperfections are taken into account in the analysis of the bracing systems, where the bracing system stabilizes beams or compressed members in the lateral direction. This is taken into account by means of equivalent geometric imperfections of the members to be restrained.

In the case of a laterally braced frame, the frame itself and the bracing system for it can be designed separately as follows [24]:

- the frame is designed for all vertical loads
- the bracing system is designed for all horizontal loads.

The subject is presented in more details in Part EN 1993-1-1 of Eurocode and in [22,23].

7.1.7 Load determination for the Example building

In order to keep the herein worked examples universally applicable, the Example building is not addressed to any specific country. Consequently, the design calculations do not correspond to any existing National Annex of any existing country, and also the values of the loads are purely fictitious.

Self-weight:
The self-weight of the purlin trusses and roof is estimated at $g_k = 0{,}5\ kN/m^2$.

Snow load:
The characteristic value of the snow load on the ground in the building area is $s_k = 1{,}5\ kN/m^2$. *In regard to the snow load, the topography of the terrain is 'normal', thus the exposure coefficient has a value* $C_e = 1{,}0$ *(Table 7.1). Thermal coefficient of the roof is* $C_t = 1{,}0$. *The roof angle is 1:6 (*$\alpha = 9{,}5°$*). For a duopitch roof, the shape coefficient is* $\mu_1 = 0{,}8$, *when* $0° \le \alpha_1 = \alpha_2 \le 30°$ *(Figure 7.5). The load arrangements to be considered for a duopitch roof are presented in Figure 7.6b. Herein, as an example, case (i) shall be checked. Hence, the snow load has its full value on both slopes of the roof:*

$$s = \mu_1 C_e C_t s_k = 0{,}8 \cdot 1{,}0 \cdot 1{,}0 \cdot 1{,}5 = 1{,}2\ kN/m^2$$

Wind load:
In the building area, the fundamental value of the basic wind velocity is $v_{b.0}$ = 23 m/s. The directional factor for the wind is c_{dir} = 1,0 and the season factor is c_{season} = 1,0. Thus the basic wind velocity is:

$$v_b = c_{dir} \cdot c_{season} \cdot v_{b.0} = 1,0 \cdot 1,0 \cdot 23 = 23 \text{ m/s}$$

The basic velocity pressure of the wind q_b is calculated from the formula:

$$q_b = \frac{1}{2} \cdot \rho \cdot v_b^2 = \frac{1}{2} \cdot 1,25 \cdot 23^2 = 0,33 \text{ kN/m}^2$$

In terrain category III, the exposure factor c_e has the following value with a building height of h = 14,5 m measured at the ridge level (Figure 7.8): c_e = 1,957

By taking a value $c_s c_d$ = 1,0 for the structural factor, the characteristic wind load can be calculated using expressions (7.2) & (7.4) & (7.7):

$$q_{wk} = \frac{F_{w.e}}{A_{ref}} = c_s c_d \cdot c_e \cdot q_b \cdot c_{pe} = 1,0 \cdot 1,957 \cdot 0,33 \cdot c_{pe} = 0,65 c_{pe} \text{ kN/m}^2$$

The following pressure coefficient values c_{pe} are obtained for the Example building (Figure 7.9 and Table 7.4):

Wind parallel to the long side wall:
h = 14,5 m
b = 48 m
d = 100 m
$e = \min(b, 2h) = \min(48 \text{ m}, 29 \text{ m}) = 29 \text{ m}$

$h \le b$ $\Rightarrow$ *also the windward wall of the building can be calculated as one horizontal strip (Figure 7.10)*
$e < d$ $\Rightarrow$ *Figure 7.9: case 1*
$h/d = 0,145 \le 0,25 \Rightarrow A = -1,2 \quad B = -0,8 \quad C = -0,5 \quad D = +0,7 \quad E = -0,3$

Wind parallel to the short side wall:
h = 14,5 m
b = 100 m
d = 48 m
$e = \min(b, 2h) = \min(100 \text{ m}, 29 \text{ m}) = 29 \text{ m}$

$h \le b$ $\Rightarrow$ *also the windward wall of the building can be calculated as one horizontal strip (Figure 7.10)*
$e < d$ $\Rightarrow$ *Figure 7.9: case 1*
$h/d = 0,30 \cong 0,25 \Rightarrow A = -1,2 \quad B = -0,8 \quad C = -0,5 \quad D = +0,7 \quad E = -0,3$

In Part EN 1991-1-4 of Eurocode, the pressure coefficients are given separately also for the roofs. ***In order to condense the calculations presented herein, the vertical actions due to suction/pressure load of the roof of the Example building have not been considered. In real designing task also those actions need to be considered.***

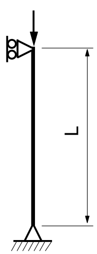
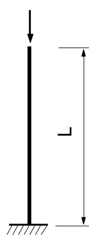
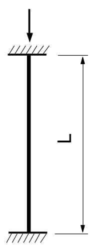
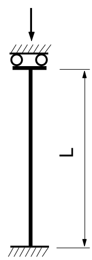
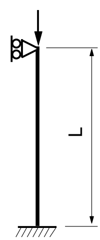
7.2 Column design

As already stated in Chapter 2, a structural hollow section is an excellent cross-section for a column. The mass of a hollow section is located far from its centroid, hence for the hollow section the second moment of area is great in all directions. In column design, the essential factors are the buckling lengths, the impact of the joint stiffness, and the column-to-foundation joint.

7.2.1 Buckling length of a column

Factors affecting the buckling length of a column are the physical length of the column, the restraint conditions of the ends, and the lateral support of the structure. Theoretical buckling lengths of columns are presented in Table 7.6.

Table 7.6 Theoretical buckling lengths of columns for basic cases

Pinned at both ends	Fixed at one end	Fixed at both ends	Fixed at both ends, one sway joint	Fixed at one end, pinned at the other
				
$L_{cr} = 1{,}0\ L$	$L_{cr} = 2{,}0\ L$	$L_{cr} = 0{,}5\ L$	$L_{cr} = 1{,}0\ L$	$L_{cr} = 0{,}7\ L$

In frame structures with rigid joints, the stiffening effect of the beams can be utilised when determining the buckling length of the columns. Another factor affecting the buckling length in frame structures is the lateral support of the frame. The frame may be classified as sway or non-sway. A non-sway structure is stiffened using either trusses or supporting it to a rigid structural member (elevator shaft or stairwell). In a non-sway frame, the additional forces and moments caused by the horizontal displacements of the frame (i.e. second-order effects) are so small, that they may be neglected in the calculations. The frame is non-sway, if the following condition is satisfied [14,15,16]:

$$\alpha_{cr} = \frac{F_{cr}}{F_{Ed}} \geq 10 \qquad \textit{for elastic analysis} \tag{7.13}$$

where α_{cr} is the factor by which the design load F_{Ed} should be multiplied to cause elastic instability in a global mode
F_{Ed} is the design load of the structure
F_{cr} is the elastic critical buckling load for global instability mode based on initial elastic stiffness

The stiffness of a sway structure is based on the columns functioning as masts that are fixed to the foundation with a rigid joint, or on the stiffness of the joints.

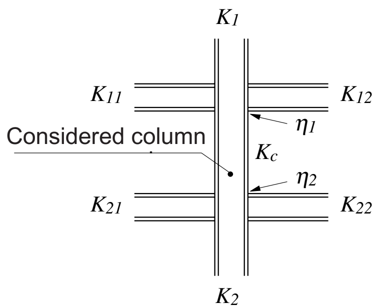
In the case of a continuous column, the buckling length can be determined using Figures 7.12 and 7.13. In the graphs, the values of the curves are the ratios of the buckling lengths to the actual column length. The distribution factors η_1 and η_2 of the moments, which are needed in the figures, are calculated using the following formulae [24,25]:

$$\eta_1 = \frac{K_c + K_1}{K_c + K_1 + K_{11} + K_{12}} \qquad \textit{upper joint} \tag{7.14}$$

$$\eta_2 = \frac{K_c + K_2}{K_c + K_2 + K_{21} + K_{22}} \qquad \textit{lower joint} \tag{7.15}$$

where

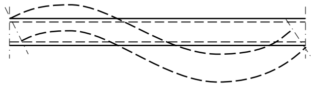
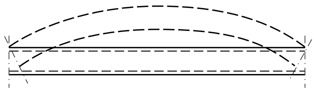
$K_c = I / L$ is the stiffness coefficient for the column to be analyzed
$K_1 = I_1 / L_1$ is the stiffness coefficient for the column above
$K_2 = I_2 / L_2$ is the stiffness coefficient for the column below
I, I_1 and I_2 are second moments of area for the corresponding columns in the direction of the frame



L, L_1 and L_2 are column lengths correspondingly
K_{11}, K_{12}, K_{21} and K_{22} are the effective beam stiffness coefficients (Table 7.7)

Table 7.7 can be used provided that the stresses in the beam remain in the elastic region ($M_{Ed} \leq W_{el} \cdot f_y / \gamma_{M0}$). The beam is supposed pinned at the joints, if the maximum bending moment exceeds the elastic moment resistance [24,25].

Table 7.7 Effective stiffness coefficient K for a beam [24,25]

Conditions of rotational restraint at the far end of the beam	Effective stiffness coefficient
Fixed	$1{,}0\ I_b / L_b\ (1\text{-}0{,}4\ N / N_e)$
Pinned	$0{,}75\ I_b / L_b\ (1\text{-}1{,}0\ N / N_e)$
Rotation as at near end (double curvature) 	$1{,}5\ I_b / L_b\ (1\text{-}0{,}2\ N / N_e)$
Rotation equal and opposite to that of near end (single curvature) 	$0{,}5\ I_b / L_b\ (1\text{-}1{,}0\ N / N_e)$

I_b is the beam's second moment of area in the frame direction
L_b is the length of the beam
N is the compressive force of the beam
$N_e = \pi^2 \cdot E \cdot I_b / L_b^2$

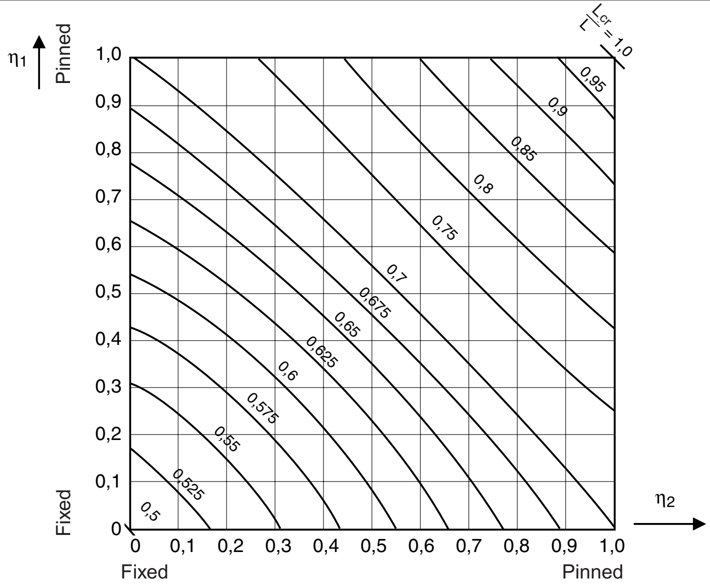


Figure 7.12 Buckling length ratio L_{cr}/L for a column in a non-sway mode [24,25]

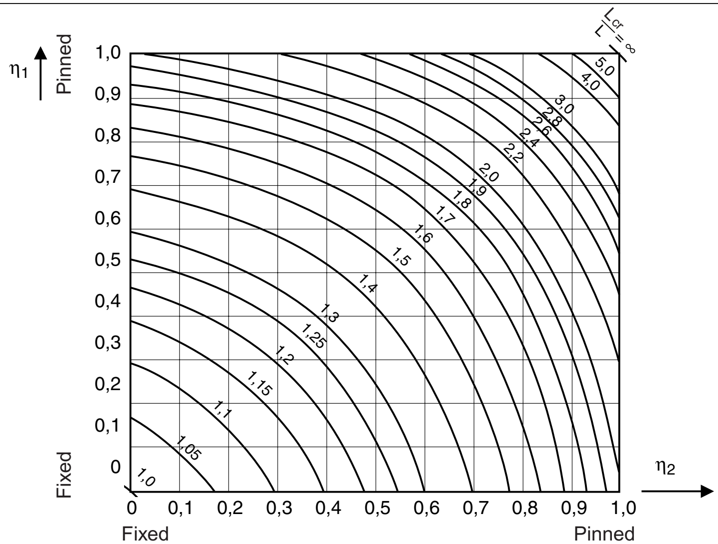


Figure 7.13 Buckling length ratio L_{cr}/L for a column in a sway mode [24,25]

7.2.2 Effect of joint stiffness on column buckling length

The joints of a frame structure may be considered rigid (fixed), if the joints are stiffened as shown in Figure 7.14. A non-stiffened joint should be considered semi-rigid or pinned (simple) when determining the buckling length of the column. The calculation of the stiffness of the non-stiffened hollow section frame joints has been presented in Annex 11.4. The effect of a semi-rigid joint on the column buckling length (formulae 7.14 and 7.15) is taken into account in the effective second moment of area of the beam, which is calculated from the following expression [26]:

$$I_{b.eff} = \frac{1}{1 + \dfrac{3EI_b}{S_j L_b}} \cdot I_b \qquad (7.16)$$

where S_j is the rotational stiffness of the joint [Nm/rad]
I_b is the second moment of area of the beam in the direction of the frame
L_b is the length of the beam
E is the Young's modulus of elasticity

The stiffness of the joint varies according to the applied moment, as the increasing moment causes the plastification of the most stressed parts of the joint. The entire moment-rotation curve should thus be known in order to utilise the effect of semi-rigid joints on the calculation of the column buckling length. In case this is not known, it is conservative to assume that the support of the column is pinned at the joint.

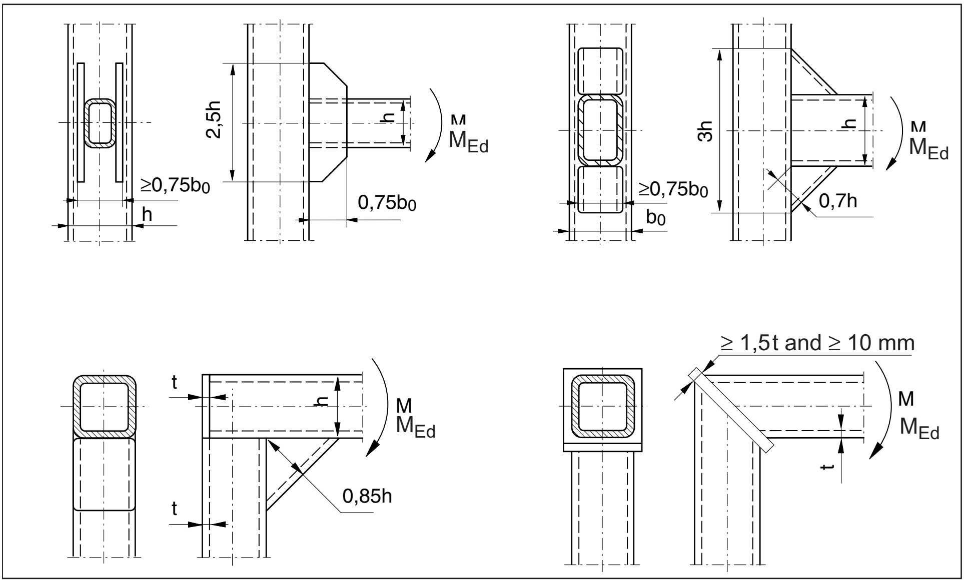


Figure 7.14 Stiffened frame joints of structural hollow sections

7.2.3 Joint of the column to the foundation

The joint of the column to the foundation has been discussed in clause 3.5. If a rigid column-to-foundation connection is assumed in the structural model, the bending moments transferred by the column into the foundation must be considered when designing the anchor bolts and the base plate. If a pinned joint is assumed in the model, the bending moments need not be taken into account.

In the erection situation the anchor bolts should be designed to carry the erection-phase loads of the column. The buckling length of the anchor bolts should be taken as the thickness of the grouting layer (Table 7.6: case 4).

7.2.4 Designing the column of the Example building

Design the columns of the primary frames. The column-to-foundation joint is in this case assumed rigid. Hence, the moment transferred from the column to the foundation must be taken into account when designing the joint. The horizontal loads on the building are carried by the bracing trusses, so the top of the columns is non-sway and pinned. The buckling length of the columns can thus be obtained directly from Table 7.6, giving $L_{cr.y} = L_{cr.z} = 0{,}7 \cdot 10{,}3 = 7{,}21$ *m.*

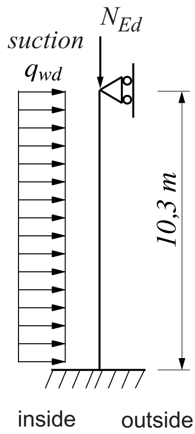
Calculation of loads:
The loads on the columns are determined simply by the loaded area of the wall of the building.

In Finland, as well as in certain other countries, the National Annex stipulates to determine the load combination according to expressions (2.4a) and (2.4b) as presented in Chapter 2, from which always the more unfavourable is selected [2]. However, in order to keep the calculations in this Example simple, the load combination is determined herein by using expression (2.3) for ultimate limit state.

The partial safety factors for loads are obtained from Table 2.1. Since in our case the building is an exhibition hall where fairs and conferences are organized, it shall be classified in consequences class CC3. Consequently, load factor K_{FI} *shall be taken as* $K_{FI} = 1{,}1$ *(Tables 2.2 and 2.3).*

The load combinations and corresponding resistances have to be checked for two different cases, since it is not possible to know in advance which one of the loads, snow or wind, will be the governing load.

In case the wind load is governing:
(wind parallel to the long side wall, suction zone B):



$\gamma_{Q.1} = 1,5 \cdot K_{FI}$ *wind*

$q_{wk} = 0,65c_{pe.B} = 0,65 \cdot (-0,8) = -0,52\ kN/m^2$ *suction*

$q_{wd} = \gamma_{Q.1} \cdot q_{wk} \cdot L_f = (1,5 \cdot 1,1) \cdot 0,52 \cdot 5 = 4,29\ kN/m$

$M_{Ed} = 4,29 \cdot \dfrac{10,3^2}{8} = 56,9\ kNm$ *at lower end of the column*

$V_{Ed} = \dfrac{5}{8} \cdot 4,29 \cdot 10,3 = 27,6\ kN$ *at lower end of the column*

$\gamma_{Q.2} = 1,5 \cdot K_{FI}$ *snow*

$s_d = \gamma_{Q.2} \cdot \psi_{0.2} \cdot s = (1,5 \cdot 1,1) \cdot 0,7 \cdot 1,2 = 1,39\ kN/m^2$

$\gamma_G = 1,35 \cdot K_{FI}$ *self-weight*

$g_d = \gamma_G \cdot g_k = (1,35 \cdot 1,1) \cdot 0,5 = 0,743\ kN/m^2$

$N_{Ed} = 0,5(g_d + s_d)L_f \cdot B = 0,5 \cdot (0,743 + 1,39) \cdot 10 \cdot 48 = 511,9\ kN$

where

L_f *is the frame spacing*
B *is the width of the building*

In case the snow load is governing:

$q_{wd} = \gamma_{Q.2} \cdot \psi_{0.2} \cdot q_{wk} \cdot L_f = (1,5 \cdot 1,1) \cdot 0,6 \cdot 0,52 \cdot 5 = 2,57\ kN/m$ *wind load*

$M_{Ed} = 2,57 \cdot \dfrac{10,3^2}{8} = 34,1\ kNm$ *at lower end of the column*

$V_{Ed} = \dfrac{5}{8} \cdot 2,57 \cdot 10,3 = 16,5\ kN$ *at lower end of the column*

$s_d = \gamma_{Q.1} \cdot s = (1,5 \cdot 1,1) \cdot 1,2 = 1,98\ kN/m^2$ *snow load*

$g_d = \gamma_G \cdot g_k = (1,35 \cdot 1,1) \cdot 0,5 = 0,743\ kN/m^2$ *self-weight*

$N_{Ed} = 0,5(g_d + s_d)L_f \cdot B = 0,5 \cdot (0,743 + 1,98) \cdot 10 \cdot 48 = 653,5\ kN$

Resistances at ultimate limit state, when the snow load is governing:

$N_{Ed} = 653{,}5\ kN$ *compression*
$M_{Ed} = 34{,}1\ kNm$ *at lower end of the column*
$V_{Ed} = 16{,}5\ kN$ *at lower end of the column*

Try a hollow section with dimensions 300×200×6. The steel grade is SSAB Domex Tube Double Grade, which fulfills the EN 10219 requirements for both steel grades S420MH and S355J2H. Thereby the design calculations may be performed at designer's own choice either according to grade S420 or grade S355. Grade S420 is chosen in this Example as design basis.

The procedure how to verify the resistance of the cross-section and the resistance of the member, taking into account the combined effects as necessary, has already been presented in Chapter 2. Therefore only the final results and a condensed version of the calculations is presented herein.

Cross-section resistance:

Shear resistance:

$$\frac{h}{t} = \frac{300}{6} = 50,0 \leq \frac{72\varepsilon}{\eta} + 3 = \frac{72 \cdot \sqrt{235/420}}{1,0} + 3 = 56,9 \qquad (clause\ 2.7.1)$$

⇒ shear buckling does not reduce the shear resistance, which thereby shall be determined according to the plastic shear resistance:

$V_{pl.Rd} = 838{,}5\ kN$ *(Annex 11.1)*

$V_{Ed} = 16,5\ kN \leq V_{pl.Rd} = 838,5\ kN$ *OK*

$V_{Ed} = 16,5\ kN \leq 0,5V_{pl.Rd} = 0,5 \cdot 838,5 = 419,3\ kN \Rightarrow$ *shear force does not reduce the bending resistance*

Since shear force does not reduce the bending resistance (clause 2.9.1.6.1), the resistance of the cross-section to the combined effect of the forces can be verified according to clause 2.9.1.5 for the combined effect of (M+N) only.

Combined effects are assessed often by taking the single force components (N_{Ed}, $M_{y.Ed}$ etc.) and corresponding single resistances (N_{Rd}, $M_{y.Rd}$ etc.) after which these characteristics are then applied to check the combined effect. In using such procedure, the cross-section Class is determined separately for each force component using the assumption that the considered force component is acting alone (for example, see design condition (2.1.35)).

Next, check the cross-section classification of the hollow section, and the effectiveness of its plane elements, for different force components (i.e. for different stress states of the plane elements):

If subjected to compression only:
flange:

$b/t = 200/6 = 33,3 \leq 34,4 \qquad \Rightarrow$ *Class 3*

web:

$h/t = 300/6 = 50,0 > 34,4 \qquad \Rightarrow$ *Class 4*

⇒ the whole cross-section shall be classified into Class 4

Nevertheless, the web 300 mm is fully effective in bending about the y-axis, because h/t = 300/6 = 50,0 ≤ 95,8 (Table 2.9). Correspondingly, it is easy to conclude that the flange 200 mm (< 300 mm) is fully effective in bending about the z-axis.

(M+N) interaction according to 2.9.1.5.3:

$A_{eff} = 4830\ mm^2$ *if subjected to compression only (clause 2.4.1)*
$W_{eff.y} = W_{el.y} = 491{,}4 \cdot 10^3\ mm^3$ *if subjected to bending only (Annex 11.1)*
$f_y = 420\ N/mm^2$
$\gamma_{M0} = 1{,}0$

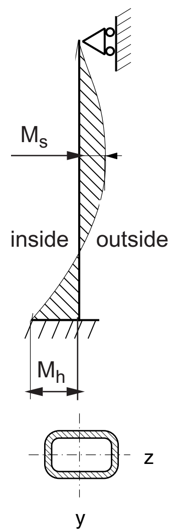
$$\frac{N_{Ed}}{A_{eff} \cdot f_y / \gamma_{M0}} + \frac{M_{y.Ed} + N_{Ed}\, e_{Ny}}{W_{eff.y} \cdot f_y / \gamma_{M0}} + \frac{M_{z.Ed} + N_{Ed}\, e_{Nz}}{W_{eff.z} \cdot f_y / \gamma_{M0}} =$$

$$\frac{653{,}5 \cdot 10^3}{4830 \cdot 420 / 1{,}0} + \frac{34{,}1 \cdot 10^6 + 0}{491{,}4 \cdot 10^3 \cdot 420 / 1{,}0} + 0 = 0{,}3221 + 0{,}1652 + 0 = 0{,}4873 \le 1{,}0 \quad OK$$

<u>Member resistance</u>:

$N_{b.y.Rd} = 1304\ kN$ *(in regard to y-axis,* $\chi_y = 0{,}6427$*)*
$N_{b.z.Rd} = 947{,}8\ kN = N_{b.Rd}$ *(in regard to z-axis,* $\chi_z = 0{,}4672$*)*

Since the preceding cross-section verification regarding (M+N) interaction has been carried out according to the fully effective elastic bending resistance (i.e. Class 3), also the lateral-torsional buckling resistance of the member shall be assessed according to Class 3, respectively. According to Table 2.18, lateral-torsional buckling does not reduce the bending resistance in case of Class 3. Thus, when checking the member against (M+N) interaction, the reduction factor χ_{LT} *for lateral-torsional buckling resistance can be taken as* $\chi_{LT} = 1{,}0$*:*



$\psi = 0$ *(Table 2.25)*

$M_h = 34{,}1\ kNm$

$$M_s = \frac{9}{128} \cdot q_{wd} L^2 = \frac{9}{128} \cdot 2{,}57 \cdot 10{,}3^2 = 19{,}2\ kNm \quad (simplification)$$

$$\alpha_s = \frac{M_s}{M_h} = -\frac{19{,}2}{34{,}1} = -0{,}5630$$

$$C_{my} = 0{,}1 - 0{,}8\alpha_s = 0{,}1 - 0{,}8 \cdot (-0{,}5630) = 0{,}5504 \ge 0{,}4$$

from Table 2.23:

$$k_{yy} = C_{my}\left[1 + 0{,}6\,\bar{\lambda}_y \frac{N_{Ed}}{\chi_y \cdot N_{Rk}/\gamma_{M1}}\right] = 0{,}5504 \cdot \left[1 + 0{,}6 \cdot 0{,}8309 \cdot \frac{653{,}5 \cdot 10^3}{0{,}6427 \cdot 4830 \cdot 420/1{,}0}\right]$$
$$= 0{,}6879$$

$$k_{yy} \le C_{my}\left[1 + 0{,}6 \cdot \frac{N_{Ed}}{\chi_y \cdot N_{Rk}/\gamma_{M1}}\right] = 0{,}5504 \cdot \left[1 + 0{,}6 \cdot \frac{653{,}5 \cdot 10^3}{0{,}6427 \cdot 4830 \cdot 420/1{,}0}\right] = 0{,}7159$$

$\Rightarrow k_{yy} = 0{,}6879$

$$k_{zy} = 0,8\,k_{yy} = 0,8 \cdot 0,6879 = 0,5503$$

$$\frac{N_{Ed}}{\chi_y \dfrac{N_{Rk}}{\gamma_{M1}}} + k_{yy}\frac{M_{y.Ed} + \Delta M_{y.Ed}}{\chi_{LT}\dfrac{M_{y.Rk}}{\gamma_{M1}}} + k_{yz}\frac{M_{z.Ed} + \Delta M_{z.Ed}}{\dfrac{M_{z.Rk}}{\gamma_{M1}}} =$$

$$\frac{653,5 \cdot 10^3}{0,6427 \cdot \dfrac{4830 \cdot 420}{1,0}} + 0,6879 \cdot \frac{34,1 \cdot 10^6 + 0}{1,0 \cdot \dfrac{491,4 \cdot 10^3 \cdot 420}{1,0}} + 0 =$$

$$0,5012 + 0,1137 = 0,6149 \le 1,0 \quad OK$$

$$\frac{N_{Ed}}{\chi_z \dfrac{N_{Rk}}{\gamma_{M1}}} + k_{zy}\frac{M_{y.Ed} + \Delta M_{y.Ed}}{\chi_{LT}\dfrac{M_{y.Rk}}{\gamma_{M1}}} + k_{zz}\frac{M_{z.Ed} + \Delta M_{z.Ed}}{\dfrac{M_{z.Rk}}{\gamma_{M1}}} \le 1,0$$

$$\frac{653,5 \cdot 10^3}{0,4672 \cdot \dfrac{4830 \cdot 420}{1,0}} + 0,5503 \cdot \frac{34,1 \cdot 10^6 + 0}{1,0 \cdot \dfrac{491,4 \cdot 10^3 \cdot 420}{1,0}} + 0 =$$

$$0,6895 + 0,0909 = 0,7804 \le 1,0 \quad OK$$

So, the resistance of the hollow section 300×200×6 is sufficient, when the snow load is governing.

Comparison S420 vs S355:
In case the design calculations would be performed according to grade S355, the calculations would follow basically the same principles, and the last (M+N) interaction verification would give the result 0,8365 (S420: 0,7804). By comparing the 'utilisation ratios' of the interaction formula, we can see that in this Example the increase of the material strength S355 → S420 improves the overall resistance approximately 7 %.

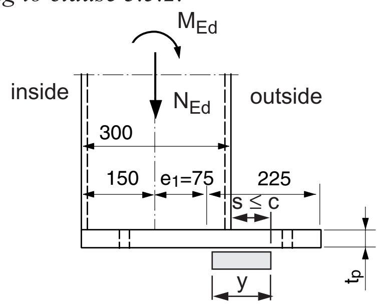
Resistances at ultimate limit state, when the wind load is governing:
The resistances of the cross-section and the member must be checked for this load combination respectively.

7.2.5 Designing the column-to-foundation joint of the Example building

The column-to-foundation joint is constructed as shown in the adjacent Figure. Since the joint was assumed rigid, it shall be designed according to clause 3.5.2.

Loads, in case the snow load is governing:

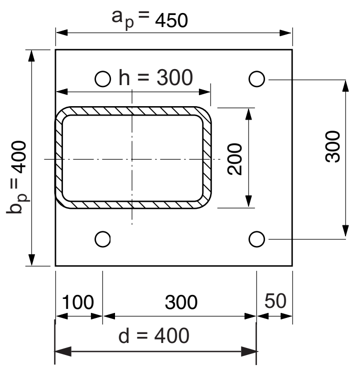
The column-to-foundation joint is subjected to the following loads (calculated already earlier):



N_{Ed} = 653,5 kN compression
M_{Ed} = 34,1 kNm at lower end of the column
V_{Ed} = 16,5 kN at lower end of the column

In addition, the resistance during the erection phase shall be checked separately:

the design load during the erection phase
$N_{Ed.min} = 162\ kN$ *(compression)*



Design of the base plate:

The concrete strength class is C25/30 for the foundation. Thus the design strength of concrete's bearing pressure is $f_{jd} = 11{,}3\ N/mm^2$ *(Example 3.17). Try a base plate having* $t_p = 30\ mm$ *(S355).*

Extension of the bearing area is calculated from expression (3.74):

$$c = t_p\sqrt{\frac{f_{yp}/\gamma_{M0}}{3f_{jd}}} = 30 \cdot \sqrt{\frac{355/1,0}{3 \cdot 11,3}} = 97,1\ mm \quad \Rightarrow \text{rounded up to } c = 100\ mm$$

$$b - 2t = 200 - 2 \cdot 6 = 188\ mm \le 2c = 2 \cdot 100 = 200\ mm$$

$\Rightarrow$ *the bearing area inside the perimeter of the hollow section is fully effective*

$$b_p = 400\ mm \le b + 2c = 200 + 2 \cdot 100 = 400\ mm$$

$\Rightarrow$ *outside the hollow section, in direction of dimension* b_p*, the base plate is fully effective*
$\Rightarrow l_{eff} = b_p = 400\ mm$

$$a_p - h = 450 - 300 = 150\ mm > c = 100\ mm$$

$\Rightarrow$ *outside the hollow section, in direction of dimension* a_p*, the base plate is effective only up to the distance* $c = 100\ mm$

Next, based on compression and the design strength of concrete's bearing pressure, check the resistance of the base plate. The bearing pressure is assumed to distribute uniformly under the base plate (a conservative assumption) within the entire effective area limited by measure c. The bending moment in the base plate at point of the column flange, caused by the bearing pressure of the concrete, is calculated from expression (3.68):

$$s = c = 97,1\ mm \quad \text{(effective extension of the base plate, calculated already earlier)}$$

$$M_{p.Ed} = \frac{l_{eff}s^2 f_{jd}}{2} = \frac{400 \cdot 97,1^2 \cdot 11,3}{2} = 21,3\ kNm$$

$$t_p \ge \sqrt{\frac{6M_{p.Ed}}{l_{eff} \cdot f_{yp}/\gamma_{M0}}} = \sqrt{\frac{6 \cdot 21,3 \cdot 10^6}{400 \cdot 355/1,0}} = 30,0 \quad \Rightarrow t_p = 30\ mm \quad OK$$

So, thickness of the base plate $t_p = 30\ mm$ *(S355) is sufficient.*

Design of the anchor bolts:

First, check whether the anchor bolts will be subjected to tension when assuming the ultimate limit state.

The depth y of the concrete area in compression can be determined by writing a moment equilibrium condition about the inside bolt line, which is the assumed tensile side of the base plate (cf. formula (3.79)):

$$M_{Ed} + N_{Ed}(300/2 - 100) = l_{eff}\,y\,f_{jd}[(300 - 100) + 100 - 0,5y] \quad \Rightarrow$$

$$y = \frac{l_{eff}f_{jd} \cdot 300 \pm \sqrt{(-l_{eff}f_{jd} \cdot 300)^2 - 2l_{eff}f_{jd}[M_{Ed} + N_{Ed} \cdot 50]}}{l_{eff}f_{jd}}$$

$$= \frac{400 \cdot 11,3 \cdot 300 \pm \sqrt{(-400 \cdot 11,3 \cdot 300)^2 - 2 \cdot 400 \cdot 11,3 \cdot [34,1 \cdot 10^6 + 653,5 \cdot 10^3 \cdot 50]}}{400 \cdot 11,3}$$

$$= 54,1 \text{ mm (or } 545,9 \text{ mm)}$$

The axial force in the bolts is determined from the vertical equilibrium condition by formula (3.77):

$$N_c = l_{eff} \cdot y \cdot f_{jd} = 400 \cdot 54,1 \cdot 11,3 = 244,5 \text{ kN} \qquad \text{(compression)}$$

$$N_s = N_c - N_{Ed} = 244,5 - 653,5 = -409,0 \text{ kN} \qquad \text{(compression)}$$

$\Rightarrow$ *no tension in the anchor bolts*

If the anchor bolts would be subjected to tension, the positioning of the bolts should fulfill the geometrical conditions presented in clause 3.5.2.

Since the anchor bolts are not subjected to tension, only shear force needs to be considered in design. Usually the size of the anchor bolts is between M24 - 36 mm, but at least M20 as the minimum. Choose ribbed steel bolts 4×M30 according to the table of the Manufacturer. The normal force resistance specified by the Manufacturer takes into account also the anchorage resistance. If the applied concrete differs from the strength class specified in the Manufacturer's table, a correction factor has to be applied. The Designer of the foundation designs also the reinforcements for the foundation, wherein the chosen anchor bolt type has an impact.

The Manufacturer's table is based on concrete strength class C25/30, thus the resistances given in the table can be directly adopted for the bolts.

Tension resistance of the bolts:
According to the Manufacturer's table the tension resistance of the bolt is:

$$N_{t.Rd} = 222,1 \text{ kN} \qquad \text{OK (the bolts are not subjected to tension)}$$

Shear resistance of the bolts:
The shear force V_{Ed} is uniformly distributed to all four bolts. The shear force per bolt is thereby:

$$V_{Ed} = 16,5/4 = 4,1 \text{ kN}$$

According to the Manufacturer's table the shear resistance of the chosen bolt is:

$$F_{v.Rd} = 27,2 \text{ kN} \geq 4,1 \text{ kN} \qquad \text{OK}$$

Combined effect of normal force and shear force:
The combined effect does not need to be checked in the finished foundation.

The 300 mm centre-to-centre distance of the bolts satisfies the minimum distance declared by the Manufacturer. Also the required minimum edge distances have to be checked, and appropriate deductions for the resistances shall be carried out according to the Manufacturer's instructions, when needed.

In addition, the resistance of the anchor bolts to the loads applied during the erection phase has to be checked (possible flexural buckling of the bolts).

Resistance of the anchor bolts during the erection phase:

During the erection, the anchor bolts shall carry alone all the loads that are applied to the column. For the outside bolts (= the most stressed bolts), the vertical load per bolt can be determined by writing a moment equilibrium condition about the inside bolt line. Thus the following is obtained for the erection phase:

$$2 \cdot N_{b.Ed} \cdot 300 = M_{Ed} + N_{Ed.min} \cdot (h/2 - 100) \quad \Rightarrow$$

$$N_{b.Ed} = \frac{M_{Ed} + N_{Ed.min} \cdot (h/2 - 100)}{2 \cdot 300} = \frac{34,1 \cdot 10^6 + 162 \cdot 10^3 \cdot (300/2 - 100)}{2 \cdot 300} = 70,3\ kN$$

The thickness of the grouting layer is chosen to be u = 70 mm. The loads in the governing bolt are then:

$$N_{b.Ed} = 70,3\ kN \quad \text{(compression)}$$

$$V_{b.Ed} = 4,1\ kN$$

$$M_{b.Ed} = \frac{V_{b.Ed} \cdot u}{2} = \frac{4,1 \cdot 0,07}{2} = 0,14\ kNm \text{ (at lower and upper end of buckling length)}$$

Shear resistance of the threaded portion of the bolt:

$$f_{yb} = 500\ N/mm^2 \quad \text{(ribbed steel A500HW)}$$

$$f_{ub} = 550\ N/mm^2 \quad \text{(ribbed steel A500HW)}$$

$$A_s = 561\ mm^2 \quad \text{tensile stress area of a M30 bolt (area in the threaded portion)}$$

$$F_{v.Rd} = \frac{0,5 f_{ub} A_s}{\gamma_{M2}} = \frac{0,5 \cdot 550 \cdot 561}{1,25} = 123,4\ kN \geq V_{b.Ed} = 4,1\ kN \quad OK$$

Bending resistance of the bolt:

$$d_s = 2 \cdot \sqrt{\frac{A_s}{\pi}} = 2 \cdot \sqrt{\frac{561}{\pi}} = 26,7\ mm$$

$$I_b = \frac{\pi d_s^4}{64} = \frac{\pi \cdot 26,7^4}{64} = 2,49 \cdot 10^4\ mm^4$$

$$W_{b.el} = \frac{I_b}{d_s/2} = \frac{2,49 \cdot 10^4}{26,7/2} = 1,87 \cdot 10^3$$

$$M_{b.Rd} = \frac{W_{b.el} f_{yb}}{\gamma_{M0}} = \frac{1,87 \cdot 10^3 \cdot 500}{1,0} = 0,94\ kNm \geq M_{b.Ed} = 0,14\ kNm \quad OK$$

Assessment of the stresses in the bolt:

$$\sigma = \frac{N_{b.Ed}}{A_s} + \frac{M_{b.Ed}}{W_{b.el}} = \frac{70 \cdot 10^3}{561} + \frac{0,14 \cdot 10^6}{1,87 \cdot 10^3} = 200\ N/mm^2 \leq f_{yb} = 500\ N/mm^2 \quad OK$$

$$\tau = \frac{V_{b.Ed}}{A_s} = \frac{4,1 \cdot 10^3}{561} = 7,3\ N/mm^2 \leq \frac{f_{yb}}{\sqrt{3}} = 289\ N/mm^2 \quad OK$$

$$\sqrt{\sigma^2 + 3\tau^2} = \sqrt{200^2 + 3 \cdot 7,3^2} = 200,4\ N/mm^2 \leq f_{yb} = 500\ N/mm^2 \quad OK$$

Buckling resistance of the bolt:
the buckling length of the bolt is:

$$L_{cr} = 1,0 \cdot u = 1,0 \cdot 70 = 70\ mm \quad (Table\ 7.6\text{: case 4})$$

$$I_b = 2,49 \cdot 10^4\ mm^4 \quad \Rightarrow \bar{\lambda} = 0,1632 \leq 0,2 \quad \Rightarrow \chi = 1,0$$

$$N_{b.Rd} = \frac{\chi A_s f_{yb}}{\gamma_{M1}} = \frac{1,0 \cdot 561 \cdot 500}{1,0} = 280,5\ kN \geq N_{b.Ed} = 70,3\ kN \quad OK$$

Combined effect of buckling and bending:

$$\psi = -1 \quad (Table\ 2.25)$$

$$C_{my} = 0,6 + 0,4\psi = 0,6 + 0,4 \cdot (-1) = 0,2 \quad but\ C_{my} \geq 0,4$$

$$\Rightarrow C_{my} = 0,4$$

Interaction factors from Table 2.23 (circular cross-section is not susceptible to torsional deformations):

$$k_{yy} = C_{my}\left[1 + 0,6\,\bar{\lambda}_y \frac{N_{Ed}}{\chi_y \cdot N_{Rk}/\gamma_{M1}}\right] = 0,4 \cdot \left[1 + 0,6 \cdot 0,1632 \cdot \frac{70,3 \cdot 10^3}{1,0 \cdot 561 \cdot 500/1,0}\right]$$

$$= 0,4098$$

$$k_{yy} \leq C_{my}\left[1 + 0,6 \cdot \frac{N_{Ed}}{\chi_y \cdot N_{Rk}/\gamma_{M1}}\right] = 0,4 \cdot \left[1 + 0,6 \cdot \frac{70,3 \cdot 10^3}{1,0 \cdot 561 \cdot 500/1,0}\right] = 0,4601$$

$$\Rightarrow k_{yy} = 0,4098$$

$$k_{zy} = 0,8k_{yy} = 0,8 \cdot 0,4098 = 0,3278$$

$$\frac{N_{Ed}}{\chi_y \frac{N_{Rk}}{\gamma_{M1}}} + k_{yy}\frac{M_{y.Ed} + \Delta M_{y.Ed}}{\chi_{LT}\frac{M_{y.Rk}}{\gamma_{M1}}} + k_{yz}\frac{M_{z.Ed} + \Delta M_{z.Ed}}{\frac{M_{z.Rk}}{\gamma_{M1}}} =$$

$$\frac{70,3 \cdot 10^3}{1,0 \cdot \frac{561 \cdot 500}{1,0}} + 0,4098 \cdot \frac{0,14 \cdot 10^6 + 0}{1,0 \cdot \frac{1,87 \cdot 10^3 \cdot 500}{1,0}} + 0 =$$

$$0,2506 + 0,0614 = 0,3120 \le 1,0 \quad OK$$

$$\frac{N_{Ed}}{\chi_z \frac{N_{Rk}}{\gamma_{M1}}} + k_{zy}\frac{M_{y.Ed} + \Delta M_{y.Ed}}{\chi_{LT}\frac{M_{y.Rk}}{\gamma_{M1}}} + k_{zz}\frac{M_{z.Ed} + \Delta M_{z.Ed}}{\frac{M_{z.Rk}}{\gamma_{M1}}} \le 1,0$$

$$\frac{70,3 \cdot 10^3}{1,0 \cdot \frac{561 \cdot 500}{1,0}} + 0,3278 \cdot \frac{0,14 \cdot 10^6 + 0}{1,0 \cdot \frac{1,87 \cdot 10^3 \cdot 500}{1,0}} + 0 =$$

$$0,2506 + 0,0491 = 0,2997 \le 1,0 \quad OK$$

Design of the welds:

Welds in the column's flange:
First, check whether the welds shall be subjected to tension. On the tension side of the bending moment, the stress at the midline of the column wall thickness is the following:

$$\sigma_x = \frac{M_{Ed}}{I_y} \cdot \frac{h-t}{2} - \frac{N_{Ed}}{A} = \frac{34,1 \cdot 10^6}{7370 \cdot 10^4} \cdot \frac{300-6}{2} - \frac{653,5 \cdot 10^3}{5763} = 68,0 - 113,4 = -45,4\ kN$$

$\Rightarrow$ the bending moment is not big enough to cause tensile stress to the column and the welds in it.

Compressive stresses can be transmitted to foundation using contact bearing. If so, the welds may be designed only for tensile and shear forces. In such case there must be full contact achieved between the base plate and the column end. This shall be presented in the execution specification/drawings of the column.

Welds in the column's web:
While transmitting the compressive stresses by full bearing contact, the welds in the web need to be designed for the shear forces only.

When designing the welds, the ultimate tensile strength for the weld shall be adopted according to the weaker material to be joined, and the strength factor β_w respectively [18,19,20]:

$$f_{uw} = min[f_u ; f_{up}] = min[500 ; 490] = 490\ N/mm^2 \quad (= S355)$$

$$\beta_w = 0,9 \quad \text{(strength factor of the weld, Table 3.8)} \quad (= S355)$$

Try a throat thickness a = 3 mm which is the smallest permitted throat thickness for a load carrying fillet weld (see clause 3.3.3):

$$\tau_{||} = \frac{V_{Ed}}{2ah} = \frac{16,5 \cdot 10^3}{2 \cdot 3 \cdot 300} = 9,2 \text{ N/mm}^2$$

When the weld is subjected to shear force only, the design condition for the weld is (the ultimate tensile strength for the weld shall be adopted according to the weaker material to be joined):

$$\sqrt{\sigma_{\perp}^2 + 3(\tau_{\perp}^2 + \tau_{||}^2)} = \sqrt{0 + 3 \cdot (0 + 9,2^2)} = 15,9 \text{ N/mm}^2$$

$$\leq \frac{f_{uw}}{\beta_w \gamma_{M2}} = \frac{490}{0,9 \cdot 1,25} = 435,6 \text{ N/mm}^2 \quad OK$$

Considering the cooling rate of the weld, the following is required (see clause 3.3.3):

$$a \geq \sqrt{t}\text{ mm} - 0,5\text{ mm} = \sqrt{30} - 0,5 = 5,0 \text{ mm} \qquad (t = \text{thicker of the plates to be joined})$$

$\Rightarrow$ a = 5 mm shall be chosen around the entire column. In the inner side of the base plate, the weld needs to be designed as a half V-groove weld instead of a fillet weld.

The concrete foundation shall be designed according to Part EN 1992-1-1 of Eurocode.

Comparison S420 vs S355:
The assessments carried out in this Example are focused on the base plate and the anchor bolts. Therefore the steel grade of the hollow section column may have influence only on the design of the welds between the column and the base plate. In respect to the welds however, in the calculation procedure in this Example it is the base plate that is governing, no matter which steel grade (S355J2H or S420MH) shall be chosen as design basis for the hollow section column

Verifications for resistance when the wind load is governing:
The resistance of the anchor bolts and the base plate shall be checked in regard to this load combination, respectively.

7.3 Beam design

When designing a beam, the use of plastic theory is recommended whenever possible. Plastic theory may be used when calculating the resistances in cross-section Classes 1 and 2. When calculating the forces and moments, plastic theory may be used only in Class 1.

Hollow sections of rectangular shape are an efficient solution when the normal force in the beam is small compared to the bending moment and the bending moment is uniaxial. Even with hollow sections having a high h/b ratio, the lateral-torsional buckling resistance is seldom governing in the design (see clause 2.6.5).

By taking into account the continuity of the beam, which reduces the field moments, it is often possible to select a smaller hollow section size. It is thus advantageous to use as long continuous beams as possible. However, the aspects regarding workshop fabrication, transportation and erection on site should be taken into account in the length of the members. By placing the splices appropriately, it is possible to keep the forces in the joints small, thus also a pinned joint becomes possible.

The beam end is recommended to be stiffened using an end-plate, to transfer the support reaction from the webs to the support without the deformation of the cross-section (Figure 7.15a). The corner rounding in the hollow section causes a risk of local buckling in bending also at the intermediate support, if the gap under the rounding is not filled with a weld (Figure 7.15b). The cross-section of a hollow section tends to become distorted, if it is subjected to torsional moment. The cross-sectional distortion can be prevented if the torsional moment is transferred to the beam according to Figure 7.15c.

The semi-rigidity of the joints can be taken into account when designing the beams in a frame structure. However, in such a case, the moment-rotation curve of the joint must be known, since the stiffness of the joint varies according to the joint moment. Reference [26] and Annex 11.4 present expressions to determine the moments in a beam having uniform load distribution and semi-rigid joints at both ends. Annex 11.4 also presents the assesment of stiffness of the hollow section joints.

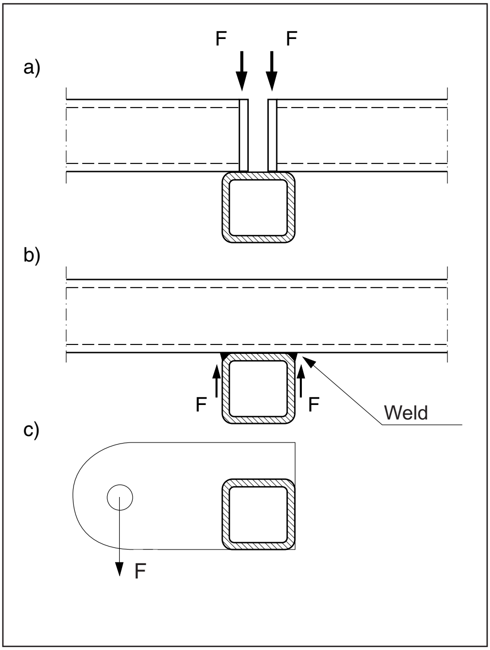


Figure 7.15 Preventing the distortion of the cross-section in a hollow section

7.3.1 Designing the gable beam of the Example building

Design the gable beam of the Example building. The width of the building is 48 m and the column spacing at the end wall is 6 m. The gable beam shall be divided into four parts (48m/ 4 = 12 m) which shall be joined to the end columns on-site using bolted joints. The gable beam is vertically loaded by the support reactions of the purlin trusses. The gable beam transmits the transverse wind load of the hall to the bracing trusses, which means it is subjected also to axial force.

As before, consider the load combination with the snow load governing. The wind direction is assumed to be 'wind parallel to the short side wall', because in this case the long side wall will be subjected to wind pressure as defined for pressure zone D (see Figure 7.9), thereby transmitting compression to the gable beam and being thus the critical case for it. (Due to the compression the beam will be susceptible to flexural buckling, which together with bending will most probably be more critical than the slightly higher tensile loading in case the wind would be parallel to the long side wall, whereby the long side wall would be subjected to suction zone A at a short region and suction zone B at the most of the wall.)

Loads, in case the snow load is governing:

The compressive force is assumed to be constant along the entire beam length. (The compressive load of the beam on the pressure side is calculated taking account the pressure load only, because the simultaneous suction load induced on the suction side does not affect the pressure side since the diagonal wall bracings, located in between, transmit the suction loads to the foundation.) Load factor $K_{FI} = 1,1$:

$$q_{wk} = 0,65 c_{pe.D} = 0,65 \cdot 0,7 = 0,455 \text{ kN/m}^2 \qquad \text{wind (pressure zone D)}$$

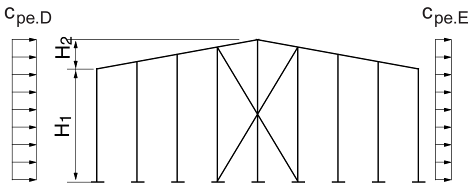
$$q_{wd} = \gamma_{Q.2} \cdot \psi_{0.2} \cdot q_{wk} = (1,5 \cdot 1,1) \cdot 0,6 \cdot 0,455 = 0,450 \text{ kN/m}^2$$

$$N_{Ed} = q_{wd}\left(\frac{3}{8}H_1 + H_2\right)\frac{L}{2} = 0,450 \cdot \left(\frac{3}{8} \cdot 10,5 + 4\right) \cdot \frac{100}{2} = 178,6 \text{ kN}$$

where

L *is the length of the building*

The projection of the roof has been now fully included when calculating the horizontal load (a conservative simplification). A more accurate result could be derived by taking into account the wind load and the pressure coefficients of the roof.



Support reaction of the purlin truss, in case the snow load is governing, will be:

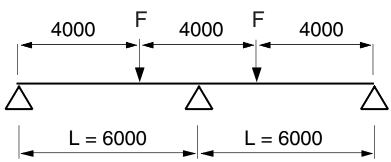
$$F_{y.Ed} = 0,5 \cdot (\gamma_G \cdot g_k + \gamma_{Q.1} \cdot q_{k.1}) \cdot L_p \cdot L_f \qquad \text{self-weight and snow load}$$
$$= 0,5 \cdot [(1,35 \cdot 1,1) \cdot 0,4 + (1,5 \cdot 1,1) \cdot 1,2] \cdot 4 \cdot 10 = 51,5 \text{ kN}$$

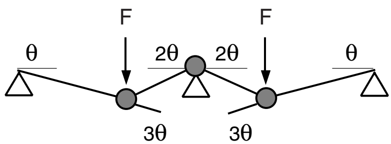
where

L_p *is the purlin spacing*
L_f *is the frame spacing*

Herein the self-weight g_k *can be assumed smaller (0,4 kN/m²), since the weight of the primary truss need not be taken into account.*

Calculate the forces and moments using plastic theory. For vertical loading, the following static model is obtained:





By equalizing the internal and external work, the limit load to generate the plastic mechanism can be determined:

$$M_{pl}(3\theta + 2\theta) = F\left(\frac{2}{3}L \cdot \theta\right) \quad \Rightarrow F = \frac{15M_{pl}}{2L}$$

The shear force acting at the location of the plastic hinge can be derived from a moment summation about the hinge subjected to force F:

$$M_{pl} + V \cdot \frac{L}{3} = -M_{pl} \quad \Rightarrow V = \frac{6M_{pl}}{L} = \frac{12}{15} \cdot F$$

The mechanism (the plastic hinges) is not allowed to form before the limit load $F = F_{y.Ed}$ = 51,5 kN, hence the plastic bending resistance of the beam shall be at least:

$$M_{pl.y.Rd} \geq \frac{2F_{y.Ed}L}{15}$$

Furthermore, in order to prevent the mechanism to form before reaching the limit load $F = F_{y.Ed}$ = 51,5 kN, also the effect of normal force and shear force shall be taken into account, if they reduce the plastic bending resistance. Thereby finally the following design conditions can be derived for the cross-section:

$$M_{N.V.y.Rd} \geq \frac{2F_{y.Ed}L}{15} = \frac{2 \cdot 51,5 \cdot 6}{15} = 41,2\ kNm$$

$$V_{y.Rd} \geq V_{Ed} = \frac{12}{15} \cdot F_{y.Ed} = \frac{12}{15} \cdot 51,5 = 41,2\ kN$$

$$N_{c.Rd} \geq N_{Ed} = 178,6\ kN$$

Since the above presented forces and moments are calculated using a mechanism based on plastic theory, a beam conforming to cross-section Class 1 shall be chosen.

Resistances at ultimate limit state:

Cross-section resistance:

Try a hollow section with dimensions 150×100×8. The steel grade is SSAB Domex Tube Double Grade, which fulfills the EN 10219 requirements for both steel grades S420MH and S355J2H. Thereby the design calculations may be performed at designer's own choice either according to grade S420 or grade S355. Grade S420 is chosen in this Example as design basis.

The cross-section is classified into Class 1 when subjected to compression only (Annex 11.1). Thus, considering the (M+N) interaction, it is not necessary to carry out a more detailed assesment for the cross-section classification (cf. clause 2.9.1.5.1).

Cross-section compression resistance (Annex 11.1):

$$N_{c.Rd} = N_{pl.Rd} = 1480\ kN \geq N_{Ed} = 178,6\ kN \quad OK$$

Shear resistance:

$$\frac{h}{t} = \frac{150}{8} = 18,8 \le \frac{72\varepsilon}{\eta} + 3 = \frac{72 \cdot \sqrt{235/420}}{1,0} + 3 = 56,9 \quad \text{(clause 2.7.1)}$$

$\Rightarrow$ *shear buckling does not reduce the shear resistance, which thereby shall be determined according to the plastic shear resistance:*

$V_{pl.Rd} = 512,8\ kN$ *(Annex 11.1)*

$V_{Ed} = 41,2\ kN \le V_{pl.Rd} = 512,8\ kN$ OK

$V_{Ed} = 41,2\ kN \le 0,5V_{pl.Rd} = 0,5 \cdot 512,8 = 256,4\ kN \Rightarrow$ *shear force does not reduce the bending resistance*

Bending resistance (Annex 11.1):

$M_{pl.y.Rd} = 71,1\ kNm \ge 41,2\ kNm$ OK

Since shear force does not reduce the bending resistance (clause 2.9.1.6.1), the resistance of the cross-section to the combined effect of the forces can be verified according to clause 2.9.1.5 for the combined effect of (M+N) only:

$$N_{Ed} = 178,6\ kN \le \frac{0,5(A-2bt)f_y}{\gamma_{M0}} = \frac{0,5 \cdot (3524 - 2 \cdot 100 \cdot 8) \cdot 420}{1,0} = 404,0\ kN$$

and:

$N_{Ed} = 178,6\ kN \le 0,25N_{pl.Rd} = 0,25 \cdot 1480 = 370,0\ kN$

$\Rightarrow$ *since the both above presented conditions are satisfied, consequently:*

$M_{N.y.Rd} = M_{pl.y.Rd} = 71,1\ kNm \ge 41,2\ kNm$ OK

Thus, the resistance of the cross-section is sufficient.

<u>Member resistance</u>:

Now, check that flexural buckling does not take place in the beam before the mechanism is created. The gable beam is laterally restrained by the profiled sheeting in the roof. Consequently, flexural buckling in lateral direction need not be considered. For flexural buckling in vertical direction, the buckling length is $L_{cr} = L = 6$ m (a conservative simplification). Thereby the buckling resistance will be (Annex 11.2):

$N_{b.y.Rd} = 422,1\ kN \ge N_{Ed} = 178,6\ kN$ OK

The impact of the bending moment to flexural buckling can be omitted in this Example, because flexural buckling in vertical direction complies with the already assumed failure mechanism.

Stresses and deflection at serviceability limit state:

Since plastic theory has now been applied for assessments at ultimate limit state, it is necessary to check whether the yield strength of the material will be exceeded due to the serv-

iceability limit state loads (the potential exceeding of yield strength would have an impact on calculating the deflections). The partial safety factors for loads are according to expression (7.10) (load factor K_{FI} is not applied at serviceability limit state assessments):

$$F_{Ed} = 0,5(g_k + q_{k.1}) \cdot L_p \cdot L_f = 0,5 \cdot (0,4 + 1,2) \cdot 4 \cdot 10 = 32\ kN \quad \textit{self-weight and snow load}$$

$$M_{el.y.Ed} = \frac{5}{27} F_{Ed} L = \frac{5}{27} \cdot 32 \cdot 6 = 35,6\ kNm = M_{max} \quad \textit{moment at intermediate support}$$

$$N_{Ed} = \psi_{0.2} \cdot q_{wk} \cdot \left(\frac{3}{8} H_1 + H_2\right) \frac{L}{2} = 0,6 \cdot 0,455 \cdot \left(\frac{3}{8} \cdot 10,5 + 4\right) \cdot \frac{100}{2} = 108,3\ kN \quad \textit{wind load}$$

$$\sigma_{max} = \frac{M_{el.y.Ed}}{W_{el.y}} + \frac{N_{Ed}}{A} = \frac{35,6 \cdot 10^6}{134,4 \cdot 10^3} + \frac{108,3 \cdot 10^3}{3524} = 295,6\ N/mm^2 \le f_y = 420\ N/mm^2$$

Since the stresses at serviceablity limit state remain at the elastic region, the deflection of the beam can be calculated using elastic theory:

$$\delta_{max} = \frac{F_{Ed} \cdot a(L-a)^2}{6EI} \sqrt{\frac{a}{2L+a}} = \frac{32 \cdot 10^3 \cdot 4(6-4)^2 \cdot 10^9}{6 \cdot 2,1 \cdot 10^5 \cdot 1008 \cdot 10^4} \sqrt{\frac{4}{2 \cdot 6 + 4}} = 20,2\ mm \approx \frac{L}{300}$$

$$\delta_{max} \le \frac{L}{300} \quad OK \quad (\text{Table 7.5})$$

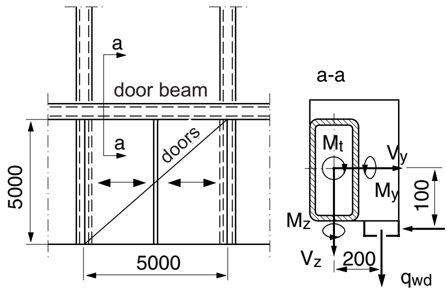
Thus, the resistance and stiffness of hollow section 150×100×8 are sufficient when the snow load is governing.

Comparison S420 vs S355:
In case the design calculations would be performed according to grade S355, the calculations would follow basically the same principles. In this Example, increase of the material strength S355 → S420 improves the buckling resistance by ca. 4 % and for the other resistances by the full increase of the yield strength (= + 18 %). When it comes to the deflection, the change of material strength has no impact when the stresses at serviceability limit state will still remain at the elastic region.

Resistances at ultimate limit state, when the wind load is governing:
The corresponding assessments must be carried out for this load combination respectively.

7.3.2 Designing the door beam of the Example building

Design the door beam placed on the long side wall. The door beam is joined to the wind column and to the column of the primary frame. The height of the door is 5 m and its self-weight is 0,75 kN/m^2. It is a sliding door, and the assembly rail is placed at a distance of 200 mm from the beam's z-axis and 100 mm from its y-axis. The door beam is subjected to the self-weight of the door and to the wind load. It is assumed that the lower edge of the door is supported by the floor, consequently only half of the wind load is transferred to the door beam.



Loads:
The beam is subjected to biaxial bending and to torsional moment. The internal forces and moments are determined using elastic theory.

Try a hollow section with dimensions 180×100×6. The steel grade is SSAB Domex Tube Double Grade, which fulfills the EN 10219 requirements for both steel grades S420MH and S355J2H. Thereby the design calculations may be performed at designer's own choice either according to grade S420 or grade S355. Grade S420 is chosen in this Example as design basis.

Next, calculate the torsional load caused by the self-weight of the door and by the wind load (pressure zone D will be governing, because then the wind and the self-weight twist the beam to the same direction). Load factor $K_{FI} = 1,1$:

$$q_{wk} = 0,65 c_{pe.D} = 0,65 \cdot 0,7 = 0,455 \ kN/m^2 \qquad \text{wind (pressure zone D)}$$

$$q_{xd} = \gamma_G \cdot g_k \cdot e_1 + 0,5 \cdot \gamma_{Q.1} \cdot q_{wk} \cdot H \cdot e_2 \qquad \text{torsion about the beam's longitudinal axis}$$

$$= (1,35 \cdot 1,1) \cdot 0,75 \cdot 5 \cdot 0,2 + 0,5 \cdot (1,5 \cdot 1,1) \cdot 0,455 \cdot 5 \cdot 0,1 = 1,30 \ kNm/m$$

where

- e_1 *is the eccentricity of the vertical load*
- e_2 *is the eccentricity of the wind load*
- H *is the height of the door*

$M_{x.Ed}$	$= 3{,}25\ kNm$	*torsional moment at the support*
g_d	$= \gamma_G \cdot g_k \cdot H_d$	*vertical load causing bending due to the self-weight*
	$= (1{,}35 \cdot 1{,}1) \cdot 0{,}75 \cdot 5$	*of the door*
	$= 5{,}57\ kN/m$	
$M_{y.Ed}$	$= 17{,}4\ kNm$	*vertical bending moment at the centre of the span*
$V_{z.Ed}$	$= 13{,}9\ kN$	*vertical shear force at the support*
q_{wd}	$= 0{,}5 \cdot \gamma_{Q.1} \cdot q_{wk} \cdot H$	*horizontal load causing bending due to the wind*
	$= 0{,}5 \cdot (1{,}5 \cdot 1{,}1) \cdot 0{,}455 \cdot 5$	
	$= 1{,}88\ kN/m$	
$M_{z.Ed}$	$= 5{,}88\ kNm$	*horizontal bending moment at the centre of the span*
$V_{y.Ed}$	$= 4{,}70\ kN$	*horizontal shear force at the support*

Resistances at ultimate limit state:
The cross-section is classified into Class 1 in bending about y-axis and Class 2 about z-axis, hence the bending resistances can be determined according to plastic theory. The effects of torsion and shear shall be checked later, since they do not have their maximum values in the same location as the bending moment.

Bending resistances (Annex 11.1):

$M_{pl.y.Rd} = 76{,}0\ kNm \geq M_{y.Ed}$ OK
$M_{pl.z.Rd} = 50{,}5\ kNm \geq M_{z.Ed}$ OK

Combined effects in biaxial bending (clause 2.6.4.1):

$$\left[\frac{M_{y.Ed}}{M_{pl.y.Rd}}\right]^{1,66} + \left[\frac{M_{z.Ed}}{M_{pl.z.Rd}}\right]^{1,66} = \left[\frac{17,4}{76,0}\right]^{1,66} + \left[\frac{5,88}{50,5}\right]^{1,66} = 0,1147 \leq 1,0 \quad OK$$

In regard to shear and torsion, plastic resistances can be applied (Annex 11.1) because local buckling will not be critical now (clause 2.7.1 and clause 2.8.1).

$M_{pl.x.Rd} = 43{,}4\ kNm \geq M_{x.Ed}$ OK
$V_{pl.z.Rd} = 486{,}9\ kN \geq V_{z.Ed}$ OK
$V_{pl.y.Rd} = 270{,}5\ kN \geq V_{y.Ed}$ OK

Shear and torsion have their maximum values at the both ends of the beam. Shear stress at beam ends due to torsion is:

$$\tau_{t.Ed} = \frac{M_{t.Ed}}{W_t} = \frac{M_{x.Ed}}{W_t} = \frac{3,25 \cdot 10^6}{178,9 \cdot 10^3} = 18,2\ N/mm^2 \quad (expression\ (2.99))$$

Check the combined effect of torsion and shear (clause 2.9.1.2.1):

$V_{pl.T.z.Rd} = 468{,}3\ kN \geq V_{z.Ed}$ *OK (z-direction, shear resistance, reduced due to torsion)*
$V_{pl.T.y.Rd} = 260{,}2\ kN \geq V_{y.Ed}$ *OK (y-direction, shear resistance, reduced due to torsion)*

Stresses and deflection at serviceability limit state:
Check whether the yield strength of the material will be exceeded due to the serviceability limit state loads (the potential exceeding of yield strength would have an impact on calculating the deflections). The partial safety factors for loads are according to expression (7.10) (load factor K_{FI} is not applied at serviceability limit state assessments):

$g_k = 3{,}75\ kN/m$ *self-weight*
$M_{el.y.Ed} = 11{,}7\ kNm$ *vertical bending moment at the centre of the span*
$q_{wk} = 1{,}14\ kN/m$ *wind*
$M_{el.z.Ed} = 3{,}56\ kNm$ *horizontal bending moment at the centre of the span*

$$\sigma_{max} = \frac{M_{el.y.Ed}}{W_{el.y}} + \frac{M_{el.z.Ed}}{W_{el.z}} = \frac{11,7 \cdot 10^6}{145,5 \cdot 10^3} + \frac{3,56 \cdot 10^6}{104,8 \cdot 10^3} = 114,4\ N/mm^2 \leq f_y = 420\ N/mm^2$$

Since the stresses at serviceablity limit state remain at the elastic region, the deflection of the beam can be calculated using elastic theory:

$$\delta_{z.max} = \frac{5}{384}\frac{g_k L^4}{EI_y} = \frac{5}{384} \cdot \frac{3,75 \cdot 5000^4}{2,1 \cdot 10^5 \cdot 1310 \cdot 10^4} = 11,1\ mm = \frac{L}{450} \quad \textit{vertically}$$

$$\delta_{z.max} \leq \frac{L}{400} \quad \textit{OK} \quad \textit{(Table 7.5)}$$

$$\delta_{y.max} = \frac{5}{384}\frac{q_{wk} L^4}{EI_z} = \frac{5}{384} \cdot \frac{1,14 \cdot 5000^4}{2,1 \cdot 10^5 \cdot 523,8 \cdot 10^4} = 8,4\ mm = \frac{L}{595} \quad \textit{horizontally}$$

$$\delta_{y.max} \leq \frac{L}{400} \quad \textit{OK} \quad \textit{(Table 7.5)}$$

Thus, the resistance and stiffness of hollow section $180 \times 100 \times 6$ are sufficient to act as a door beam. In order to ensure the proper functioning of the door, it is often necessary to limit the deflection of a door beam to be quite small.

Comparison S420 vs S355:
In case the design calculations would be performed according to grade S355, the cross-section classification and all the calculations would follow basically the same principles. In this Example, increase of the material strength S355 → S420 improves all resistances by the full increase of the yield strength (= + 18 %). When it comes to the deflection, the change of material strength has no impact when the stresses at serviceability limit state will still remain at the elastic region.

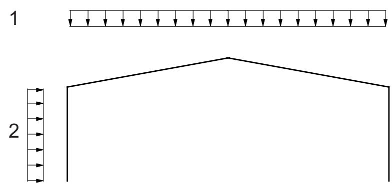
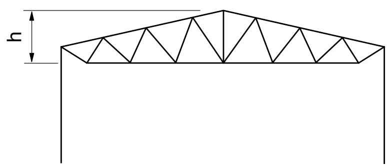
7.4 Truss design

When designing trusses, the joints between brace members and chords are usually assumed pinned, so the brace members are subjected to normal force only. Bending moments need not be taken into account in the design of joints if the validity conditions presented in the joint resistance tables in Annex 11.3 are fulfilled. However, the chords as continuous members are subjected also to bending stresses (see Table 3.4). The bending moment in the chord becomes smaller, if the loading (for example the loads from the purlins) can be directed to the locations of the joints in the truss. Hollow sections function efficiently as compression and tension members, which makes the truss a light-weight structure in relation to its load-bearing capacity.

In lattice design, it is recommended that design software package are used which include resistance data for structural hollow sections (for example WinRami, Annex 11.6).

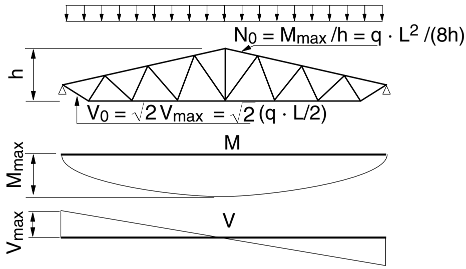
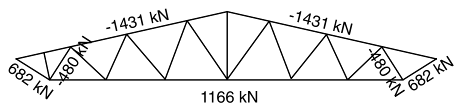
The cost of a lattice does not only consist of weight of the steel, but also of workshop fabrication and erection on site. A lattice with gap joints and few members may thus be less costly than a lighter weight lattice with lots of members and overlap joints. The most profitable type of truss and geometry of joints must be decided on a case by case basis.

Table 7.8 Steps in truss design

Task	
1. Determine the loads in the structure. Determine the most severe load combination. The direction of the load is important to identify, because tension and buckling resistances of a structural hollow section differ from each other.	
2. Determine the height of the lattice. According to [27] the optimum height is h = L / 9 ... L / 12. According to [28] the optimum height is slightly lower, h = L / 10 ... L / 16. The heavier the loading, the greater height is chosen. The height is the most decisive factor regarding the steel consumption in the truss. The roof slope, on the other hand, has not any big influence on the steel consumption (optimum slope regarding the truss alone is about 1:8...1:10). Often the height of the truss is determined by the space requirement and the functional requirements of the building as well as requirements concerning transportation and erection. Select the type of truss and the purlin spacing. Based on above data it is possible to make the static structural model of the truss.	

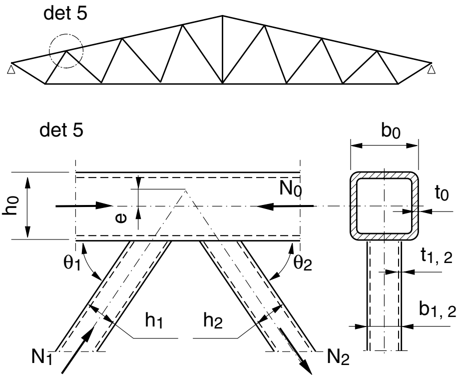
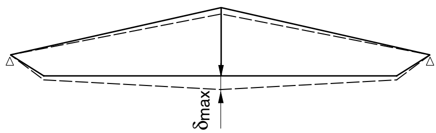
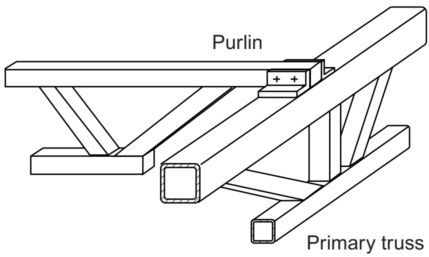
(continues)

Table 7.8 Steps in truss design
(continued)

3. The members are chosen preliminary by calculating the moment of the truss as if it was a beam. Calculate the initial value for the chord forces by dividing the maximum value of the moment by the lattice height ($N_0 \approx M_{max}/h$). Determine the initial value for the brace member forces using the shear force value of the beam ($V_0 \approx V_{max}$). Check the resistance of the joints subjected to the greatest forces (usually the nearest to the support joints of the brace members to the top and bottom chord). Only members of cross-section Classes 1 - 2 are used (more detailed requirements are presented for each joint type in the tables of Annex 11.3). Choose the dimensions of the brace members and the chords so that the ratio of the brace member width to the chord width is approximately 0,7 - 0,8. The whole weight of the truss is composed roughly as follows: - chord in compression about 50 % - chord in tension about 30 % - brace members about 20 % In regard to the resistance of the joints, it is good to choose relatively thick-walled chords. It is recommended to limit the number of different sizes of brace members to 2 - 4 (oversizing of single brace members has very marginal impact on the steel consumption of the whole truss).	
4. Calculate the actual member forces by using an appropriate design software. Determine the member forces for all load combinations. In the calculation model, assume the chord to be continuous and the brace members to have pinned connections. Check the resistance of the selected members to actual member forces. The resistance of the brace members is calculated as either tension or buckling resistance. The resistance of the tension chord is calculated as tension resistance (see Table 3.4). The resistance of the compression chord is determined by the combined effect of moment and normal force. If the member sizes need to be adjusted, recalculate the member forces using the new dimensions. Check that the dimensions of the members and the eccentricities of the joints fulfill the validity conditions presented in the joint resistance tables in Annex 11.3 for the relevant joint type.	

(continues)

Table 7.8 Steps in truss design
(continued)

5. Calculate the local resistances of the joints and design the welds. Decide whether to reinforce the joints subjected to the greatest forces or to select stronger members. The reinforcement of joints is profitable if the fabrication costs due to reinforcement are smaller than the increased material costs of the stronger members.	
6. Calculate the deflection and compare it with the permitted value.	
7. Design the lateral support of the truss and the purlin-to-truss joints. Determine the location of assembly joints of the lattice, taking transport into account.	

7.4.1 Selection of truss type

The most commonly used lattice types are K, KT and N trusses.

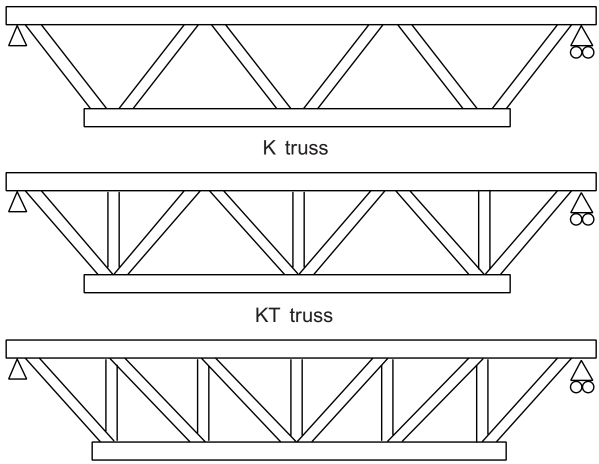


Figure 7.16 Various truss types

K truss is suitable for long-span structures where loads can be transferred directly to lattice joint locations. In K type trusses, the number of members is small and joints are simple. Wide spacing between members also leaves room for tube lead-throughs, like for HVAC-installations. However, the buckling length of the top chord is great, which may result in a heavier chord than in the other lattice types. K truss, however, is simple and affordable in terms of fabrication costs.

In a **KT truss**, the spacing of the top chord's lateral supports is more dense, so the buckling resistance of the chord is better than in K truss. The joints in a KT truss are, however, more complex to prepare. The joints in the bottom chord must often be constructed as overlap joints, which increases the fabrication costs.

In an **N truss** the number of members is greater than in K truss. In deep and short lattices the brace member forces are great compared with the chord forces. In such a case an N truss is efficient, since the compressed brace members are shorter than in KT trusses. The joints must usually be constructed overlapped to avoid great eccentricities.

In long-span lattices there are big differences in brace member forces. Close to the support, the brace member forces are greater than in the mid-region of the lattice. To reduce the weight, the brace members subjected to smaller forces can be made of lighter-weight hollow sections. However, to make the workshop fabrication easier, it is not advisable to use more than 2 - 4 different brace member sizes. With hollow sections having the same external dimensions, only one wall thickness should be used in one lattice to avoid confusing them during workshop fabrication. For simplicity, the chord is usually made of one hollow section size, although the normal force varies along the length of the chord. The chord can be constructed of hollow sections of different sizes if the truss is divided into assembly blocks. In such a case, the size of the chord changes at the assembly joint.

On a continuous lattice girder, the support force and bending moment reach their maximum values at the intermediate support. Through placing a vertical brace member at the support, the buckling length of the compressed bottom chord can be reduced. The support force is beneficial to carry by using a vertical brace member, since the longer diagonal braces are subjected to tension. In the subsequent diagonal compression braces, the normal force then becomes smaller from the value at the support (Figure 7.17).

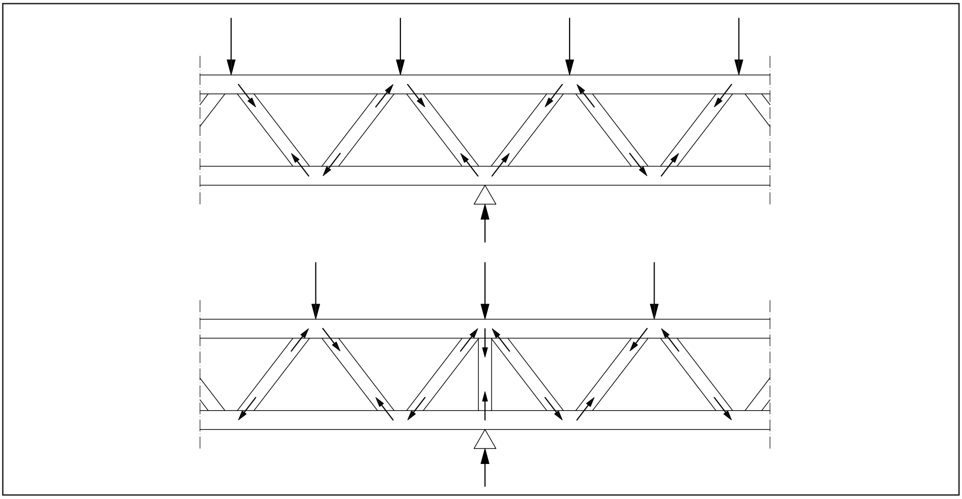


Figure 7.17 Intermediate support of a continuous lattice girder

Due to lead-throughs for various pipings, openings must sometimes be constructed in the truss. To ensure sufficient shear resistance, the lattice must be reinforced at the openings. If the height of the opening is smaller than that of the lattice, a lower truss can be constructed between the top chord and the opening (Figure 7.18a). If the opening and the lattice are of equal height, the opening must be reinforced with a frame (Figure 7.18b). The bending moment due to the shear force must be taken into account in the design of the frame and the chords. Generally it is advisable to place the openings in regions with the least possible shear force.

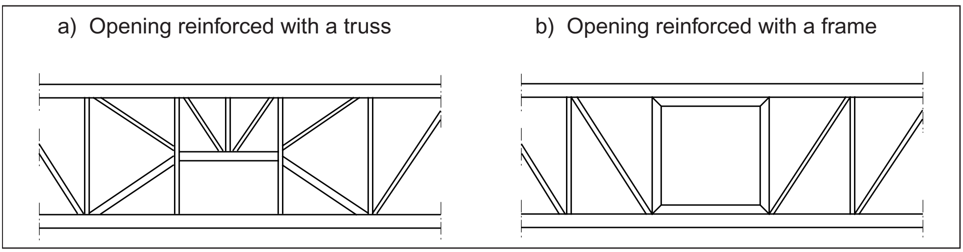
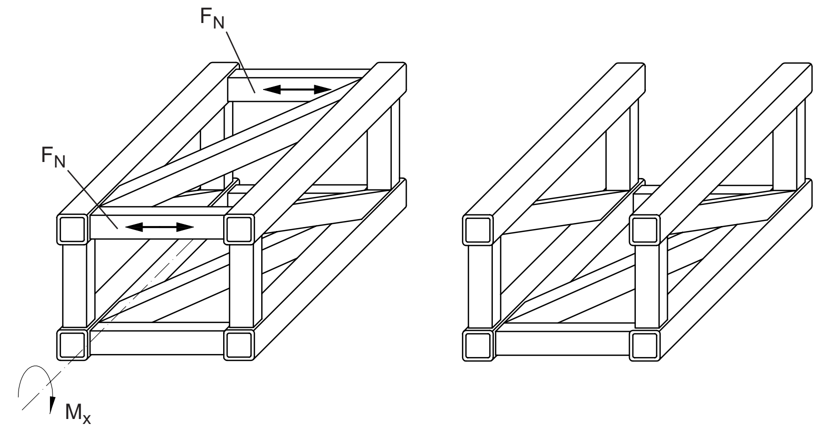


Figure 7.18 Reinforcing openings in trusses

Laterally joined trusses

Girders for pipelines, conveyors and working platforms are generally constructed as a bridge by joining the primary trusses with horizontal wind trusses (Figure 7.19). The lattices can also be joined with plates whose joints are designed by the horizontal loads. The structure composed of two trusses is advisable to be stiffened using diagonal braces perpendicular to the plane of the trusses according to Figure 7.19a if the trusses are subjected also to torsional load. In case of Figure 7.19a, the transverse force generated by the compressed top chord shall be taken into account in design of the horizontal truss (or the lateral supports of the top chord and their fixings). As a preliminary approximation, the transverse force F_N can be assumed to be 2,5 % of the normal force N_{Ed} acting in the chord to be supported. A more accurate calculation of the transverse force is presented in Part EN 1993-1-1 of Eurocode and in [22,23]. In both cases of Figure 7.19, the buckling length of the compressed top chord can be determined according to clause 7.4.2, if a more detailed analysis is not made.



a) Bottom and top chords are joined laterally (top chords are in compression)

b) Only bottom chords are joined laterally

Figure 7.19 Laterally joined trusses (F_N is the transverse force preventing buckling)

7.4.2 Selection of the chord member

A decisive factor in the selection of chord member are the buckling lengths about different axes. When the buckling lengths are close to equal in both directions, square hollow sections are the most profitable. The use of rectangular hollow sections is efficient, if the buckling lengths differ considerably from each other. However, a broad and low chord is not a good solution in regard to the local strength of the chord face and the shear resistance of the chord. If the chord has significant bending loads between the lattice nodal points, it is advisable to select a deep chord.

On a hollow section truss, the buckling length of the chord for in-plane buckling as well as for out-of-plane buckling can be taken as L_{cr} = 0,9L. For the in-plane buckling, L is the distance between the nodal points of the brace members (system length). For the out-of-plane buckling, L is the distance between the lateral supports of the chord, unless a smaller value is justified by analysis [14,15,16]. Lateral bracing elements are designed for lateral loads and for the lateral forces caused by the compressed chords. As a preliminary approximation, the transverse force can be assumed to be 2,5 % of the normal force acting in the chord to be supported. A more accurate calculation of the transverse force is presented in Part EN 1993-1-1 of Eurocode and in [22,23].

The bending moments due to the eccentricities of the joints shall be taken into account in the design of the compressed chord. Thus the chord shall be designed as a member subjected to compression and bending according to clauses 2.9.1.5 and 2.10. In the design of the tension chord, the bending moments due to eccentricity need not be taken into account, if the conditions presented in Table 3.4 are fulfilled.

The joints of square and rectangular hollow sections are simpler than the joints of circular hollow sections. Exceptional cases in which the use of circular hollow sections is advisable, are triangular trusses and space frame trusses.

A thick chord wall is efficient in terms of joint resistance, but in terms of buckling resistance the situation is quite the opposite (the material is more effectively utilised when the external dimensions of the cross-section are increased instead of increasing the wall thickness). A feasible compromise must be reached in design, or the chord face must be reinforced using additional plates.

Heavily loaded trusses having a large spacing between the lateral restraints of the compressed chord, can be constructed using a double chord. In a double chord truss, the chords are connected to each other directly (Figure 7.20b) or through the brace members (Figures 7.20a and c). The horizontal inertia of the chord increases significantly if the chords are connected through the brace members. Regarding the resistance of the joint, chord face yielding is not possible, since the brace member forces are transferred directly to chord webs. In the design of a joint shown in Figure 7.20c, the forces due to three-dimensionality must be taken into account. When designing the joint shown in Figure 7.20b, the same formulae as with an I-section chord can be used, if the interspace between the corner roundings is welded at joint locations as shown in the figure. The web thickness of the I-section t_w is substituted by the combined thickness of both chord webs $2t_0$, and the internal radius of the chord corner is applied as the rounding radius. The dimensions of a joint shown in Figure 7.20a should be chosen so that the brace member welds are accessible. In addition, all brace members must have equal width, but the wall thickness and the depth of the cross-sections may be varied. Regarding the shear resistance of the chords in Figure 7.20a, the following shear areas shall be used [29]:

$$A_V = 2,6 h_0 t_0 \qquad \text{for } h_0/b_0 \geq 1 \qquad (7.17a)$$

$$A_V = 2,0 h_0 t_0 \qquad \text{for } h_0/b_0 < 1 \qquad (7.17b)$$

where h_0 is the depth of the chord
b_0 is the width of the chord

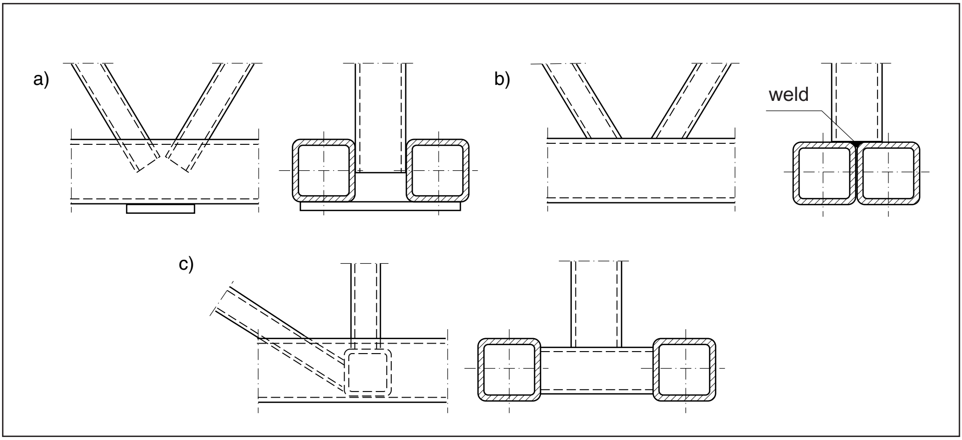


Figure 7.20 Double chord

7.4.3 Selection of brace members

The selection of brace members is less complex, since a brace member with thin walls and great external dimensions is better in terms of both joint strength and member resistance. A thin-walled brace member is advisable also in respect to the throat thickness of the joint weld. However, the slenderness of the cross-section of the brace member must be kept within the limits specified in the joint resistance tables in Annex 11.3. As a thumb rule it can be said that the members of a truss shall belong into Class 1 - 2. Special attention must be paid to the brace-to-chord weld when the chord and the brace member are of nearly equal width (see Chapter 8, Figures 8.15 - 8.18). Usually it is advisable to choose the dimensions of the brace member such that the ratio of its width to the chord width is 0,7 - 0,8.

It is always conservative to take the system length of a brace member as its buckling length. However, the joints welded on all sides have rigidity, thus according to Part EN 1993-1-1 of Eurocode the buckling length can be taken as L_{cr} = 0,75L, where L is the system length (distance between the nodal points) [14,15,16]. The National Annex may give additional information to determine the buckling length of the brace members.

Finnish National Annex to standard EN 1993-1-1 [17]:

The instructions given in Eurocode is used. Smaller buckling lengths may be used based on testing or calculations.

Thus, on the basis of the Finnish National Annex, the buckling length of the brace members may be calculated also using the formulae in [31], which are valid when the ratio b_1 / b_0 is less than 0,6. However, in case of fully overlapped joints (λ_{ov} = 100 %) or flattened brace members, the buckling length shall always be the system length of the member.

With great values of the joint angle θ_i it is advisable to use rectangular brace members in order to keep the joint eccentricities small (Figure 7.21a). With small joint angles or near the supports, it is possible to use square brace members in single span trusses (Figure 7.21b). Near to the support, the normal forces in the chord are small, so it is easier for the chord to resist the bending moments due to joint eccentricities.

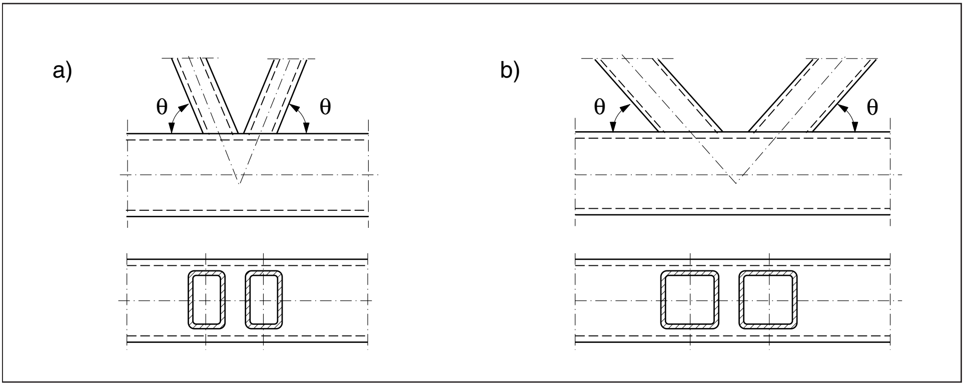


Figure 7.21 The effect of brace member shape

7.4.4 Design of truss joints

Truss joints can be divided into two main groups; gap joints and overlap joints. Gap joints are easier to manufacture since the brace members can be cut to the required angle in one go. In the assembly of the truss, there is also somewhat better fitting allowance available. However, in a gap joint made of square brace members, the eccentricity becomes usually great. Consequently, the eccentricity increases the bending load of the chord. Furthermore, the shear resistance of the joint may become governing in a gap joint. An overlap joint is more complex to manufacture, because the overlapping member must be cut to two different angles. The fitting tolerances of the parts need to be more restricted than with the gap joints. On the other hand, the resistance of the joint is better, and eccentricity may disappear completely with an appropriate overlap.

The smallest angle permitted for the brace members in the joint resistance tables is 30°. In practice, it is advisable to avoid small joint angles, as they make the welding of the heel-side quite difficult. With small angles, even minor deviations in cutting the hollow section can result in great root gaps in the joints. If the joint angle θ_i is smaller than 60°, the ends of brace members must be chamfered (Figure 7.22, also Chapter 8 Figures 8.15 - 8.18).

In gap joints, the gap between brace members shall be at least $g \geq (t_1 + t_2)$, in order to have sufficient space for executing the welds and to ensure sufficient plastic deformation capacity for the chord face (Figure 7.22 and Table 3.5). In some joint types, even greater gap than this is required. More detailed requirements by each joint type are presented in the resistance tables in Annex 11.3.

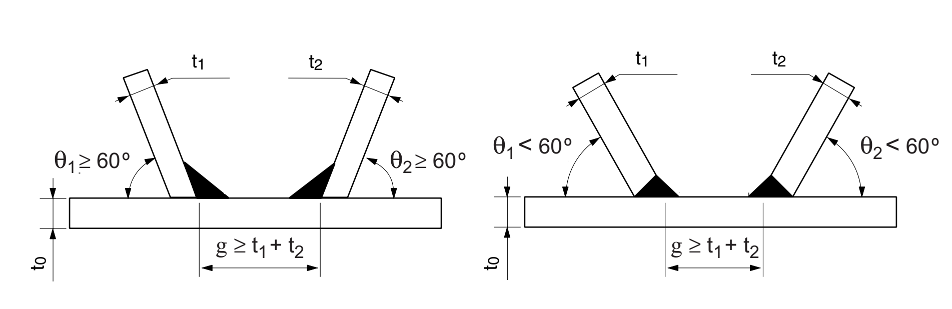


Figure 7.22 Minimum value of the gap

In the joints of hollow section lattices, local stress concentrations are generated in the region of the joint. The stress peaks are, however, somewhat evened out by the yielding of the brace members and chords. Nevertheless, due to the stress peaks the welds of the joints are usually designed to have equal strength with the members. **A fillet weld made around the perimeter of the brace member has equal strength with the brace member, when its throat thickness is at least equal to that presented in Table 3.9.** A butt weld of the brace member can be assumed to have equal strength when it is fully penetrated (see more details in clause 3.3.4).

Reinforced truss joints

Reinforcing a joint is profitable if the number of joints to be reinforced is small compared to the total number of members in the lattice. On one hand, reinforcement of joints increases fabrication costs; but on the other hand, it reduces the weight of the structure and removes the need to use too many hollow section sizes. The design rules for reinforced joints are presented in Part EN 1993-1-8 of Eurocode and in Tables 11.3.17 - 11.3.21 of this handbook.

The chord face can be reinforced by using additional plates (Figure 7.23a). This is an effective method if the brace members are notably narrower than the chord. The thickness of the reinforcing plate shall be at least twice the wall thickness of the brace member ($t_p \geq 2t_i$). When calculating the resistance of the joint, the thickness of the chord face t_0 is replaced by the thickness of the reinforcement plate t_p, and the width of the chord is replaced by the width of the reinforcement plate b_p. The resistance of joints reinforced on the chord face is given in Tables 11.3.17 and 11.3.19. The reinforcement plate is prone to lamellar tearing, which must be taken into account when selecting the plate material (provisions regarding lamellar tearing, see Chapter 5).

The shear resistance of the chord can be improved with plates welded to the sides of the chord (Figure 7.23b). The height of the plates is equal to that of the chord. When calculating the shear area, the thickness of the chord web t_0 is replaced with the sum $t_0 + t_p$. The resistance of joints reinforced on the chord webs is given in Tables 11.3.18 and 11.3.20. The shear resistance of the joint often governs, when the brace member and the chord are of equal width.

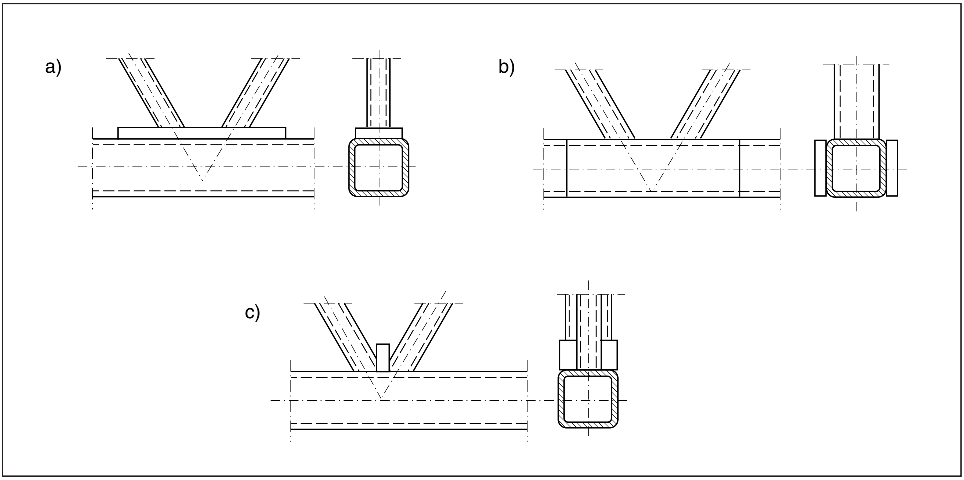


Figure 7.23 Reinforcement of truss joints

In overlap joints, an intermediate plate can be used between the brace members, if as a normal overlap joint the amount of overlapping would be smaller than the requirement given in the resistance tables, or if the width difference between the brace members to be overlapped is so great that the requirements given for it in the resistance tables would not be fulfilled (Figure 7.23c). The thickness of the intermediate plate shall be at least twice the wall thickness of the brace member ($t_p \geq 2\,t_i$). The resistance of overlap joints reinforced using an intermediate plate is given in Table 11.3.21.

7.4.5 Truss joint at the support

The transmission of the truss shear force to the column should be examined carefully. If the centroidal axes of the diagonal member, the chord and the column do not intersect at the same point, the truss shear force is transferred to become a bending moment to the column. In practice, it is often favourable to allow a small eccentricity if the joint becomes thereby easier to manufacture. The end of the chord is usually closed (stiffened) by using a plate in order to achieve sufficient resistance to concentrated forces. At the joint location, the chord shall be designed for the combined effect of normal force, shear force and bending moment. Especially at intermediate support of a continuous lattice girder, the effect of the normal force is significant.

Various methods to construct a joint have been presented in Figure 7.24. In case of Figure 7.24a, the joint between the brace member and chord is treated as a K / N joint if the gap g fulfills the requirements given in the resistance tables for K / N joints. On greater values of gap, the joint is treated as two separate Y joints.

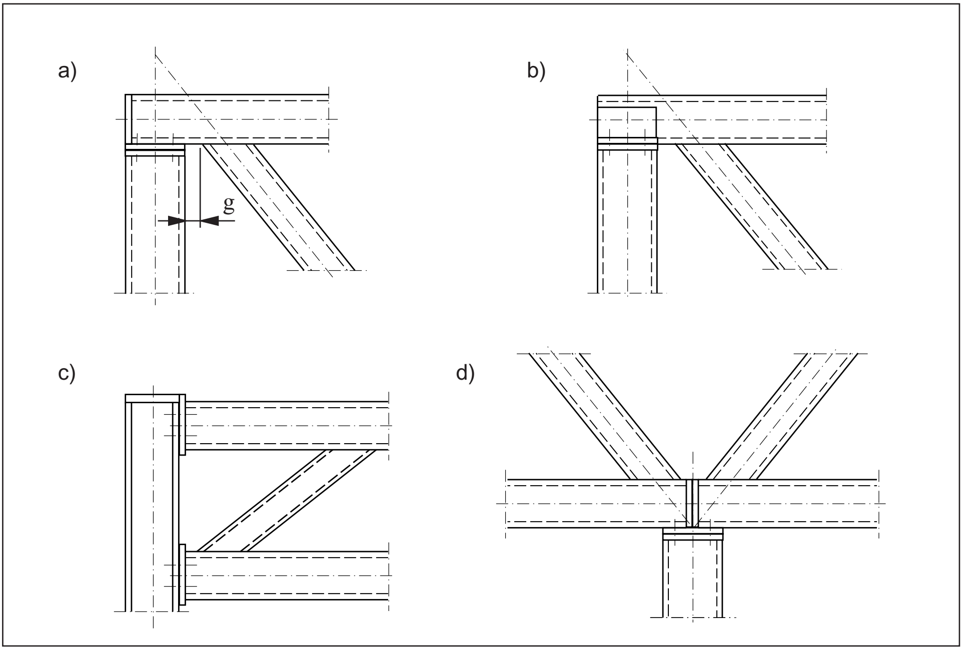


Figure 7.24 Truss joints at the support

7.4.6 Truss joint at the ridge

At the ridge of the truss, the normal forces in brace members act usually in the same direction as shown in Figure 7.25 (i.e. tension in both brace members or compression in both brace members). In case of Figures 7.25a and b, the joint is then designed applying the rules of an X joint. First, the joint resistance of brace members 1 and 2 is checked in the normal way for each member. Then additionally, the resistance of the joint is checked for the sum of the vertical component of both brace members as follows [18,19,20]:

$$N_{1.Ed} \sin\theta_1 + N_{2.Ed} \sin\theta_2 \leq N_{x.Rd} \sin\theta_x \tag{7.18}$$

where $N_{x.Rd} sin\theta_x$ is the greater value of $|N_{1.Rd} sin\theta_1|$ or $|N_{2.Rd} sin\theta_2|$ calculated according to the X joint resistance table.

If an intermediate plate is used in the joint according to Figure 7.25c, either side of the joint is designed with the same principles as at support (see clause 7.4.5): formulae for K / N joint are used when the gap g fulfills the requirements given in the resistance tables for K / N joints, but on greater values of the gap the joint is treated as two separate Y joints.

In addition to that presented above, if a hip is constructed in the chord member according to Figure 7.25b or c, the resistance of the joint must be checked also as a Knee joint. The resistance of a Knee joints is given in Table 11.3.14. Using a reinforcing plate improves the resistance of a Knee joint considerably. The thickness of the reinforcing plate shall be $t_p \geq 1{,}5\, t_0$ (where t_0 is the wall thickness of the chord), however at least $t_p \geq 10$ mm.

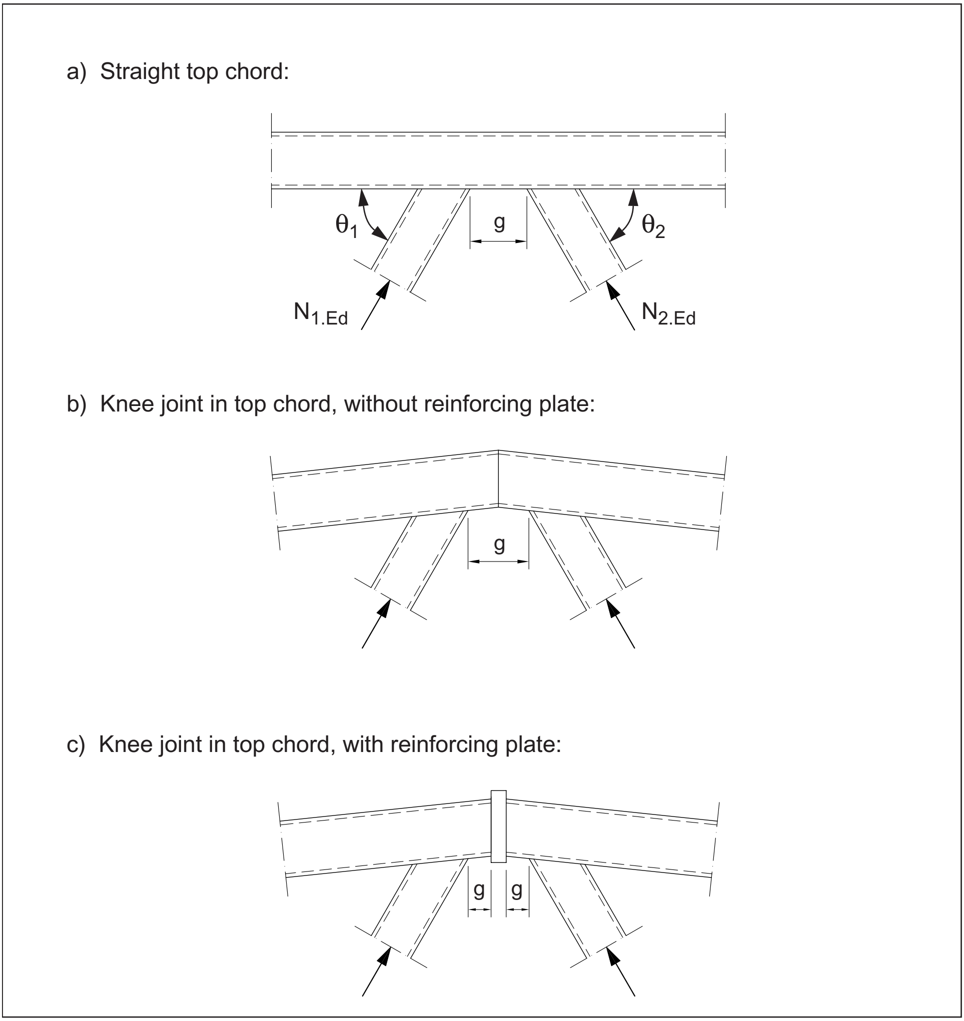


Figure 7.25 Truss joints at the ridge

7.4.7 Overlap joints in the truss

In overlap joints, the amount of overlap shall be at least $\lambda_{ov} \geq 25$ % in order to ensure that the shear force is transferred from one brace member to another. General guidelines regarding overlap joints are presented in Table 3.5 of Chapter 3. More detailed requirements by each joint type are presented in the resistance tables in Annex 11.3.

The basic principle of the instructions presented in Table 3.5 for overlapped brace members is, that first the ‘stronger’ brace member is welded to the chord, and then the ’weaker’ brace member (with partial or full overlapping) is welded onto the first one. In case the selected brace members are equal in terms of the criteria presented in the table, it is recommended that the brace member in tension shall be first welded to the chord, and the compressed brace member shall be welded onto it [30].

In overlap N joints the vertical member is usually first welded to the chord. However, in case the diagonal member needs to be chosen larger in its external dimensions than the vertical member (the diagonal is typically subjected to higher loading than the vertical), the diagonal shall be placed in the joint as the undermost [30].

In partially overlapping K joints the end of the overlapped (i.e. underneath) brace member is cut using one single mitre-cut, and the end of the overlapping (i.e. on top) brace member is cut twice using two mitre-cuts. **Partially overlapping joints are not allowed to be constructed so that <u>both</u> brace members are cut using two mitre-cuts, since thereby the outcome is that in regard to the side between the brace members, the brace members are welded only into each other (Figure 7.26). In this case <u>neither</u> of the brace members is fully welded to the chord. The resistance of this kind of overlap joint is significantly worse than that of the correctly constructed joint.**

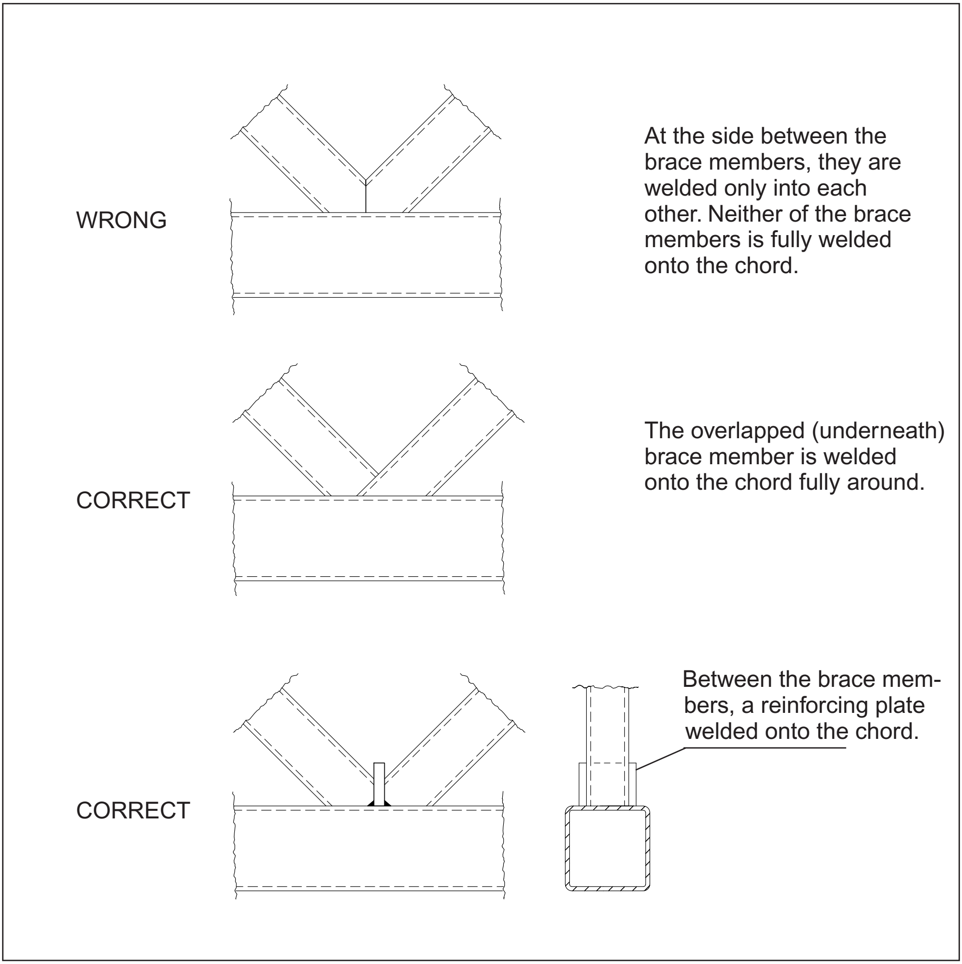


Figure 7.26 An overlap K joint. Incorrect and correct way of executing

Under certain conditions, the hidden seam of an overlapped brace member may be left without welding to the chord (see Table 3.5). SSAB, however, recommends that the hidden seam of the overlapped brace member shall always be welded to the chord to secure the resistance of the joint.

According to the corrigendum [20] published for EN 1993-1-8, in case of overlap joints the following additional assessment shall be performed when verifying the joint resistance:

If $\lambda_{ov} > \lambda_{ov.lim}$ (circular or square or rectangular brace members) or if on rectangular brace members $h_i < b_i$ or $h_j < b_j$, the shear of the brace members off the chord in the direction of the chord shall be checked (Figure 7.27).

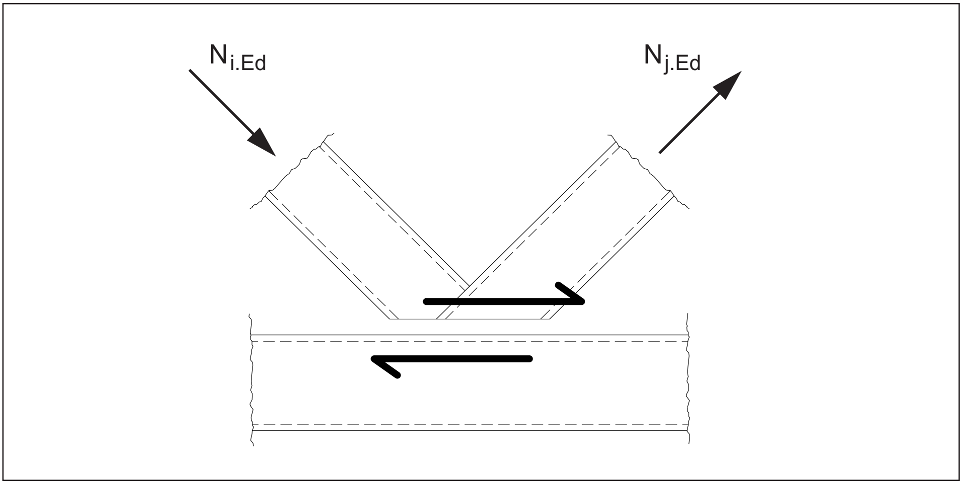


Figure 7.27 Overlap joint. Shear of the brace members off the chord

For above mentioned case, the limit value of overlapping $\lambda_{ov.lim}$ is determined as follows:

- $\lambda_{ov.lim}$ = 60 % , if the hidden seam of the overlapped brace member is not welded to the chord
- $\lambda_{ov.lim}$ = 80 % , if the hidden seam of the overlapped brace member is welded to the chord.

Eurocode does not present, however, instructions how to check the concerned shear resistance. It is anticipated herein, that the following method recommended in this handbook, should be published in the forthcoming Finnish National Annex to EN 1993-1-8:

Circular hollow sections as brace members:

- If: 60 % < λ_{ov} < 100 % when the hidden seam of the overlapped brace member is not welded to the chord
- or: 80 % < λ_{ov} < 100 % when the hidden seam of the overlapped brace member is welded to the chord

the following condition shall be checked:

$$N_{i.Ed} \cos\theta_i + N_{j.Ed} \cos\theta_j \leq N_{s.Rd} \qquad (7.19)$$

where:

$$N_{s.Rd} = \frac{\pi}{4} \cdot \left[\frac{f_{ui}}{\sqrt{3}} \cdot \frac{\left[\left(\frac{100-\lambda_{ov}}{100}\right) \cdot 2d_i + d_{eff.i}\right] \cdot t_i}{\sin\theta_i} + \frac{f_{uj}}{\sqrt{3}} \cdot \frac{(2d_j + c_s d_{eff.j}) \cdot t_j}{\sin\theta_j} \right] \cdot \frac{1}{\gamma_{M5}} \qquad (7.20)$$

- If: λ_{ov} = 100 %

the following condition shall be checked:

$$N_{i.Ed} \cos\theta_i + N_{j.Ed} \cos\theta_j \leq N_{s.Rd} \tag{7.21}$$

where:

$$N_{s.Rd} = \frac{\pi}{4} \cdot \frac{f_{uj}}{\sqrt{3}} \cdot \frac{(3d_j + d_{eff.j}) \cdot t_j}{\sin\theta_j} \cdot \frac{1}{\gamma_{M5}} \tag{7.22}$$

Square or rectangular hollow sections as brace members:

- If: 60 % < λ_{ov} < 100 % when the hidden seam of the overlapped brace member <u>is not</u> welded to the chord
- or: 80 % < λ_{ov} < 100 % when the hidden seam of the overlapped brace member is welded to the chord
- or: $h_i < b_i$ and λ_{ov} < 100 %
- or: $h_j < b_j$ and λ_{ov} < 100 %

the following condition shall be checked:

$$N_{i.Ed} \cos\theta_i + N_{j.Ed} \cos\theta_j \leq N_{s.Rd} \tag{7.23}$$

where:

$$N_{s.Rd} = \left[\frac{f_{ui}}{\sqrt{3}} \cdot \frac{\left[\left(\frac{100 - \lambda_{ov}}{100}\right) \cdot 2h_i + b_{eff.i}\right] \cdot t_i}{\sin\theta_i} + \frac{f_{uj}}{\sqrt{3}} \cdot \frac{(2h_j + c_s b_{eff.j}) \cdot t_j}{\sin\theta_j} \right] \cdot \frac{1}{\gamma_{M5}} \tag{7.24}$$

- If: λ_{ov} = 100 %

the following condition shall be checked:

$$N_{i.Ed} \cos\theta_i + N_{j.Ed} \cos\theta_j \leq N_{s.Rd} \tag{7.25}$$

where:

$$N_{s.Rd} = \frac{f_{uj}}{\sqrt{3}} \cdot \frac{(2h_j + b_j + b_{eff.j}) \cdot t_j}{\sin\theta_j} \cdot \frac{1}{\gamma_{M5}} \tag{7.26}$$

In the formulae (7.19) - (7.26) the subindex i means the overlapping and the subindex j the overlapped brace member.

The other factors are determined as follows:

$N_{s.Rd}$ is the shear resistance of the brace member joint, which shall be reduced using the steel grade dependent correction factor according to Table 3.1

f_u is the nominal ultimate strength of the brace member
c_s is the factor for the effective shear area:
$c_s = 1$, when the hidden seam of the overlapped brace member is not welded to the chord
$c_s = 2$, when the hidden seam of the overlapped brace member is welded to the chord
b_{eff} is the effective width of the brace member according to Table 7.9
d_{eff} is the effective diameter of the brace member according to Table 7.9

Table 7.9 Brace member shear in the overlap joints. Effective dimensions

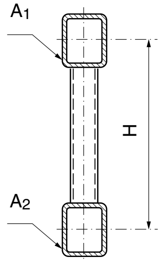
		Brace members	
		Circular (CHS)	Square or rectangular (RHS)
Chord	Circular (CHS)	Overlapping CHS brace member in CHS-chord $d_{eff.i} = \frac{12}{d_0/t_0} \cdot \frac{f_{y0} \cdot t_0}{f_{yi} \cdot t_i} \cdot d_i \le d_i$	–
		Overlapped CHS brace member in CHS-chord $d_{eff.j} = \frac{12}{d_0/t_0} \cdot \frac{f_{y0} \cdot t_0}{f_{yj} \cdot t_j} \cdot d_j \le d_j$	–
	Square or rectangular (RHS)	Overlapping CHS brace member in RHS-chord $d_{eff.i} = \frac{10}{d_0/t_0} \cdot \frac{f_{y0} \cdot t_0}{f_{yi} \cdot t_i} \cdot d_i \le d_i$	Overlapping RHS brace member in RHS-chord $b_{eff.i} = \frac{10}{b_0/t_0} \cdot \frac{f_{y0} \cdot t_0}{f_{yi} \cdot t_i} \cdot b_i \le b_i$
		Overlapped CHS brace member in RHS-chord $d_{eff.j} = \frac{10}{d_0/t_0} \cdot \frac{f_{y0} \cdot t_0}{f_{yj} \cdot t_j} \cdot d_j \le d_j$	Overlapped RHS brace member in RHS-chord $b_{eff.j} = \frac{10}{b_0/t_0} \cdot \frac{f_{y0} \cdot t_0}{f_{yj} \cdot t_j} \cdot b_j \le b_j$
	I-section	Overlapping CHS brace member in I-chord $d_{eff.i} = t_w + 2r + 7t_0 \cdot \frac{f_{y0}}{f_{yi}} \le d_i$	Overlapping RHS brace member in I-chord $b_{eff.i} = t_w + 2r + 7t_0 \cdot \frac{f_{y0}}{f_{yi}} \le b_i$
		Overlapped CHS brace member in I-chord $d_{eff.j} = t_w + 2r + 7t_0 \cdot \frac{f_{y0}}{f_{yj}} \le d_j$	Overlapped RHS brace member in I-chord $b_{eff.j} = t_w + 2r + 7t_0 \cdot \frac{f_{y0}}{f_{yj}} \le b_j$

7.4.8 Estimation of the truss stiffness

Usually, the deflection of the lattice is obtained directly as an output from the lattice design software. In the software, the lattices can be modelled so that the chords are continuous and the brace member joints to the chord are pinned. With gap joints, due to the flexibility of the joints the actual deflection can be 12 - 15 % greater than the calculated deflection [30]. In the pre-design stage it may be necessary to estimate the lattice deflection by manual calculations. The stiffness of the lattice can be calculated approximately by taking into account the effect of the chords only:

$$I = A_1 \cdot H^2 \left[\frac{\frac{A_2}{A_1}}{1 + \frac{A_2}{A_1}} \right]^2 + A_2 \cdot H^2 \left[\frac{1}{1 + \frac{A_2}{A_1}} \right]^2 \qquad (7.27)$$

where A_1 and A_2 are the cross-section areas of the chords
H is the distance of the centroidal axes of the chords



7.4.9 Designing the roof truss of the Example building

Loads:
Design the primary trusses in the Example building. Loads on the lattice consist of the self-weight of the structure and the snow load. The purlin spacing is 4 m, so it is favourable to use the same spacing in the brace member joints. Load factor K_{FI} = 1,1. With the 10 m spacing of primary lattices, the following load on the lattice is obtained:

$$q_d = [\gamma_G \cdot g_k + \gamma_{Q.1} \cdot q_{k.1}]L_f = [(1{,}35 \cdot 1{,}1) \cdot 0{,}5 + (1{,}5 \cdot 1{,}1) \cdot 1{,}2] \cdot 10 = 27{,}2 \text{ kN/m}$$

$$F_{Ed} = q_d \cdot L_p = 27{,}2 \cdot 4 = 108{,}8 \text{ kN} \quad \text{(load on the node)}$$

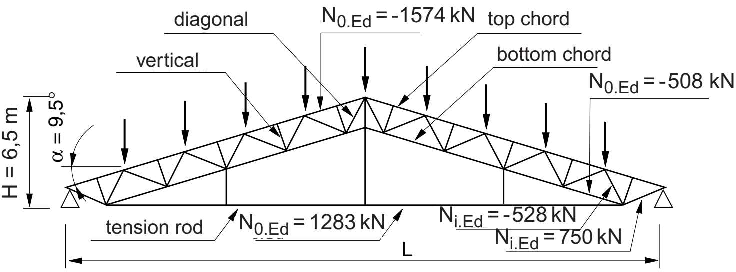
where

L_f *is the frame spacing*
L_p *is the purlin spacing*

Truss shape:
In a roof lattice, the bottom chord can be constructed either straight or scissors-shaped. Normal forces are greater in a scissors-shaped bottom chord. With a straight bottom chord, the height of the lattice is greater in the centre of the span, which is also the location of the maximum moment. Thus, the normal force is smaller in a straight bottom chord. Due to a straight bottom chord, the length (as well as the buckling length) of brace members becomes

greater, which may increase the weight of the truss as compared to a truss with a scissors-shaped bottom chord. In the Example building, the bottom chord is scissors-shaped, but the normal force of the bottom chord is carried by a tension rod. The use of the tension rod reduces the amount of steel in the brace members, but increases the number of joints in the lattice.

The span is long, which makes the shear force small compared to the bending moment. An N truss is thus unnecessarily heavy to be used in the Example building. Since the spacing of the purlins is large, a KT truss is selected for a better top chord resistance. The height of the lattice is estimated by the span. In the Example building, a lattice height of 6,5 m is selected (because the truss is designed using a tension rod, the selected lattice height differs from the height advised in Table 7.8).



Determining the member forces:
The forces of the chords and the brace members can be estimated in pre-design by calculating the lattice forces with the formulae of a simply supported beam:

$$N_{0.Ed} \approx \frac{q_d \cdot L^2}{8H} = \frac{27,2 \cdot 48^2}{8 \cdot 6,5} = 1205\ kN \quad \textit{normal force in top chord}$$

$$N_{i.Ed} \approx 0,5 q_d \cdot L\sqrt{2} = 0,5 \cdot 27,2 \cdot 48\sqrt{2} = 923,2\ kN \quad \textit{normal force in brace member nearest to the support (tension)}$$

For the buckling length of the top chord, for buckling in lateral direction take 90 % of the distance between the purlins and, for buckling in vertical direction take similarly 90 % of the distance between the lattice nodes (clause 7.4.2). Thus the following buckling lengths for the top chord are obtained:

$$L_{cr.y} = 0,9 \cdot \frac{2}{\cos 9,5} = 1,83\ m$$

$$L_{cr.z} = 0,9 \cdot \frac{4}{\cos 9,5} = 3,65\ m$$

In the lattice, the steel grade of all hollow sections is SSAB Domex Tube Double Grade, which fulfills the EN 10219 requirements for both steel grades S420MH and S355J2H. Thereby the design calculations may be performed at designer's own choice either according to grade S420 or grade S355. Grade S420 is chosen in this Example as design basis.

As for the top chord, by having above information and the buckling resistances given in Annex 11.2, hollow section 200×200×8 can be preliminary selected. As for the brace members, based on the tensile force in the brace member nearest to the support, hollow section 120 × 120× 5 can be preliminary selected (the force in the subsequent diagonal already has much smaller value, though being there compression).

The size of the bottom chord is more complex to define manually in this particular case, but thinking of the brace member joint, select a bottom chord as 140×140×5. The normal force in the tension rod can be assumed equal to that in the top chord. Select a tension rod of Ø 80 mm (f_y = 345 N/mm²). The output from the design software gives the following maximum values for the forces:

Top chord:	$N_{0.Ed}$	= - 1574 kN	*(compression)*
	$M_{0.Ed}$	= 11,4 kNm	*(bending at node points due to eccentricity)*
Bottom chord:	$N_{0.Ed}$	= - 508 kN	*(compression)*
Tension rod:	$N_{0.Ed}$	= 1283 kN	*(tension)*
Brace members:	$N_{i.Ed}$	= - 528 kN	*(compression)*
	$N_{i.Ed}$	= 750 kN	*(tension)*

Resistance of the top chord:

First, check the resistance of the top chord made of hollow section 200×200×8. The cross-section classification is Class 1.

The cross-section resistances:

$N_{c.Rd}$ = 2488 kN ≥ 1574 kN (Annex 11.1) OK

$M_{pl.y.Rd}$ = 176,8 kNm ≥ 11,4 kNm (Annex 11.1) OK

(M+N) interaction:

the normal force is so high, that it reduces the bending resistance (expressions (2.123a) and (2.123b)):

$$n = \frac{N_{Ed}}{N_{pl.Rd}} = \frac{1574}{2488} = 0,6326$$

$$a_w = \frac{A - 2bt}{A} = \frac{5924 - 2 \cdot 200 \cdot 8}{5924} = 0,4598 \le 0,5$$

$$M_{N.y.Rd} = M_{pl.y.Rd} \frac{1-n}{1-0,5a_w} = 176,8 \cdot \frac{1-0,6326}{1-0,5 \cdot 0,4598} = 84,3\ kNm \le M_{pl.y.Rd}$$

$$M_{N.y.Rd} = 84,3\ kNm \ge 11,4\ kNm \quad OK$$

Buckling resistance:

$N_{b.y.Rd}$ = 2316 kN (L_{cr} = 1,83 m , $\bar{\lambda}_y$ = 0,3358 , χ_y = 0,9307)

$N_{b.Rd} = N_{b.z.Rd}$ = 1850 kN (L_{cr} = 3,65 m , $\bar{\lambda}_z$ = 0,6697 , χ_z = 0,7434)

Combined effect of buckling and bending:

On square hollow sections, lateral-torsional buckling does not reduce the bending resistance of the cross-section. Thus, when checking the combined effect of (M+N), the reduction factor for lateral-torsional buckling can be taken as $\chi_{LT} = 1{,}0$:

$\psi = 0$ *(Table 2.25, a conservative simplification)*

$$M_{Ed} = 11{,}4\ kNm$$

$$C_{my} = 0{,}6 + 0{,}4\psi = 0{,}6 + 0 = 0{,}6 \geq 0{,}4$$

From Table 2.23 (cross-section Class 1):

$$k_{yy} = C_{my}\left[1 + (\bar{\lambda}_y - 0{,}2) \cdot \frac{N_{Ed}}{\chi_y \cdot N_{Rk}/\gamma_{M1}}\right]$$

$$= 0{,}6 \cdot \left[1 + (0{,}3358 - 0{,}2) \cdot \frac{1574 \cdot 10^3}{0{,}9307 \cdot 5924 \cdot 420/1{,}0}\right] = 0{,}6554$$

$$k_{yy} \leq C_{my}\left[1 + 0{,}8 \cdot \frac{N_{Ed}}{\chi_y \cdot N_{Rk}/\gamma_{M1}}\right] = 0{,}6\left[1 + 0{,}8 \cdot \frac{1574 \cdot 10^3}{0{,}9307 \cdot 5924 \cdot 420/1{,}0}\right] = 0{,}9263$$

$$\Rightarrow k_{yy} = 0{,}6554$$

$$k_{zy} = 0{,}6k_{yy} = 0{,}6 \cdot 0{,}6554 = 0{,}3932$$

$$\frac{N_{Ed}}{\chi_y \dfrac{N_{Rk}}{\gamma_{M1}}} + k_{yy}\frac{M_{y.Ed} + \Delta M_{y.Ed}}{\chi_{LT}\dfrac{M_{y.Rk}}{\gamma_{M1}}} + k_{yz}\frac{M_{z.Ed} + \Delta M_{z.Ed}}{\dfrac{M_{z.Rk}}{\gamma_{M1}}} =$$

$$\frac{1574 \cdot 10^3}{0{,}9307 \cdot \dfrac{5924 \cdot 420}{1{,}0}} + 0{,}6554 \cdot \frac{11{,}4 \cdot 10^6 + 0}{1{,}0 \cdot \dfrac{420{,}9 \cdot 10^3 \cdot 420}{1{,}0}} + 0 =$$

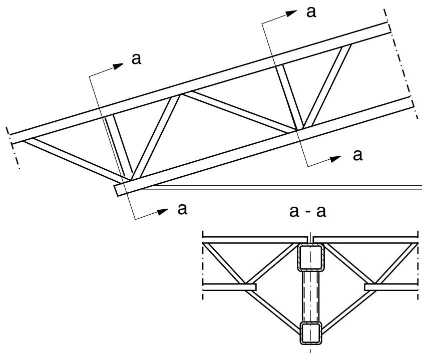
$$0{,}6797 + 0{,}0423 = 0{,}7220 \leq 1{,}0 \quad OK$$

$$\frac{N_{Ed}}{\chi_z \dfrac{N_{Rk}}{\gamma_{M1}}} + k_{zy}\frac{M_{y.Ed} + \Delta M_{y.Ed}}{\chi_{LT}\dfrac{M_{y.Rk}}{\gamma_{M1}}} + k_{zz}\frac{M_{z.Ed} + \Delta M_{z.Ed}}{\dfrac{M_{z.Rk}}{\gamma_{M1}}} \leq 1{,}0$$

$$\frac{1574 \cdot 10^3}{0{,}7434 \cdot \dfrac{5924 \cdot 420}{1{,}0}} + 0{,}3932 \cdot \frac{11{,}4 \cdot 10^6 + 0}{1{,}0 \cdot \dfrac{420{,}9 \cdot 10^3 \cdot 420}{1{,}0}} + 0 =$$

$$0{,}8510 + 0{,}0254 = 0{,}8764 \leq 1{,}0 \quad OK$$

Designing the bottom chord and the tension rod:



The use of the tension rod generates compression in the bottom chord. Compression is present only in the first diagonal spacing, after which the bottom chord is subjected to tension. Brace the compression element of the bottom chord laterally to the purlin with anti-sag rods. The buckling length of the bottom chord 140×140×5 is equal in both directions:

$$L_{cr} = 0,9 \cdot \frac{4}{cos 9,5} = 3,65 \ m$$

$$N_{b.Rd} = \frac{\chi \cdot A \cdot f_y}{\gamma_{M1}} = \frac{0,5706 \cdot 2636 \cdot 420}{1,0}$$

$$= 631,7 \ kN \geq 508 \ kN \quad OK$$

The resistance of the tension rod (Ø 80 mm) is determined simply by the area:

$$N_{t.Rd} = \frac{A \cdot f_y}{\gamma_{M0}} = \frac{\pi \cdot 40^2 \cdot 345}{1,0} \quad (\varnothing > 16 \ mm \Rightarrow f_y = 345 \ N/mm^2)$$

$$= 1734 \ kN \geq 1283 \ kN \quad OK$$

Resistance of the brace members:

The resistance of the tension brace member 120×120×5 is determined in a similar way to that of the tension rod:

$$N_{t.Rd} = \frac{A \cdot f_y}{\gamma_{M0}} = \frac{2236 \cdot 420}{1,0} = 939,1 \ kN \geq 750 \ kN$$

The buckling length of the compression brace member 120×120×5 is 0,75L (clause 7.4.3):

$L_{cr} = 0,75 \cdot 3,54 = 2,66 \ m$

Above buckling length is applied to determine the buckling resistance:

$$N_{b.Rd} = \frac{\chi \cdot A \cdot f_y}{\gamma_{M1}} = \frac{0,6541 \cdot 2236 \cdot 420}{1,0} = 614,3 \ kN \geq 528 \ kN$$

In brace members subjected to smaller loads, a smaller hollow section can be used.

Resistance of the truss joints:

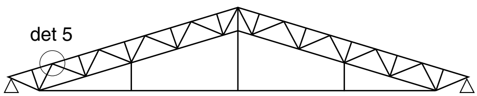
To simplify the workshop fabrication, the joints of the top chord are herein constructed to be gap joints. However, the joints of the bottom chord are constructed to be overlap joints in order to keep the eccentricities small.

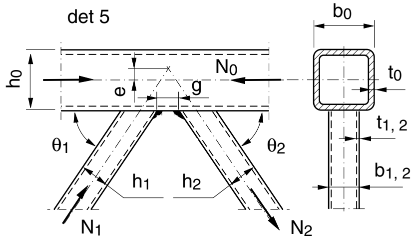
The throat thicknesses for the welded lattice joints are determined according to Table 3.9.

In a simply supported lattice girder, the shear force is at its greatest in the vicinity of the supports, thus that is where also the brace member forces are at their greatest. Due to this, the resistance of the joints at the two outermost purlins (joints 1 and 5 in the calculation model) shall be checked. ***In a real design case, resistance of all joints must be checked.***

Truss joint No. 5:

Joint No.5





Joint No.5 is a gap K joint. The formulae for this type of joint are given in Table 11.3.2. The geometry and forces at the joint are as follows:

Chord: $200 \times 200 \times 8$ $(A_0 = 5924\ mm^2)$
Brace members: $120 \times 120 \times 5$ $(A_i = 2236\ mm^2)$

$N_{0.Ed} = -945\ kN$ *(compression)*
$N_{1.Ed} = -528\ kN$ *(compression)*
$N_{2.Ed} = 528\ kN$ *(tension)*

$\theta_1 = \theta_2 = 54°$
$g = 50\ mm$ *(gap)*

$$\beta = \frac{b_1 + b_2 + h_1 + h_2}{4b_0} = \frac{120 + 120 + 120 + 120}{4 \cdot 200} = 0,6$$

$$\gamma = \frac{b_0}{2t_0} = \frac{200}{2 \cdot 8} = 12,5$$

Check the validity conditions of the joint's gap:

$$g = 50\ mm \ge t_1 + t_2 = 5 + 5 = 10\ mm \quad OK$$

$$g/b_0 = 50/200 = 0,25 \ge 0,5(1-\beta) = 0,5 \cdot (1-0,6) = 0,2 \quad OK$$

$$g/b_0 = 0,25 \le 1,5(1-\beta) = 1,5 \cdot (1-0,6) = 0,6 \quad OK$$

$\Rightarrow$ *the joint's gap meets the validity conditions*

Check the validity conditions of the joint's eccentricity:

$$e = \left(\frac{h_1}{2\sin\theta_1} + \frac{h_2}{2\sin\theta_2} + g\right)\frac{\sin\theta_1 \cdot \sin\theta_2}{\sin(\theta_1 + \theta_2)} - \frac{h_0}{2} = 36,5mm$$

$$-0,55h_0 = -110\ mm \le e = 36,5\ mm \le 0,25h_0 = 50\ mm \quad OK$$

$\Rightarrow$ *the joint's eccentricity meets the validity conditions*

The chord and the brace members are classified in cross-section Class 1. Thus, in this respect, there is no need to study the joint's geometrical validity conditions any further. Also the other validity conditions regarding the joint's geometry are met, though the assessment procedure for them is not presented here.

When calculating the resistance of the joint, only the brace member 1 needs to be considered, because both brace members are equal in respect to their size, their angle and their loading.

Chord face failure by yielding:

$$n = \frac{\sigma_{0.Ed}}{f_{y0}/\gamma_{M5}} = \frac{N_{0.Ed}}{A_0 f_{y0}/\gamma_{M5}} + \frac{M_{0.Ed}}{W_{el.0} f_{y0}/\gamma_{M5}}$$

$$= \frac{945 \cdot 10^3}{5924 \cdot 420/1,0} + 0 = 0,3798$$

(the bending moment due to the joint eccentricity can be disregarded when the eccentricity is within the permitted limits)

$$k_n = 1,3 - \frac{0,4|n|}{\beta} = 1,3 - \frac{0,4 \cdot 0,3798}{0,6} = 1,047 > 1,0 \quad \Rightarrow k_n = 1,0$$

$$N_{1.Rd} = 0,9 \cdot \frac{8,9 \cdot k_n \cdot f_{y0} \cdot t_0^2 \cdot \sqrt{\gamma}}{\sin\theta_1} \cdot \left(\frac{b_1 + b_2 + h_1 + h_2}{4b_0}\right)/\gamma_{M5} \quad \text{(S420: factor = 0,9)}$$

$$= 0,9 \cdot \frac{8,9 \cdot 1,0 \cdot 420 \cdot 8^2 \cdot \sqrt{12,5}}{\sin 54} \cdot \left(\frac{120 + 120 + 120 + 120}{4 \cdot 200}\right)/1,0$$

$$= 564,6\ kN \ge 528\ kN \quad OK$$

Chord face punching shear:
Since $\beta = 0{,}6 \leq 1 - (1/\gamma) = 0{,}92$, chord face punching shear must be checked:

$$b_{e.p} = \frac{10}{b_0/t_0} \cdot b_1 = \frac{10}{200/8} \cdot 120 = 48\ mm \leq b_1 = 120\ mm$$

$$N_{1.Rd} = 0,9 \cdot \frac{f_{y0} \cdot t_0}{\sqrt{3} \sin\theta_1} \cdot \left(\frac{2h_1}{\sin\theta_1} + b_1 + b_{e.p}\right) / \gamma_{M5} \qquad \text{(S420: resistance factor = 0,9)}$$

$$= 0,9 \cdot \frac{420 \cdot 8}{\sqrt{3} \sin 54} \cdot \left(\frac{2 \cdot 120}{\sin 54} + 120 + 48\right) / 1,0 = 1003\ kN \geq 528\ kN \quad OK$$

Chord shear:
The shear resistance of the chord <u>at the location of the joint</u> is:

$$\alpha = \sqrt{\frac{1}{1 + \frac{4g^2}{3t_0^2}}} = \sqrt{\frac{1}{1 + \frac{4 \cdot 50^2}{3 \cdot 8^2}}} = 0,1373$$

$$A_{v0} = (2h_0 + \alpha b_0)t_0 = (2 \cdot 200 + 0,1373 \cdot 200) \cdot 8 = 3420\ mm^2$$

$$N_{1.Rd} = 0,9 \cdot \frac{f_{y0} \cdot A_{v0}}{\sqrt{3} \sin\theta_1} / \gamma_{M5} \qquad \text{(S420: resistance factor = 0,9)}$$

$$= 0,9 \cdot \frac{420 \cdot 3420}{\sqrt{3} \sin 54} / 1,0 = 922,6\ kN \geq 528\ kN \quad OK$$

Chord's resistance to normal force at the location of the <u>joint's gap</u>:

$$N_{0.gap.Ed} = N_{0.Ed} - N_{2.Ed} \cos\theta_2 = 945 - 528 \cdot \cos 54 = 634,6\ kN$$

$$V_{0.gap.Ed} = N_{1.Ed} \sin\theta_1 = 528 \cdot \sin 54 = 427,2\ kN$$

$$A_{V0} = A_0 \cdot \frac{h_0}{b_0 + h_0} = 5924 \cdot \frac{200}{200 + 200} = 2962\ mm^2$$

$$V_{pl.Rd} = A_{V0} \cdot \frac{f_{y0}/\sqrt{3}}{\gamma_{M0}} = 2962 \cdot \frac{420/\sqrt{3}}{1,0} = 718,2\ kN$$

$$N_{0.gap.Rd} = 0,9 \cdot [(A_0 - A_{v0})f_{y0} + A_{v0}f_{y0}\sqrt{1 - (V_{0.gap.Ed}/V_{pl.Rd})^2}] / \gamma_{M5} \qquad \text{(S420 = 0,9)}$$

$$= 0,9 \cdot [(5924 - 3420) \cdot 420 + 3420 \cdot 420 \cdot \sqrt{1 - (427,2/718,2)^2}] / 1,0 = 1986\ kN$$

$$N_{0.gap.Rd} = 1986\ kN \geq N_{0.gap.Ed} = 634,6\ kN \quad OK$$

Brace member failure by yielding:
The effective width of the brace member is:

$$b_{eff} = \frac{10}{b_0/t_0} \cdot \frac{f_{y0} \cdot t_0}{f_{y1} \cdot t_1} \cdot b_1 = \frac{10}{200/8} \cdot \frac{420 \cdot 8}{420 \cdot 5} \cdot 120 = 76,8\ mm \le b_1 = 120\ mm$$

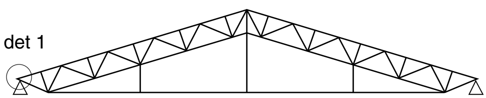
$$N_{1.Rd} = 0,9 \cdot f_{y1} t_1 (2h_1 - 4t_1 + b_1 + b_{eff}) / \gamma_{M5} \quad (S420\text{: resistance factor} = 0,9)$$

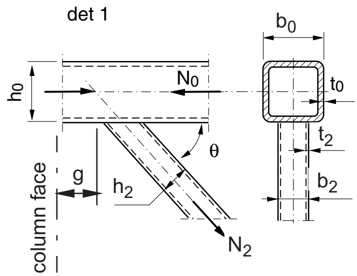
$$= 0,9 \cdot 420 \cdot 5 \cdot (2 \cdot 120 - 4 \cdot 5 + 120 + 76,8) / 1,0 = 787,8\ kN \ge 528\ kN \quad OK$$

Joint's resistance:
The governing failure mode is thus chord face failure by yielding:
$N_{1.Rd} = 564{,}6\ kN \ge N_{1.Ed} = 528\ kN$ *OK*

Truss joint No. 1 (truss joint at the support):

Joint No.1





At the support, the truss joint acts as a K/N joint where the column now represents fictitious brace member 1 (the compressed brace member). The top end of the column is provided with an end-plate, to which the truss will be connected. Due to this, the general validity conditions regarding truss joints can be omitted regarding the column. Thus, in respect to the truss itself, the resistance of the joint No. 1 shall be determined only in regard to the actual brace member 2.

First, check the resistance without any additional reinforcing plates. The geometry and forces at the joint are as follows:

Chord: $200 \times 200 \times 8$ *(cross-section Class 1,* $A_0 = 5924\ mm^2$*)*
Brace member: $120 \times 120 \times 5$ *(cross-section Class 1,* $A_i = 2236\ mm^2$*)*
Column (='brace member 1'): $300 \times 200 \times 6$

$$N_{0.Ed} = -527\ kN \quad (\text{compression})$$

$$N_{1.Ed} = 27,2 \cdot 24 = 653\ kN \quad (\text{compressive load to the column})$$

$$N_{2.Ed} = 750\ kN \quad (\text{tension})$$

$$\theta = 54°$$

$$\beta = \frac{b_1 + b_2 + h_1 + h_2}{4b_0} = \frac{200 + 120 + 300 + 120}{4 \cdot 200} = 0,925$$

$$\gamma = \frac{b_0}{2t_0} = \frac{200}{2 \cdot 8} = 12,5$$

$$g = 65\ mm$$

The joint shall be designed as a K/N joint or as a Y joint depending on the value of the gap (clause 7.4.5):

$$g = 65\ mm \geq t_1 + t_2 = 6 + 5 = 11\ mm \quad OK \quad (t_1 \text{ is now the column wall thickness})$$

$$g = 65\ mm \geq 0,5(1-\beta)b_0 = 0,5 \cdot (1 - 0,925) \cdot 200 = 7,5\ mm \quad OK$$

$$g = 65\ mm < 1,5(1-\beta)b_0 = 1,5 \cdot (1 - 0,925) \cdot 200 = 22,5\ mm \quad not\ OK$$

⇒ the validity conditions set for the gap in a K/N joint are not met
⇒ the joint shall be designed as a Y joint

Joint assessment as a Y joint (Table 11.3.1):
The validity conditions regarding the joint's geometry are met, though the assessment procedure for them is not presented here.

The normal stress $\sigma_{0.Ed}$ in the chord face has an impact on the joint's resistance by the parameter k_n, which is now determined by knowing the column support reaction (653 kN) and the moment generated by the lateral eccentricity (200 mm) in the joint. Now, calculate the stress acting in the chord face:

$$\frac{N_{0.Ed}}{A_0} = \frac{527 \cdot 10^3}{5924} = 89,0\ N/mm^2 \qquad \text{compressive stress due to the normal force}$$

$$\frac{M_{0.Ed}}{W_{el.0}} = \frac{653 \cdot 10^3 \cdot 200}{356,6 \cdot 10^3} = 366\ N/mm^2 \qquad \text{tensile stress due to the bending moment}$$

$$\sigma_{0.Ed} = 366 - 89 = 277\ N/mm^2 \qquad \text{tension} \Rightarrow k_n = 1,0$$

$$\beta = \frac{b_2}{b_0} = \frac{120}{200} = 0,6$$

$$\eta = h_2/b_0 = 120/200 = 0,6$$

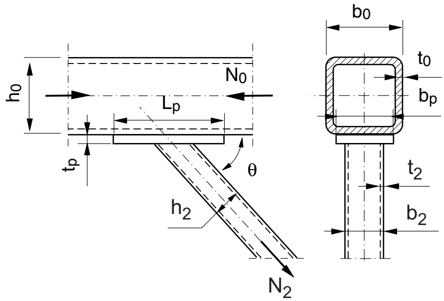
$\beta \leq 0,85$ ⇒ chord face failure by yielding:

$$N_{2.Rd} = 0,9 \cdot \frac{k_n \cdot f_{y0} \cdot t_0^2}{(1-\beta)\sin\theta_2} \cdot \left(\frac{2\eta}{\sin\theta_2} + 4\sqrt{1-\beta}\right)/\gamma_{M5} \qquad \text{(S420: resistance factor = 0,9)}$$

$$= 0,9 \cdot \frac{1,0 \cdot 420 \cdot 8^2}{(1-0,6) \cdot \sin 54} \cdot \left(\frac{2 \cdot 0,6}{\sin 54} + 4\sqrt{1-0,6}\right)/\gamma_{M5}$$

$$= 300,0\ kN < 750\ kN \quad not\ OK$$

The joint's resistance is not sufficient because $N_{2.Ed} = 750\ kN > N_{2.Rd}$. Since the chord face is now critical, an effective way to improve the joint's resistance is a reinforcing plate to be welded to the chord face. In this Example, the reinforcement of the joint is profitable, since only the outermost joints need to be reinforced.



Resistance of a joint reinforced on the chord face (Table 11.3.17):

Calculate the joint's resistance when a plate 275×185×15 made of steel grade S420 (f_y = 420 N/mm²) is welded to the chord face:

$$f_{yp} = 420\ N/mm^2 \geq f_{y0} = 420\ N/mm^2 \quad OK$$

$$b_p = 185\ mm \geq b_0 - 2t_0 = 200 - 2 \cdot 8 = 184\ mm \quad OK$$

$$t_p = 15\ mm \geq 2t_2 = 2 \cdot 5 = 10\ mm \quad OK$$

$$L_p = 275\ mm \geq \frac{h_2}{\sin\theta_2} + \sqrt{b_p(b_p - b_2)} = \frac{120}{\sin 54} + \sqrt{185(185 - 120)} = 258\ mm \quad OK$$

⇒ the reinforcing plate meets the geometrical validity conditions OK

$$\beta_p = \frac{b_2}{b_p} = \frac{120}{185} = 0,6486$$

$$\eta_p = \frac{h_2}{b_p} = \frac{120}{185} = 0,6486$$

Chord face failure by yielding:

$$N_{2.Rd} = 0,9 \cdot \frac{f_{yp} \cdot t_p^2}{(1 - \beta_p)\sin\theta_2} \cdot \left(\frac{2\eta_p}{\sin\theta_2} + 4\sqrt{1 - \beta_p}\right) / \gamma_{M5} \quad \textit{(S420: resistance factor = 0,9)}$$

$$= 0,9 \cdot \frac{420 \cdot 15^2}{(1 - 0,6486) \cdot \sin 54} \cdot \left(\frac{2 \cdot 0,6486}{\sin 54} + 4\sqrt{1 - 0,6486}\right) / 1,0$$

$$= 1189\ kN \geq 750\ kN \quad OK$$

Chord face punching shear:

$$b_{e.p} = \frac{10}{b_p / t_p} \cdot b_2 = \frac{10}{185/15} \cdot 120 = 97,3 \ mm \le b_2 = 120 \ mm$$

$$N_{2.Rd} = 0,9 \cdot \frac{f_{yp} \cdot t_p}{\sqrt{3} \sin\theta_2} \cdot \left(\frac{2h_2}{\sin\theta_2} + 2b_{e.p} \right) / \gamma_{M5} \qquad \text{(S420: resistance factor = 0,9)}$$

$$= 0,9 \cdot \frac{420 \cdot 15}{\sqrt{3} \sin 54} \cdot \left(\frac{2 \cdot 120}{\sin 54} + 2 \cdot 97,3 \right) / 1,0 = 1988 \ kN \ge 750 \ kN \quad OK$$

Chord side wall buckling / yielding:
$N_{2.Ed}$ *causes tension in chord web* $\Rightarrow$

$$f_b = f_{y0} = 420 \ N/mm^2$$

$$N_{2.Rd} = 0,9 \cdot \frac{f_b \cdot t_0}{\sin\theta_2} \cdot \left(\frac{2h_2}{\sin\theta_2} + 10t_0 \right) / \gamma_{M5} \qquad \text{(S420: resistance factor = 0,9)}$$

$$= 0,9 \cdot \frac{420 \cdot 8}{\sin 54} \cdot \left(\frac{2 \cdot 120}{\sin 54} + 10 \cdot 8 \right) / 1,0 = 1408 \ kN \ge 750 \ kN \quad OK$$

Brace member failure by yielding:
The effective width of the brace member is:

$$b_{eff} = \frac{10}{b_p / t_p} \cdot \frac{f_{yp} \cdot t_p}{f_{y2} \cdot t_2} \cdot b_2 = \frac{10}{185/15} \cdot \frac{420 \cdot 15}{420 \cdot 5} \cdot 120 = 292 \ mm > b_2 = 120 \ mm$$

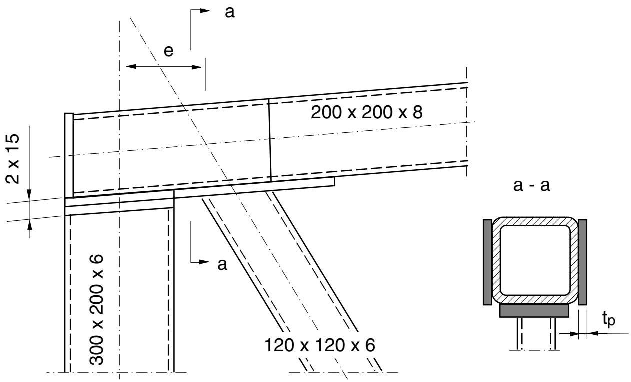
$$\Rightarrow b_{eff} = b_2 = 120 \ mm$$

$$N_{2.Rd} = 0,9 \cdot f_{y2} t_2 (2h_2 - 4t_2 + 2b_{eff}) / \gamma_{M5} \qquad \text{(S420: resistance factor = 0,9)}$$

$$= 0,9 \cdot 420 \cdot 5 \cdot (2 \cdot 120 - 4 \cdot 5 + 2 \cdot 120) / 1,0 = 869,4 \ kN \ge 750 \ kN \quad OK$$

Joint's resistance:
The governing failure mode is thus brace member failure by yielding:
$N_{2.Rd} = 869,4 \ kN \ge N_{2.Ed} = 750 \ kN$ *OK*

Resistance of the top chord at the support:



The truss-to-column joint is constructed by using an end-plate. In order to make the workshop fabrication easier, some eccentricity is permitted in respect to the column neutral axis. Consequently, the top chord must be able to transfer the shear force from the truss to the column. The eccentricity generates also bending moment in the top chord. When having the lateral eccentricity of 200 mm, the end of the top chord is subjected to the following actions:

V_{Ed} = 653 kN
M_{Ed} = 653 · 0,2 = 130,6 kNm
N_{Ed} = 527 - 750 cos54 = 86,2 kN *compression (at left from joint No.1)*

Resistance of the chord end to the combined load:

The chord's shear resistance $V_{pl.Rd}$ = 718,2 kN is sufficient to carry the shear force V_{Ed} = 653 kN. Also the chord cross-section's bending resistance $M_{pl.Rd}$ = 176,8 kNm and resistance for (M+N+V) interaction are sufficient (this is a task for the Reader to check). However, later on it would turn out that the resistance of the chord end to a concentrated force is not sufficient, thus the chord side walls must be reinforced with side plates. Due to this, the required side plates will be considered already in the following assessments.

Reinforce the chord with full-height plates welded to the chord webs. The thickness of the plates is t_p = 5 mm. To simplify the calculations, the plates are of the same steel grade S420 (f_y = 420 N/mm^2) as the chord. Now, as the need for reinforcing is not initiated by the joint resistance but by the shear resistance of the tube itself, the thickness of the reinforcing plates does not need to meet the validity conditions set for the reinforced truss joints. The reinforment plates extend beyond the brace member joint. The plates welded to the chord sides are *taken into account when calculating the resistance of the chord end. The resistances of the reinforced top chord are as follows:*

$$V_{pl.Rd} = \left[A \cdot \frac{h}{b+h} + 2(t_p \cdot h_p)\right] \frac{f_y}{\sqrt{3} \cdot \gamma_{M0}}$$

$$= \left[5924 \cdot \frac{200}{200+200} + 2 \cdot (5 \cdot 200)\right] \frac{420}{\sqrt{3} \cdot 1,0} = 1203\ kN \geq 653\ kN \quad OK$$

$$N_{pl.Rd} = [A + 2(t_p \cdot h_p)]\frac{f_y}{\gamma_{M0}} = [5924 + 2 \cdot (5 \cdot 200)]\frac{420}{1,0} = 3328\ kN \geq 86,2\ kN \quad OK$$

$$M_{pl.Rd} = \left[W_{pl} + 2\frac{t_p h_p^2}{4}\right]\frac{f_y}{\gamma_{M0}} = \left[420,9 \cdot 10^3 + 2 \cdot \frac{5 \cdot 200^2}{4}\right]\frac{420}{1,0} = 218,8\ kNm$$

Now, the shear force exceeds half of the shear resistance, thus its impact must be considered when calculating the bending resistance:

$$\rho = \left[\frac{2V_{Ed}}{V_{pl.Rd}} - 1\right]^2 = \left[\frac{2 \cdot 653}{1203} - 1\right]^2 = 0,0073$$

$$M_{V.Rd} = \left(W_{pl} - \frac{\rho \cdot A_V^2}{8t}\right) \cdot \frac{f_y}{\gamma_{M0}} = \left(520,9 \cdot 10^3 - \frac{0,0073 \cdot 4962^2}{8 \cdot 13}\right) \cdot \frac{420}{1,0}$$

$$= 218,1\ kNm \leq M_{c.Rd} = M_{pl.Rd} = 218,8\ kNm$$

In addition, check the impact of the normal force to bending resistance by applying clause 2.9.1.6.1.1 (the side plates shall also be taken into account):

$$A_{w.red} = (1-\rho)(A - 2bt) = (1 - 0,0073)(7924 - 2 \cdot 200 \cdot 8) = 4690\ mm^2$$

$$A_{tot.red} = A - \rho(A - 2bt) = 7924 - 0,0073 \cdot (7924 - 2 \cdot 200 \cdot 8) = 7890\ mm^2$$

$$N_{V.Rd} = A_{tot.red} f_y / \gamma_{M0} = 7890 \cdot 420/1,0 = 3314\ kN$$

$$N_{Ed} = 86,2\ kN \leq \frac{0,5 A_{w.red} f_y}{\gamma_{M0}} = \frac{0,5 \cdot 4690 \cdot 420}{1,0} = 984,9\ kN$$

and:

$$N_{Ed} = 86,2\ kN \leq 0,25 N_{V.Rd} = 0,25 \cdot 3314 = 828,5\ kN$$

$\Rightarrow$ *the normal force is so small that it has no effect on bending resistance:*

$$M_{N.V.Rd} = M_{V.Rd} = 218,1\ kNm \geq 130,6\ kNm \quad OK$$

<u>*Resistance of the chord end to concentrated force:*</u>

Next, calculate the resistance of the top chord webs to the support reaction by using the formulae presented in clause 2.11. The end of the top chord is closed with a plate, and the gap between the corner rounding and the splice plate is filled with a weld. The thickness of the splice plate welded under the chord as well as the thickness of the end-plate in the column are both $t_p = 15$ mm.

The value of the concentrated load per one web is:

$$F_{Ed} = \frac{V_{Ed}}{2} = \frac{653}{2} = 326,5\ kN$$

Assume that due to the deflection of the lattice, the support reaction will focus on the inner face of the column 300×200×6 (a conservative simplification). The support reaction is distributed at 45° angle from the column through the splice plates. Hence the length of stiff bearing is:

$$s_s = 6 + 2(t_p + t_p) = 6 + 2 \cdot (15 + 15) = 66\ mm$$

Since the chord end is stiffened by closing it with a plate, the concentrated load may now be considered as 'a loading placed far from the beam end', no matter does it actually meet the required distance to the beam end (Table 2.26, load type 1b). From Table 2.26:

$$\frac{s_s}{t} = \frac{66}{8+5} = 5,08 \le 60$$

$$\Rightarrow$$

$$k = f_y / 228 = 420/228 = 1,842$$

$$k_3 = 1,0$$

$$k_4 = 1,22 - 0,22k = 1,22 - 0,22 \cdot 1,842 = 0,8148$$

$$\frac{r_i}{t} = \frac{12}{8+5} = 0,9231 \quad (r_i \text{ is the internal corner radius of the hollow section chord})$$

$$k_5 = 1,06 - 0,06 \cdot (r_i/t) = 1,06 - 0,06 \cdot 0,9231 = 1,005 \quad \text{but } k_5 \le 1,0 \Rightarrow k_5 = 1,0$$

$$C_F = k_3 k_4 k_5 \left[14,7 - \frac{(h-t)/t}{49,5}\right]\left[1 + 0,007 \cdot \frac{s_s}{t}\right]$$

$$= 1,0 \cdot 0,8148 \cdot 1,0 \cdot \left[14,7 - \frac{(200-8)/13}{49,5}\right]\left[1 + 0,007 \cdot \frac{66}{13}\right] = 12,15$$

$$F_{1w.Rd} = C_F \cdot \frac{t^2 f_y}{\gamma_{M1}} = 12,15 \cdot \frac{13^2 \cdot 420}{1,0} = 862,4\ kN \ge 326,5\ kN \quad OK$$

The above resistance has been calculated by having the thickness of the side plates added to the thickness of the webs. Thus, the calculatory thickness of a web is $t = 8 + 5 = 13$ mm.

Deflection of the lattice:
For the lattice, the second moment of area is estimated with formula (7.18):

$$I = A_1 \cdot H^2 \left[\frac{\frac{A_2}{A_1}}{1 + \frac{A_2}{A_1}} \right]^2 + A_2 \cdot H^2 \left[\frac{1}{1 + \frac{A_2}{A_1}} \right]^2$$

$$= 5924 \cdot 6500^2 \left[\frac{\frac{5027}{5924}}{1 + \frac{5027}{5924}} \right]^2 + 5027 \cdot 6500^2 \left[\frac{1}{1 + \frac{5027}{5924}} \right]^2 = 0,1149\ m^4$$

where

A_1 *is the cross-section area of the top chord*
A_2 *is the cross-section area of the tension rod*

At serviceability limit state, the partial safety factors for loads are as determined in formula (7.10). Thus, the following design load is obtained for the lattice (load factor K_{FI} *is not applied at serviceability limit state):*

$$q_{d.SLS} = (g_k + q_k)L_f = (0,5 + 1,2) \cdot 10 = 17,0\ kN/m \quad \text{self-weight + snow}$$

where

L_f *is the frame spacing*

The deflection is calculated by using the formulae for simply supported beam:

$$\delta = \frac{5q_{d.SLS} \cdot L^4}{384EI} = \frac{5 \cdot 17,0 \cdot 10^3 \cdot 48^4}{384 \cdot 2,1 \cdot 10^{11} \cdot 0,1149} = 49\ mm = \frac{L}{980} \le \frac{L}{300} \quad OK\ \ (Table\ 7.5)$$

Comparison S420 vs S355:
In case the design calculations would be performed according to grade S355, the resistance of the lattice would not be sufficient with the selected hollow sections.

It should be noted, that a more detailed comparison is not possible by considering only single members and joints, since in a real design case the lattice should be considered in its entirety, where the entire lattice geometry (i.e. the height of the lattice and the number/spacing of the diagonal and vertical members) and the given design input data (external loading and the truss span) are considered together with the selected steel grade and available hollow sections sizes. If the selected steel grade is changed, usually also the lattice geometry should be adjusted, if the optimum solution should be found. Only by this it is possible to find out the final benefits by different steel grades, in respect to the steel amount and overall costs. Thus, a universally valid comparison is not possible to present.

7.5 Stiffening of the structure

The following stiffening systems (bracing systems) or their combinations may be used to stiffen a steel framework:

- lattice stiffening
- shear wall stiffening
- frame stiffening
- mast stiffening
- core stiffening.

Lattice stiffening

Lattice stiffening is commonly used as a stiffening system or as a part of it, for example, in industrial buildings. It is also very suitable for stiffening tall buildings. The deformations of the stiffening lattice can be diminished by increasing the breadth of the lattice. Part of the deflection of the lattice arises, however, from the elongations of the members of the lattice. This can be diminished by reducing the stresses in the members of the lattice, i.e. by selecting larger cross-section for the lattice members.

Shear wall stiffening

A building can be braced using suitably chosen stiffening walls extending from the bottom to the top. The walls, rigidly connected to the steel framework, can be cast in-situ or they may be constructed from prefabricated elements. In some cases, corrugated sheeting may be used as stiffening elements to form a stressed skin. Stiffening walls are used, for example, in commercial buildings and office buildings as well as in residential buildings together with stiffening stair shafts.

Frame stiffening

Frame stiffening is mainly used in low-rise single-storey buildings. The displacements of the building are larger than, for example, in a building with lattice stiffening, because the horizontal loads are transferred to the foundations by means of the rigid joints of the frame.

Mast stiffening

Mast columns work as the stiffening part of the building. This method is only suitable for low-rise buildings because the bending moment transferred to the foundations grows rapidly in taller buildings and, in addition, the displacements of the top of the building are large.

Core stiffening

The elevator shaft or stair shaft of the building serve as stiffening structures either alone or together with the stiffening walls. This kind of stiffening is commonly used both in residential buildings and in commercial buildings and office buildings.

7.5.1 Designing the stiffening elements of the Example building

In the Example building, the lattice stiffening method is used. In the herein worked example, only the design of the transverse stiffening elements of the Example building is presented. The stiffening elements in the long side wall of the building are designed according to the same principles. In the end of the herein worked example, also the design of the lateral stiffening lattice in the roof structure is considered briefly.

Transverse stiffening:
The wind loads on the long side wall are resisted by the columns which transmit part of the loads directly to the foundation, and the rest is transmitted by the lateral lattice (parallel with the long side) to the stiffening lattice which is placed in the short side. The horizontal force in the wind columns on the long side wall is transmitted to the primary columns through the corrugated sheeting in the roof. This way, the eaves beam on the long side can be made lighter. Consequently, the joints of the corrugated sheeting, acting now as a stiffening element, must be checked against the lateral forces.

Due to initial imperfections (equivalent initial inclination) of the columns, the vertical loads generate also horizontal forces in the steel frame (a fictitious additional horizontal force, clause 7.1.6). Also this load is transferred in the roof plane by the lateral lattice (parallel with the long side) to the stiffening lattice which is placed in the short side wall.

Short side's stiffening lattice:
Design the stiffening elements of the short side wall. Assume a load combination where the wind load is taken as the governing load. In respect to the stiffening lattice, the critical situation is when the wind is parallel with the short side of the building. In such a case, the pressure load on the long side wall (zone D) and the suction load on the opposite side (zone E) both cause a load in the same direction to the stiffening lattice.

The horizontal load is divided evenly for the short side walls, as the building is symmetrical and also the stiffening elements on the short side walls are similar. Thus, using the formulae presented in clause 7.1.7 and 7.3.1, the normal force acting in the gable beam due to the horizontal force on the short side wall, is:

$$q_{wk} = c_s c_d \cdot c_e \cdot q_b \cdot c_{pe} = 1,0 \cdot 1,957 \cdot 0,33 \cdot c_{pe} = 0,65 c_{pe}$$

$$= 0,65 \cdot (0,7 + 0,3) = 0,65 \text{ kN/m}^2$$

$$q_{wd} = \gamma_{Q.1} \cdot q_{wk} = (1,5 \cdot 1,1) \cdot 0,65 = 1,07 \text{ kN/m}^2$$

$$F_{Ed} = q_{wd} \cdot \left(\frac{3}{8}H_1 + H_2\right)\frac{L}{2} = 1,07 \cdot \left(\frac{3}{8} \cdot 10,5 + 4\right) \cdot \frac{100}{2} = 424,7 \text{ kN}$$

where

- H_1 *is the height at eaves*
- H_2 *is the height of the building's pediment (see figure on page 417)*
- L *is the length of the building*

When the wind load is governing, the vertical load due to the self-weight and snow load is:

$$\gamma_G = 1,35 \cdot K_{FI} \qquad \textit{self-weight}$$

$$g_d = \gamma_G \cdot g_k = (1,35 \cdot 1,1) \cdot 0,5 = 0,743\ kN/m^2 \qquad \textit{self-weight}$$

$$s_d = \gamma_{Q.2} \cdot \psi_{0.2} \cdot s = (1,5 \cdot 1,1) \cdot 0,7 \cdot 1,2 = 1,39\ kN/m^2 \qquad \textit{snow load}$$

The vertical load generates a fictitious additional horizontal force to the columns in the primary frame, the calculation of which is presented in EN 1993-1-1 and in [22,23]. As the height of the columns in the primary frame is 10,3 m, the equivalent initial inclination of the columns is:

$$\alpha_m = \sqrt{0,5\left(1 + \frac{1}{m}\right)} = \sqrt{0,5\left(1 + \frac{1}{2}\right)} = 0,8660 \qquad \textit{(m is the number of primary columns)}$$

$$\alpha_h = \frac{2}{\sqrt{h}} = \frac{2}{\sqrt{10,3}} = 0,6232 \qquad \textit{but } \frac{2}{3} \le \alpha_h \le 1,0 \qquad \Rightarrow \alpha_h = 0,6667$$

$$\phi = \alpha_h \alpha_m / 200 = 0,6667 \cdot 0,8660 / 200 = \frac{1}{346} \qquad \textit{initial inclination of primary columns}$$

As in case of wind load, also the fictitious additional horizontal force is divided evenly for the short side walls, as the building is symmetrical and also the stiffening elements on the short side walls are similar:

$$V_{Ed} = (g_d + s_d) \cdot B \cdot \frac{L}{2} = (0,743 + 1,39) \cdot 48 \cdot \frac{100}{2} = 5119\ kN \qquad \textit{self-weight + snow}$$

$$H_{fic} = \phi \cdot \left[(g_d + s_d) \cdot B \cdot \frac{L}{2}\right] = \frac{1}{346} \cdot 5119 = 14,8\ kN$$

where

- B *is the width of the building*
- L *is the length of the building*

The additional horizontal force may be neglected, if the total horizontal load (including the fictitious additional horizontal force) is great enough in respect to the vertical loads:

$$H_{Ed} = F_{Ed} + H_{fic} = 424,7 + 14,8 = 439,5\ kN < 0,15\, V_{Ed} = 0,15 \cdot 5119 = 767,9\ kN$$

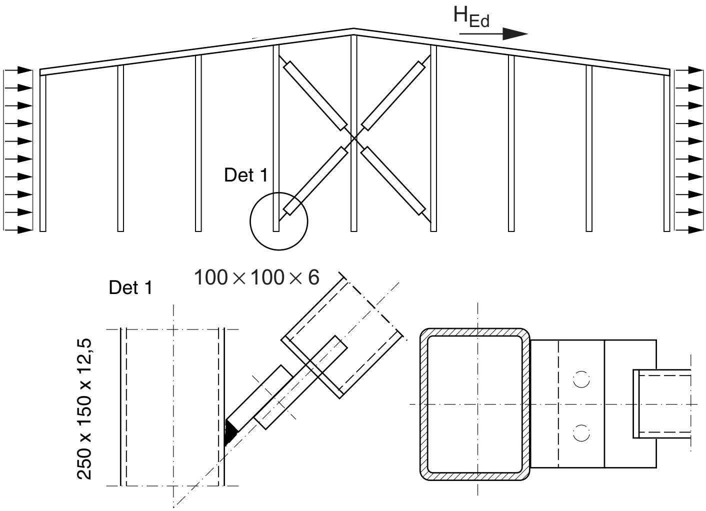
$\Rightarrow$ *the fictitious additional horizontal force must be considered*

The stiffening is executed with two diagonal bracings, one subject to compression and the other subject to tension. The tension bracing is designed to carry alone the whole horizontal load. The tensile force is as follows:

$$N_{Ed} = 439,5/\cos 48 = 656,8\ kN$$

Try a hollow section with dimensions 100×100×5. The steel grade is SSAB Domex Tube Double Grade, which fulfills the EN 10219 requirements for both steel grades S420MH and S355J2H. Thereby the design calculations may be performed at designer's own choice either according to grade S420 or grade S355. Grade S420 is chosen in this Example as design basis.

$$N_{t.Rd} = \frac{Af_y}{\gamma_{M0}} = \frac{1836 \cdot 420}{1,0} = 771,1\ kN \geq 656,8\ kN \quad OK$$



The bracing-to-column connection is made using a plate joint. The joint is a single lap joint made with plates (S355J2), each having the thickness of 20 mm and the width of 230 mm. The M30 bolts are of class 10.9 (2 pcs). Check the shear resistance of the bolts (it is assumed that the shear plane does not pass through the threaded portion of the bolts):

$$F_{v.Rd} = 0,6 f_{ub} A/\gamma_{M2} = 0,6 \cdot 1000 \cdot 707/1,25 = 339,4\ kN \geq \frac{656,8}{2} = 328,4\ kN \quad OK$$

Next, check the bearing resistance for the bolts when the end distance of the bolts is $e_1 = 60$ mm, the edge distance is $e_2 = 65$ mm and the hole diameter is $d_0 = 33$ mm (clause 3.4.1.2). In the direction parallel to the load both bolts act as end bolts, and in the direction perpen-

dicular to the load both bolts act as edge bolts. Thus, both bolts will have equal bearing resistance:

$$e_1 = 60\ mm \geq 1,2d_0 = 1,2 \cdot 33 = 39,6\ mm \quad OK$$

$$e_2 = 65\ mm \geq 1,2d_0 = 1,2 \cdot 33 = 39,6\ mm \quad OK$$

$$p_2 = 230 - 2 \cdot 65 = 100\ mm \geq 2,4d_0 = 2,4 \cdot 33 = 79,2\ mm \quad OK$$

$$\alpha_b = min\left[1,0\ ; \frac{f_{ub}}{f_{up}}\ ; \frac{e_1}{3d_0}\right] = min\left[1,0\ ; \frac{1000}{490}\ ; \frac{60}{3 \cdot 33}\right] = 0,606$$

$$k_1 = min\left[2,5\ ; \left(2,8\frac{e_2}{d_0} - 1,7\right); \left(1,4\frac{p_2}{d_0} - 1,7\right)\right] = min[2,5\ ; 3,82\ ; 2,54] = 2,5$$

$$F_{b.Rd} = k_1\alpha_b f_u dt/\gamma_{M2} = 2,5 \cdot 0,606 \cdot 490 \cdot 30 \cdot 20/1,25 = 356,3\ kN$$

However, in single lap joints with only one bolt row, the bearing resistance per bolt is limited to [18,19,20]:

$$F_{b.Rd} = 1,5f_u dt/\gamma_{M2} = 1,5 \cdot 490 \cdot 30 \cdot 20/1,25 = 352,8\ kN \geq \frac{656,8}{2} = 328,4\ kN \quad OK$$

The resistance of the plate-to-column joint is calculated with the formulae in Table 11.3.15. The face failure of the column (= the chord) is now not critical, because $\beta = 0,92 > 0,85$. However, the width of the plate is so close to the limit $b_0 - 2t_0$ given in the table for different failure modes, that also the following resistances shall be checked:

Chord face punching shear: $N_{1.Rd} = 736,6\ kN$
Chord side wall yielding: $N_{1.Rd} = 779,6\ kN$

The resistance of the joint is sufficient, since the horizontal component of the normal force of the diagonal bracing is:

$$N_{Ed} = 439,5\ kN \leq N_{1.Rd} \quad OK$$

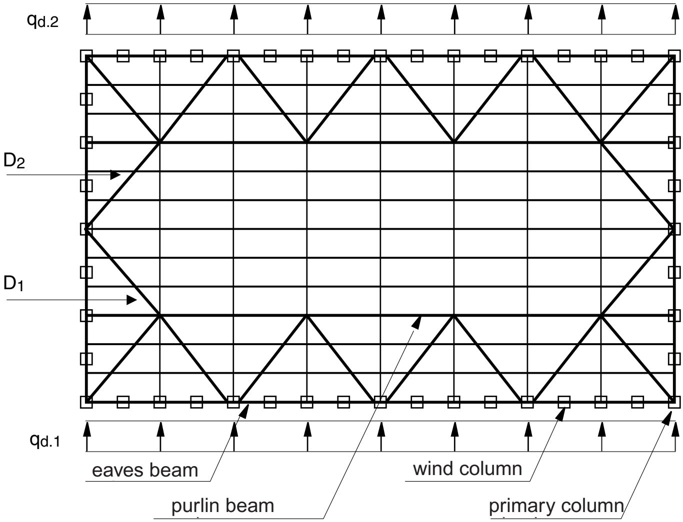
In addition, check the tension resistance of the plates in the gross cross-section as well as in the net cross-section due to the fastener holes:

$$N_{t.Rd} = \frac{Af_y}{\gamma_{M0}} = \frac{230 \cdot 20 \cdot 355}{1,0} = 1633\ kN \geq 656,8\ kN \quad OK$$

$$N_{t.Rd} = \frac{0,9A_{net}f_u}{\gamma_{M2}} = \frac{0,9 \cdot (230 - 2 \cdot 33) \cdot 20 \cdot 490}{1,25} = 1157\ kN \geq 656,8\ kN \quad OK$$

Long side's stiffening elements (diagonal bracing):
The stiffening elements in the long side wall shall be designed with the similar principle. In order to calculate the fictitious additional horizontal force due to the vertical loads, the equivalent initial inclination of the columns must first be determined. This is calculated using now value m = 9 for the m-factor (number of the columns to be restrained on the long side wall).

Lateral stiffening lattice in the roof structure:



The stiffening lattice is subjected to transverse wind load. The loads on the wind columns are transferred as support reactions, so part of the wind load is transferred directly to the foundation. The part of the wind load which is distributed for the roof, is transferred entirely to the stiffening lattice. When considering the wind load, it is important to distinguish between the effects of suction and pressure, since the compression resistance and tension resistance of a hollow section differ from each other. The following wind load is obtained on the long side wall:

$$q_{wk} = 0,65 c_{pe} = 0,65 c_{pe.D} = 0,65 \cdot 0,7 = 0,455\ kN/m^2 \quad \text{(pressure)}$$

$$q_{wd} = \gamma_{Q.1} \cdot q_{wk} = (1,5 \cdot 1,1) \cdot 0,455 = 0,751\ kN/m^2$$

$$q_{d1} = q_{wd} \cdot \left(\frac{3}{8}H_1 + H_2\right) = 0,751 \cdot \left(\frac{3}{8} \cdot 10,5 + 4\right) = 5,96\ kN/m$$

$$q_{wk} = 0,65 c_{pe} = 0,65 c_{pe.E} = 0,65 \cdot 0,3 = 0,195\ kN/m^2 \quad \text{(suction)}$$

$$q_{wd} = \gamma_{Q.1} \cdot q_{wk} = (1,5 \cdot 1,1) \cdot 0,195 = 0,322\ kN/m^2$$

$$q_{d2} = q_{wd} \cdot \left(\frac{3}{8}H_1 + H_2\right) = 0,322 \cdot \left(\frac{3}{8} \cdot 10,5 + 4\right) = 2,56\ kN/m$$

where

H_1 *is the height at eaves*

H_2 *is the height of the building's pediment (see figure on page 417)*

The normal forces in diagonal members D1 and D2 (see adjacent figure) are obtained from the support reaction of the wind lattice at the short side wall. Also the above calculated fictitious additional horizontal force due to the vertical loads shall be now taken into account. Due to the symmetry, this force can be divided evenly for the diagonal members D1 and D2:

$$D1: \quad N_{1.Ed} = \frac{q_{d1} \cdot \frac{L}{2} + \frac{H_{fic}}{2}}{\cos 40} = \frac{5,96 \cdot \frac{100}{2} + \frac{14,8}{2}}{\cos 40} = 398,7\ kN \quad (tension)$$

$$D2: \quad N_{2.Ed} = \frac{q_{d2} \cdot \frac{L}{2} + \frac{H_{fic}}{2}}{\cos 40} = \frac{2,56 \cdot \frac{100}{2} + \frac{14,8}{2}}{\cos 40} = 176,8\ kN \quad (compression)$$

The diagonal is restrained at purlin trusses, so the buckling length is (as a conservative simplification, the buckling length factor is taken here as L_{cr} = 1,0L):

$$L_{cr.y} = L_{cr.z} = \frac{\sqrt{20^2 + 24^2}}{6} = 5,2\ m$$

Try a hollow section with dimensions 120×120×4. The tension resistance and the buckling resistance are as follows:

$$N_{t.Rd} = \frac{A f_y}{\gamma_{M0}} = \frac{1815 \cdot 420}{1,0} = 762,3\ kN \geq 398,7\ kN \qquad OK$$

$$N_{b.Rd} = \frac{\chi \cdot A \cdot f_y}{\gamma_{M1}} = \frac{0,2922 \cdot 1815 \cdot 420}{1,0} = 222,7\ kN \geq 176,8\ kN \qquad OK$$

To complete, also the purlin trusses and the eaves beam must be designed.

Comparison S420 vs S355:

In case the design calculations would be performed according to grade S355, the calculations would follow basically the same principles. The increase of the material strength S355 → S420 improves the buckling resistance in this Example by ca. 4 % and the tension resistance by the full increase of yield strength (= + 18 %).

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8. WORKSHOP FABRICATION AND ON-SITE ERECTION

8.1 General

Design of a structure includes, in addition of structural calculations, also tasks regarding workshop fabrication and erection on site. Designing aims at minimizing the weight of the structure, since the price of a hollow section is almost directly proportional to its weight. However, the lightest structure is not necessarily the most economical solution in regard to the ensemble of the construction project. An extremely optimized structure may be expensive to fabricate and erect, which may result in losing the savings gained in material costs to increased manufacturing and erection costs. To reach an optimal result, it is important that the designer, the workshop and the site all work in close cooperation and that all parties have sufficient knowledge about on all aspects of the whole construction project.

The general principle is to perform the most demanding and complex operations at the workshop to enable quick and cost-efficient on-site erection. In practice this means that all welded joints are made at the workshop, and on-site erection then consists of connecting the erection blocks to each other using bolted joints. Special consideration should be taken to minimize the needed working times for the cranes (especially if mobile cranes have to be used).

8.1.1 General requirements to the manufacturer

According to EN 1090-1, the manufacturer is obliged to establish and maintain Factory Production Control (FPC) covering the workshop fabrication. The fabrication shall comply with various standards, most of which are found as the reference standards listed in EN 1090-1 and EN 1090-2.

8.1.2 Execution specification

The steel structure designer is obliged to present the requirements concerning fabrication and erection in the execution specification that is based on the standard EN 1090-2.

8.1.3 Execution classes

EN 1090-2 presents four execution classes EXC1 - EXC4, for which the requirement strictness increases from class 1 to class 4. The execution class shall be determined latest in the design phase. However, it would be good to have a general idea about the execution class already in the tender phase, because execution class related requirements have a cost effect that also affects the price setting of the project.

Annex B of EN 1090-2, i.e. determination of execution class, will be moved over in its totality to Eurocode to become an annex of EN 1993-1-1. When this happens, the following instructions for the determination of execution class (EXC) that are based on the guidance presented in EN 1090-2, may be subject to changes.

The execution class may apply to the whole structure or to a part of the structure or to a detail. A structure can include several execution classes, whereas a single detail has normally only one execution class. If no execution class has been specified, execution class EXC2 shall apply. The requirements related to execution classes are presented in the normative Annex A of EN 1090-2.

The determination of execution class is governed by the consequences classes CC1-CC3, service categories SC1-SC2 and production categories PC1-PC2. The consequences classes are presented in Annex B of EN 1990 and in Chapter 2 of this handbook. Welded steel structures belong to production category PC2 when steels of grade S355 or higher grade are used. Service categories, production categories, determination of execution class and the requirements in different execution classes are presented in Annex B of EN 1090-2 and in Tables 8.1 - 8.4 of this handbook.

The execution class has an impact for example on the required tolerances, weld quality levels, inspection methods and scope of inspection. The requirements of execution classes should be cleared up latest before beginning of the design work.

Table 8.1 Determination of service category according to Annex B of EN 1090-2 [1]

Category	Criteria
SC1	• Structures and components designed for quasi static actions only (for example buildings) • Structures and components with their connections designed for seismic actions in regions with low seismic activity and in DCL a) • Structures and components designed for fatigue actions from cranes (class S_0) b)
SC2	• Structures and components designed for fatigue actions according to EN 1993. (Examples: road and railway bridges, cranes (class S_1 to S_9) b), structures susceptible to vibrations induced by wind, crowd or rotating machinery) .• Structures and components with their connections designed for seismic actions in regions with medium or high seismic activity and in classes DCM a) and DCH a)
a) DCL, DCM, DCH; ductility classes according to EN 1998-1 b) For classification of fatigue actions from cranes, see EN 1991-3 and EN 13001-1.	

Table 8.2 Determination of production category according to Annex B of EN 1090-2 [1]

Category	Criteria
PC1	• Non-welded components manufactured from any steel grade procucts • Welded components manufactured from steel grade products below S355
PC2	• Welded components manufactured from steel grade products from S355 and above • Components essential for structural integrity that are assembled by welding on construction site • Components with hot forming manufacturing or receiving thermic treatment during manufacturing • Components of CHS lattice girders requiring end profile cuts

Table 8.3 Determination of execution class according to Annex B of EN 1090-2 [1]

Consequences class		CC1		CC2		CC3	
Service category		**SC1**	**SC2**	**SC1**	**SC2**	**SC1**	**SC2**
Production category	**PC1**	EXC1	EXC2	EXC2	EXC3	EXC3 a)	EXC3 a)
	PC2	EXC2	EXC2	EXC2	EXC3	EXC3 a)	EXC4

a) EXC4 should be applied to special structures or structures with extreme consequences of a structural failure as required by national provisions.

Table 8.4 Requirements to each execution class according to EN 1090-2 [1]

Clauses	**EXC1**	**EXC2**	**EXC3**	**EXC4**
Specifications and documentation				
Quality documentation	No requirements	Required	Required	Required
Constituent products; identification, inspection documents and traceability				
Inspection documents	see Table 1.8			
Traceability	No requirements	Required (partial)	Required (full)	Required (full)
Marking	No requirements	Required	Required	Required
Structural steel products				
Thickness tolerances	Class A	Class A	Class A	Class B
Surface conditions	Flats: Class A2 Long products: Class C1	Flats: Class A2 Long products: Class C1	More stringent conditions if specified	
Special properties	No requirements	No requirements	Internal discontinuity quality class S1 for welded cruciform joints (Avoiding lamellar tearing).	
Preparation and assembly				
Identification	No requirements	No requirements	Finished components / Inspection certificates	
Thermal cutting	Free from significant irregularities Hardness according to Table 8.5, if specified	EN ISO 9013 u=range 4 Rz5=range 4 Hardness according to Table 8.5, if specified	EN ISO 9013 u=range 4 Rz5=range 4 Hardness according to Table 8.5, if specified	EN ISO 9013 u=range 3 Rz5=range 3 Hardness according to Table 8.5, if specified
Flame straightening	No requirements	No requirements	Suitable procedure to be developed	
Holing	Punching	Punching	Material thickness t ≤ 3 mm: punching to full size permitted Material thickness t > 3 mm: punching + reaming (punching to 2 mm undersize) Thermally cut holes; hardness of cut edges according to Table 8.5.	

(continues)

Table 8.4 Requirements to each execution class according to EN 1090-2 [1] *(continued)*

Cut-outs	No requirements	Min. radius 5 mm		Min. radius 10 mm, punching not permitted
Assembly	Drifting: Elongation Functional tolerance Class 1		Drifting: Elongation Functional tolerance Class 2	
Welding				
General	EN ISO 3834-4	EN ISO 3834-3	EN ISO 3834-2	EN ISO 3834-2
Qualification of welding procedures	No requirements	see EN 1090-2 Tables 12 and 13		
Qualification of welders and operators	Welders: EN 287-1. Operators: EN 1418			
Welding coordinating	No requirements	Technical knowledge: see EN 1090-2 Tables 14 and 15		
Joint preparation	No requirements	No requirements	Prefabrication primers not allowed	
Temporary attachments	No requirements	No requirements	Use to be specified Cutting and chipping not permitted	
Tack welds	No requirements	Qualified welding procedure		
Butt welds	No requirements	Run on / run off pieces if specified	Run on / run off pieces Permanent backing continuous in single sided welds	
Execution of welding			Removal of spatter	
Acceptance criteria	EN ISO 5817 Quality level D	EN ISO 5817 Quality level C generally	EN ISO 5817 Quality level B	EN ISO 5817 Quality level B+ [a)]
Erection and work at site				
Handling and storage on site	No requirements	Documented restoration procedure		
Fit up and alignment	No requirements	No requirements	Securing shims by welding subject to requirements above in item **Welding**	
Inspection, testing and repair				
Scope of inspection	Visual inspection	NDT according to Table 8.13		
Correction of welds	No WPQ required	According to WPQ		
Production tests	No requirements		If specified	
Inspection and testing of preloaded bolts connections	No requirements	Inspection method: see EN 1090-2 clause 12.5.2		

(continues)

Table 8.4 Requirements to each execution class according to EN 1090-2 [1] *(continued)*

Inspection, testing and repair of hot rivets	No requirements	Ring test: Lightly knock the rivet head using a 0,5 kg hammer Inspection sequence A (EN 1090-2 Annex M)		Ring test: Lightly knock the rivet head using a 0,5 kg hammer Inspection sequence B (EN 1090-2 Annex M)
Survey of the geometrical position of connection nodes	No requirements		Record of the survey	
a) EN ISO 5817 Quality level B + additional requirements EN 1090-2 Table 17.				

8.1.4 Traceability of parts and components

To identify the parts and components, every piece or package including similar parts should be sufficiently well marked to enable the identification. Moreover, in execution classes EXC3 and EXC 4 there shall be traceability to the inspection certificates.

If the applied steel is of higher grade than S355, hard stamping, punching, drilling or other corresponding material machining or removing marking method is not allowed [1].

8.2 Cutting of structural hollow sections

Structural hollow sections can be ordered either in standard lengths (6, 12 or 18 m) or cut to size (up to 24 m). No waste pieces are produced when ordering cut-to-size sections. The cutting is usually done at the workshop, if the structure requires mitre-cuts (for example lattices). Cutting of square and rectangular hollow sections is simple, because cutting can be done along one plane. However, multiple cutting planes are needed, for example, in overlap joints. Circular hollow sections are more complex to cut particularly for joints where circular sections need to be joined to each others. In such a case, the end of the circular hollow section often needs a ‘profiled’ cut.

8.2.1 Cutting of circular hollow sections

At its simplest, a circular hollow section can be joined onto another circular hollow section by cutting the hollow section as a straight or mitre cut along one plane. This presumes, however, that the external diameters of the hollow sections to be joined differ from each other distinctly, so that the root gap remaining at the edges of the smaller hollow section (Figure 8.1) is small enough for welding. Requirements regarding the root gap dimensions and the fitting between the hollow sections are presented in clause 8.5.4.2.

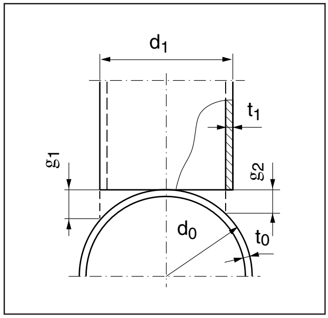


Figure 8.1 Root gaps at the edges of the smaller hollow section, when the hollow section is cut along one plane surface only

In practice, most structures made of circular hollow sections do not fulfill the fit requirements of clause 8.5.4.2, if cutting is made as one-plane-cut only. The best fit is achieved by cutting the end of the smaller hollow section to the required shape by using thermal cutting methods and applying a template drawing, or by utilising more modern methods such as 3D laser cutting (see clause 8.2.2).

In circular hollow section joints, it is also possible to reduce the size of the root gap and improve the fit with an approximate method being 'between' the above methods by cutting the hollow section in several planes according to Figure 8.2. The geometry of the joint is then determined by the cutting angles α_g and α_d, for which the following equations are obtained [2,3]:

$$\alpha_g = 90° - \theta + arc\,tan\left(\frac{h \cdot sin\theta}{r_1 + h \cdot cos\theta - L \cdot sin\theta}\right) \tag{8.1}$$

$$\alpha_d = -90° + \theta + arc\,tan\left(\frac{h \cdot sin\theta}{r_1 - h \cdot cos\theta - L \cdot sin\theta}\right) \tag{8.2}$$

where $L = \sqrt{r_1^2 - (r_1 - t_1)^2}$

$$h = \frac{d_0}{2} - \sqrt{\frac{d_0^2}{4} - (r_1 - t_1)^2}$$

$$r_1 = \frac{d_1 - 2t_1}{2}$$

d_0 is the diameter of the greater hollow section

d_1 is the diameter of the smaller hollow section

t_1 is the wall thickness of the smaller hollow section

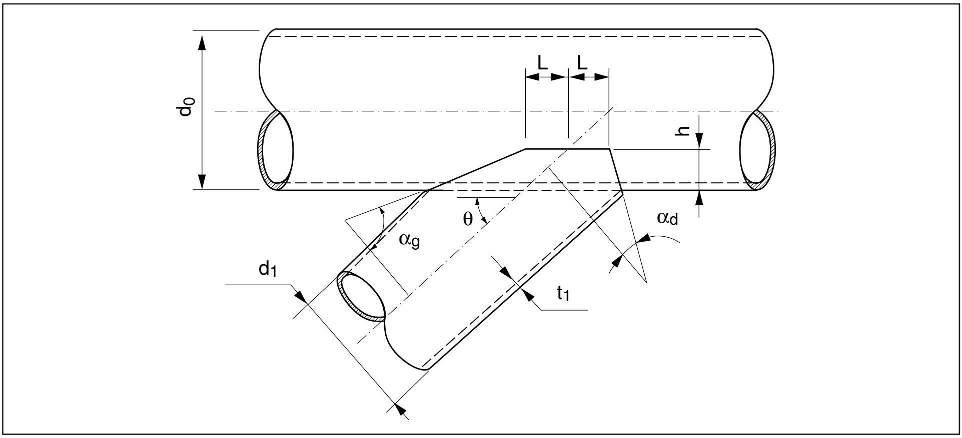


Figure 8.2 Cutting of a circular hollow section along three planes

8.2.2 Cutting methods

Sawing

Sawing is the most commonly used method for cutting hollow sections. Usually, a disc saw or a band saw is used. An increase in the sawing speed usually decreases the accuracy in cutting and generates burrs which then need to be removed. In addition to the sawing speed, the easy transport of hollow sections to / from the sawing site is a factor worth considering. It is also possible to cut both ends of a hollow section simultaneously which saves time. Saw blades must be changed often enough, as the decreased sharpness of the blade increases the dimensional deviation and the quality of the cut surface deteriorates.

Other cutting methods

a) Thermal cutting

Thermal cutting, when made free-hand, is a less accurate cutting method than sawing. Thermal cutting is suitable, for instance, for shaping the ends of circular hollow sections in lattice joints.

b) Cutting by punching

Cutting by punching is feasible only with thin-walled hollow sections. A benefit of this method is to achieve also complex cutting surfaces. In circular hollow section joints, the ends can be shaped in one go with punching.

c) Laser cutting

Laser cutting is an accurate method. A further advantage is the small heat affected region in the vicinity of the cut. A disadvantage of laser cutting is the expensive equipment needed.

With 3D lasers available nowadays, the end of a hollow section can be shaped directly to fit the joint, which reduces the need for additional costly handwork. This is a benefit especially with the joints of circular hollow sections.

For example, in the SSAB service centres there are 3D lasers, which make it possible to cut material thicknesses up to ca.16 mm, with the maximum external dimensions being on circular hollow sections 508 mm, on square hollow sections 400×400 mm and on rectangular ones 300×500 mm. For the data transfer the IGES and STEP formats are applied, which are available in all commonly used 3D CAD software.

The maximum values given in EN 1090-2 for the hardness of the cut edge are presented in Table 8.5, which is valid if the bolt holes are made with thermal cutting.

In other cases the table is valid, if so required in the execution specification.

The cut edges shall be inspected and repaired, if there are significant defects in the cut surface. If the cut edge is machined or ground, the removed material depth shall be at least 0,5 mm.

Table 8.5 Maximum hardness of thermally cut edge according to EN 1090-2 [1]

Product standards	Steel grades	Hardness values (HV10)
EN 10025-2...5	S235 - S460	380
EN 10210-1, EN 10219-1		
EN 10149-2, EN 10149-3	S260 - S700	450
EN 10025-6	S460 - S690	
NOTE: These values are in accordance with EN ISO 15614-1 applied to steel grades listed in ISO/TR 20172.		

8.2.3 Notching of hollow section ends

The use of splice plates in bolted joints may require notches to be done in the hollow section wall (Figure 8.3a). In lightly loaded joints, however, it is more profitable to cut the splice plate rather than the hollow section (Figure 8.3b). To transfer the forces in heavily loaded joints it is, conversely, better to cut the hollow section and keep the splice plate intact, since it is difficult to make a sufficiently long weld inside the hollow section. Also in joints using small size hollow sections, is necessary to cut the hollow section, since internal welding is practically impossible.

Notches can be done using the same methods as cutting.

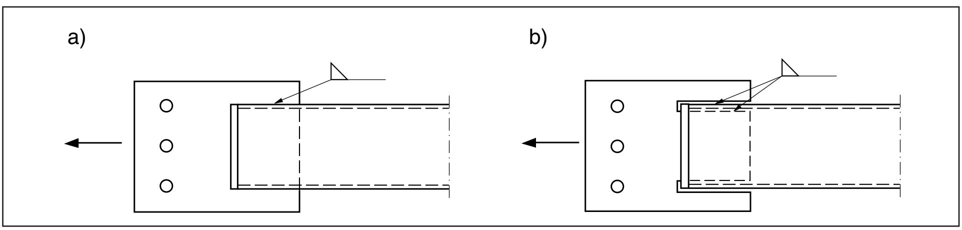


Figure 8.3 Joining splice plate to hollow section

8.3 Bending of structural hollow sections

A structural hollow section can be bent either by cold or hot bending.

Hot bending

Different steel products and steel grades react differently to heating. Incorrect heating can deteriorate the properties of steel and decrease the strength of steel even considerably. That is why EN 1090-2 presents steel type and steel grade dependent instructions, limitations and prohibitions regarding hot forming and the temperature ranges to be applied. If hot bending of structural hollow sections is needed, the technical customer service of structural hollow sections should be consulted.

Cold bending

Cold bending is a more cost-efficient and simpler method, and thus more commonly used. Circular hollow sections are easier to bend than square and rectangular hollow sections, since in the latter case the shape of the cross-section tends to get distorted.

Factors affecting the success of bending are:

- ultimate strength and yield strength of the material
- chemical composition and the elongation at fracture of the material
- ratio of wall thickness to the height and width of the hollow section (t/h, t/b, t/d)
- ratio of the bending radius to the height and width of the hollow section (r/h, r/d, r/b)

When using square and rectangular hollow sections, the effect of distortion of the cross-section shape to the visual outlook of the structure should be evaluated on a case by case basis. According to EN 1090-2 cold bending is allowed providing that the hardness and geometry of the bent hollow sections are checked [1]. The more critical the visual appearance of the structure, the greater the bending radius must be. Hollow sections with lower ultimate strength and yield strength are easier to bend. Also a greater wall thickness in relation to the height and width of the hollow section improves the bendability. A small bending radius is always more difficult to execute than a great bending radius.

The cross-sectional properties of square and rectangular structural hollow sections decrease in bending. Reduced values for the second moment of area are presented in Annex 11.5. If subject to compression, the distortion of the hollow section's wall during bending must also be taken into account when determining the compression resistance of the hollow section.

According to EN 1090-2, cold bending of circular hollow sections is allowed provided that the following conditions are met, if not otherwise presented [1]:

- the ratio of the external diameter of the hollow section to the wall thickness shall be $d/t \leq 15$
- the bend radius at the centre line of the hollow section shall not be less than the value of max [1,5d ; d+100 mm] where d is the external diameter of the hollow section
- the longitudinal seam weld in the cross-section shall be positioned close to the neutral axis, in order to reduce the bending stresses at the weld.

The most demanding bendings usually require practical expertise, i.e. a workshop that is specialized in bending and is equipped with appropriate machinery. For successful bending, the cooperation and expertise of the designer and the manufacturer is important.

8.3.1 Bending methods for structural hollow sections

Roller bending

In roller bending, the hollow section passes through three or four shaped rollers. The size of the rollers is determined by the size of the hollow sections. The middle rollers determine the magnitude of the bending radius. Normally, one of the rollers is freely rotating. Minimum bending radii for square and rectangular hollow sections are presented in Annex 11.5.

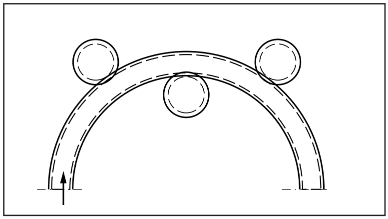


Figure 8.4 Bending of a hollow section as a 3-roller bending

Induction bending

In induction bending, the hollow section is heated during bending with an inductance coil. At a time, only a small portion of the hollow section is heated and simultaneously bent. This is repeated until the entire hollow section is processed. Compared to roller bending, induction bending is a more expensive method, but the minimum bending radii are smaller.

Curved lattice structures can be made from straight elements. The curved shape is produced by joining the straight chord member elements together in an angle corresponding to the bending radius (Figure 8.5).

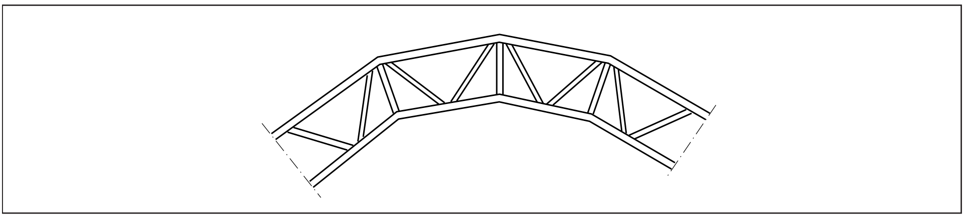


Figure 8.5 Hollow section structure in a curved shape

8.4 Bolted joints

In bolted joints of hollow sections, usually the holes are placed in the component partnering the hollow section, i.e. usually in a plate. The simplest way would be to drill the holes directly in the hollow section, but in practice, the slender walls of hollow sections cannot bear much load. However, by using special drilling methods or special bolts, the hollow section joints can be made directly via the hollow section wall. Those methods can be utilised in joints subjected to lighter loads. They are presented shortly later on in clause 8.4.3.

8.4.1 Bolts, nuts and washers to be used

The nominal diameter of the bolt shall be at least 12 mm, if not otherwise specified. For thin gauge components and sheeting the minimum diameter shall be specified for each type of fastener. The bolt length shall be chosen such that after tightening at least one thread pitch comes outside the nut (preloaded as well as non-preloaded bolt assemblies). Also, for non-preloaded bolts, it shall be checked that at least one full thread (in addition to the thread run-out) shall remain clear between the bearing surface of the nut and the unthreaded portion of the bolt shank. For preloaded bolts, at least four full threads (in addition to the thread run-out) shall remain clear between the bearing surface of the nut and the unthreaded portion of the bolt shank [1].

The nut shall run freely on its partnering bolt. Any nut and bolt assembly where the nut does not run freely shall be discarded. The nuts shall be assembled so that their designation markings are visible for the inspection after assembly.

According to EN 1090-2, generally washers are not required for non-preloaded joints, when the bolts are assembled in normal round holes. However, if washers are required, it shall be specified whether washers are to be placed only under the nut or the bolt head, whichever is rotated (usually the nut), or under both of them. For single lap joints with only one bolt row, washers are required both under the bolt head and the nut.

For preloaded joints when using class 8.8 bolts, plain washers (or if necessary taper washers being hardened to corresponding 300 HV) shall be used under the part to be rotated (nut or bolt head). For preloaded joints when using class 10.9 bolts, washers shall be used both under the nut and the bolt head [1].

Washers used under heads of preloaded bolts shall be chamfered washers conforming to EN 14399-6 and positioned with the chamfer towards the bolt head. Washers conforming to EN 14399-5 (i.e. without chamfer) are permitted only under the nut. To adjust the grip length of bolt assemblies, one additional plate washer or at maximum three additional washers with a maximum combined thickness of 12 mm, may be used. When torque control method or system HRC is used as the tightening method, only one additional washer may be placed under the part to be rotated, alternatively an additional washer (or additional washers) may be placed under the part not to be rotated. In other case, in preloaded as well as non-preloaded bolt assemblies, an additional washer (or additional washers) may be placed either under the part to be rotated or under the part not to be rotated [1].

The nominal clearances for bolt holes have been presented in Table 8.6 according to EN 1090-2. For fit bolts the nominal hole diameter shall be equal to the nominal diameter of the bolt shank.

The holes may be made by drilling or punching. However, the holes for joints in fatigue loaded structures shall always be made by drilling (though also then punching can be utilised by first punching a prehole that is at least 2 mm undersize in diameter, which is then reamed to its final diameter).

Table 8.6 Nominal clearances for bolt holes (mm) [1]

<table>
<tr><td>Bolt size</td><td>M12</td><td>M16</td><td>M20</td><td>M22</td><td>M24</td><td>M27
and over</td></tr>
<tr><td>Normal round holes a)</td><td>1 b) c)</td><td colspan="4">2</td><td>3</td></tr>
<tr><td>Oversize round holes</td><td>3</td><td colspan="3">4</td><td>6</td><td>8</td></tr>
<tr><td>Short slotted holes (on the length) d)</td><td>4</td><td colspan="3">6</td><td>8</td><td>10</td></tr>
<tr><td>Long slotted holes (on the length) d)</td><td colspan="6">1,5d</td></tr>
<tr><td colspan="7">a) For fit bolts the nominal hole diameter shall be equal to the shank diameter of the bolt (for fit bolts to EN 14399-8 the nominal diameter of the shank is 1 mm larger than the nominal diameter of the threaded portion).

b) For coated fasteners, 1 mm nominal clearance can be increased by the coating thickness of the fastener.

c) M12 and M14 bolts or countersunk bolts may also be used in 2 mm clearance holes provided that the design resistance of a bolt or bolt group basing on bearing is less or equal to the value basing on the shear resistance of the bolt or bolt group. In addition, for class 4.8, 5.8, 6.8, 8.8 and 10.9 bolts the design value of shear resistance $F_{v.Rd}$ should be reduced as presented in Part EN 1993-1-8 of Eurocode.

d) For bolts in slotted holes the nominal clearances across the width shall be the same as the clearances on diameter specified for normal round holes.</td></tr>
<tr><td colspan="7">Footnote c) in this table is corrected according to [EN 1993-1-8:AC:2009].</td></tr>
</table>

8.4.2 Installation of bolted joints

8.4.2.1 Installation of non-preloaded bolts

With the non-preloaded bolts it is recommendable to use a washer on that side of the bolt (normally under the nut), where it will be tightened. Otherwise the coating of the component to be joined may be damaged.

Separate plate components forming part of a common ply shall not differ in thickness by more than 2 mm generally and 1 mm when using preloaded bolts. Packing plates may be used for adjustment, but their thickness shall be at least 2 mm. The thickness of the packing plates shall be chosen such that the number of them is not more than three. The connected components shall be drawn together such that they achieve firm contact. When fastening plates of material thickness $t \geq 4$ mm or sections of material thickness $t \geq 8$ mm, unless full contact bearing is specified, residual gaps of maximum 4 mm may be left at the edges provided that contact bearing is achieved at the central part of the joint [1].

The bolt is tightened using a suitable tool such that there shall be a snug-tight contact between the joint surfaces. Generally, the required so-called ‘snug-tight contact’ is deemed to be achieved by the effort of one man using a normal sized spanner without an extension arm, or when a percussion wrench starts hammering.

Normally the bolt is tightened by turning the nut. The bolts in a bolt group are tightened bolt by bolt, starting from the most rigid part of the joint and moving progressively towards the least rigid part. To achieve a uniform tightness, more than one cycle of tightening may be necessary. Large bolt groups are tightened starting from the middle and progressing towards the edges alternately on both sides of the middle part.

EN 1090-2 does not require locking of the nuts (except for bolted joints with small clamp lengths in thin gauge components subject to significant vibrations, such as storage racks), but the necessity of locking is left up to the designer. It shall be explicitly specified if, in addition to tightening, other measures or means are to be used to secure the nuts. This shall be then presented in the execution specification. For locking the nuts, welding is not permitted, unless otherwise specified [1].

8.4.2.2 Installation of preloaded bolts

For preloaded bolts, washers are used such, that with class 8.8 bolts a washer shall be used under the part to be rotated (normally the nut), and with class 10.9 bolts a washer shall be used both under the nut and the bolt head [1].

For preloaded bolts, no other measures or means are to be used to secure the nuts, unless otherwise specified.

For preloaded bolts, the minimum value of preloading force $F_{p.C}$ is presented in Table 8.7. This level of preload shall be used for all slip resistant preloaded joints and for all other preloaded joints, unless a lower level of preload is specified. In the latter case, the bolt assemblies, the tightening method, the tightening parameters and the inspection requirements shall also be specified.

The tightening is started from the most rigid part of the joint and moving progressively towards the least rigid part. To achieve a uniform tightness, more than one cycle of tightening may be necessary.

For tightening the preloaded bolts, there are four different methods given in EN 1090-2 [1]:

- torque method
- combined method
- HRC tightening method and
- direct tension indicator (DTI) method.

Direct tension indicator method is based on washers indicating the preload force as presented in EN 14399-9.

HRC method is based on special HRC bolts conforming to EN 14399-10. Also a special wrench is needed. The bolt assembly installation is complete when the spline end of the bolt shank shears off from the shank at the break-neck section. The specified preload requirement is controllod in the HRC method automatically by means of the bolt's geometrical and torsion mechanical characteristics together with the lubrication conditions. It does not need calibration.

In the torque method, a calibrated torque wrench shall be used, the accuracy of which shall be ± 4 % according to EN ISO 6789. The accuracy of the torque wrench shall be checked at least once a week. For torque wrenches used in the first step of the combined method, it is sufficient that the accuracy is ± 10 % and it is checked once a year. Checking shall always be renewed after any non-intended incident occurring during the use (significant impact, fall, overloading etc). Tightening shall be performed in two steps as follows:

a) first step:
the bolts are tightened to the torque which is 75 % of the torque that is calculated from the formula:

$$M_{r.2} = k_m \cdot d \cdot F_{p.C} \tag{8.3}$$

where k_m is according to class K2 the mean value of k-factor (declared by the fastener manufacturer)
d is the nominal diameter of the bolt
$F_{p.C}$ is the minimum value of the bolt's preloading force,
$F_{p.C} = 0{,}7 f_{ub} A_s$ (Table 8.7)

The first step shall be completed for all bolts of the joint prior to commencement of the second step.

b) second step:
the bolts are tightened to the torque, the value of which is $1{,}1 \times M_{r.2}$.

In the combined method the bolts shall be tightened in two steps as follows:

a) first step:
the bolts are tightened to 75 % torque as in the torque method.

b) second step:
the turn specified in Table 8.8 is applied to the turned part of the assembly, thus reaching the final preloading force. The position of the nut (if the part to be turned is the nut) relative to the bolt threads shall be marked after the first step, using a marking crayon or marking paint, so that the final rotation of the nut relative to the thread in this second step can be easily determined.

Table 8.7 Minimum values of the preloading force of the bolts $F_{p.C}$ (kN) [1]

Bolt class	Bolt size							
	M12	M16	M20	M22	M24	M27	M30	M36
8.8	47	88	137	170	198	257	314	458
10.9	59	110	172	212	247	321	393	572

Table 8.8 Combined method, additional rotation for the second step (8.8 and 10.9 bolts) [1]

Total nominal thickness t of parts to be connected (including all packs and washers) d=bolt diameter	Further rotation to be applied during the second step of tightening	
	Degrees	Part turn
t < 2d	60°	1/6
2d ≤ t < 6d	90°	1/4
6d ≤ t ≤ 10d	120°	1/3
Where the surface under the bolt head or nut is not perpendicular to the bolt axis (allowing for taper washers, if used), the required angle of rotation should be determined by testing.		

8.4.3 Use of special fasteners and methods

8.4.3.1 Self-tapping screws and self-drilling screws

In joints subjected to light loads only, it is possible to use self-tapping or self-drilling screws. The most commonly used screw sizes are 5,5 and 6,3 mm. In self-drilling joints, the screw shall always be fastened by working on the thinner part side. For self-tapping screws, the prehole shall be somewhat smaller than the nominal diameter of the screw (the value is declared by the screw manufacturer). Self-drilling screws are easier and quicker to use, since they do not require a prehole to be drilled. When using self-tapping or self-drilling screws, the instructions by the screw manufacturer shall be followed.

8.4.3.2 Friction drilling of the hollow section wall

In friction drilling, a special hard alloy tool is used, which heats up the wall of the hollow section due to friction. The heated material extrudes first to the outside of the tube surface, and after the penetration to the inside. Thus the base material itself forms a bushing, which can be threaded. The threads are made in a separate workphase, either with a traditional thread cutting tool or with a thread rolling die. Due to the bushing, the threaded length is almost double compared to the original wall thickness. The method has been tested up to the screw size M24 and material thickness 12,5 mm [3]. However, in Finland, the method has no approval for structural joints.

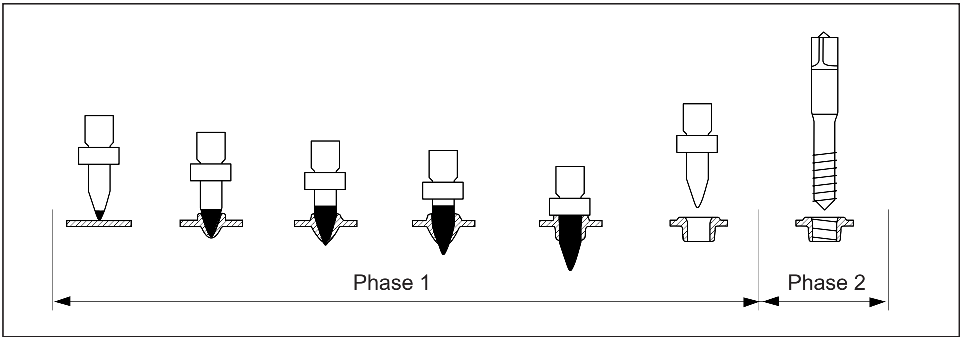


Figure 8.6 Stages of the process in Flowdrill friction drilling

8.4.3.3 Joints using expansion bolts

In expansion bolt joints, the hole to the hollow section wall is drilled using conventional methods. The hole is then equipped with an expansion bolt, which functions in the same way as those used in the joints of concrete structures. Inside the expansion bolt there is a threaded hole in which the bolt is placed. When tightening the bolt, the conical head of the expansion bolt is pressed against the hollow section wall. Simultaneously, the cone spreads the sleeve of the expansion bolt, which generates the necessary tightening torque in the bolt. The method is applicable for bolts M8 - M16 and for all wall thicknesses [3,4]. In Finland, the method has no approval for structural joints.

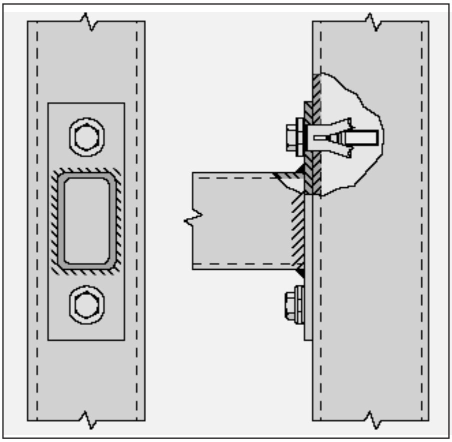


Figure 8.7 Hollo-Bolt expansion bolt joint

8.4.3.4 Joints using blind rivets

In addition to conventional small-size blind rivets ('pop rivets') there are on the market also more special solutions for more robust applications, such as the Huck Ultra-Twist blind bolt presented in Figure 8.8. The diameter sizes of the fastener shank are d = 20 / 24 / 27 mm. The hole needed for the fastener in the wall of the hollow section is drilled using conventional methods. In Finland, the method has no approval for structural joints.

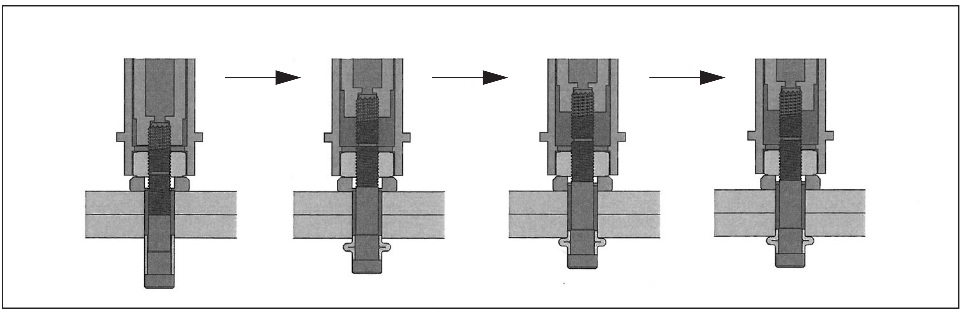
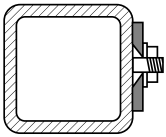


Figure 8.8 Stages of operation with Huck Ultra-Twist fastener

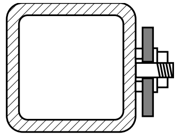
8.4.3.5 Joints using threaded studs

Joints subjected to light loads only can also be made using threaded studs which are welded to the hollow section wall. The hole in the component to be joined shall be enlarged by reaming from the hollow section side, so that the weld shall not bear against it after the bolts are tightened. Another alternative is the use of washers between the hollow section and the component to be joined. Special attention should be paid to protection of the studs during transportation and installation.

With fatigue loaded structures, it must be kept in mind that welding of any outfittings often decreases the fatigue strength, why they are not allowed unless permitted by the designer.



a) The space needed for the weld of the stud is gained by chamfering the splice plate



b) The weld of the stud is allowed by using a fit-for-purpose washer

Figure 8.9 Joints using a threaded stud

8.5 Welding of structural hollow sections

The structural steels used in SSAB Domex Tube structural hollow sections have good weldability with all common welding methods. In normal workshop conditions, elevated working temperature is not required when welding material thicknesses applied in hollow sections.

Weldability depends on the welding procedure and the chemical composition of the material. The weldability of steel is usually evaluated by the value of carbon equivalent (CEV) calculated on the basis of chemical composition of steel. The smaller the carbon equivalent, the better the steel is to weld. The most commonly used formula to calculate the carbon equivalent is the formula by IIW (International Institute of Welding):

$$CEV = C + \frac{Mn}{6} + \frac{Cr + Mo + V}{5} + \frac{Ni + Cu}{15} \qquad (8.4)$$

As regard to cold cracking (hydrogen cracking), the steel is easily weldable by any common process, when the CEV value is less than 0,41. With normal wall thicknesses of longitudinally welded structural hollow sections by SSAB (≤ 12,5 mm), made of SSAB's standard steel grade SSAB Domex Tube Double Grade, no special actions are needed for welding (CEV ≤ 0,39). Preheating is necessary, when the carbon equivalent is higher than 0,45. The components to be welded must be dry and free from grease and oil.

The cold-formed SSAB Domex Tube structural hollow sections have good weldability also within the corner region. For welding at the corner of the cold-formed structural hollow sections, there are provisions and requirements in Part EN 1993-1-8 of Eurocode. The SSAB Domex Tube structural hollow sections fulfill the given requirements, why they can be welded also at the corner without special actions (see more details in Chapter 3, Table 3.7).

The choice of the welding consumable has been discussed in clause 3.3.5.1 of Chapter 3.

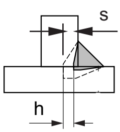
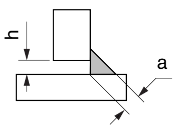
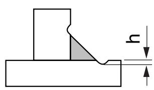
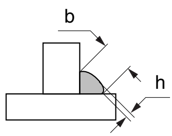
8.5.1 Weld quality levels and tolerances for weld imperfections

Standard EN ISO 5817 defines weld quality levels that are applied in the fabrication of welded joints. Moreover, in terms of weld quality levels, the standard comprises three different levels, designated with symbols D (moderate), C (intermediate) and B (stringent). The weld quality level to be applied is determined on the basis of the EXC execution class of the component as presented in Annex A of EN 1090-2 (Table 8.4 in this handbook). Usually the weld quality level B or C is arrived at.

Tolerances for fillet welds in each weld quality level B...D according to EN ISO 5817 are presented in Table 8.9.

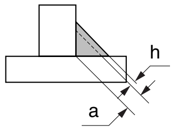
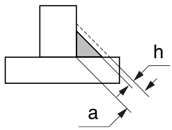
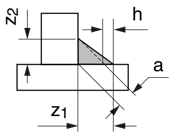
In regard to fatigue loaded structures, standard EN 1090-2 and Parts EN 1993-1-8 and EN 1993-1-9 of Eurocode, which refer to EN ISO 5817 in respect to weld quality levels, define the basic provisions for the quality level. In regard to fatigue loaded structures, in addition to above provisions it may be necessary to determine some supplementary quality requirements for the welds, as presented in [5].

Table 8.9 Permitted weld imperfections for fillet welds [1,6]

Imperfection designation ISO 6520-1		Weld quality level (EN ISO 5817)		
		D [a)] Moderate	C Intermediate	B Stringent
402 Lack of penetration t > 0,5 mm		Short imperfections [b)] h ≤ 0,2s max 2 mm	Short imperfections [b)] h ≤ 0,1s max 2 mm	Not permitted
617 Incorrect root gap		h ≤ 1 mm + 0,3a max 4 mm	h ≤ 0,5 mm + 0,2a max 3 mm	h ≤ 0,5 mm + 0,1a max 2 mm
		Gaps exceeding the appropriate limit may, in certain cases, be permitted if compensated for by a corresponding increase in the throat thickness, see EN 1090-2: clause 7.5.8.1		
5011 and 5012 Continuous and intermittent undercut t > 3 mm		h ≤ 0,2 t max 1 mm	h ≤ 0,1 t max 0,5 mm	h ≤ 0,05 t max 0,5 mm
		Smooth transition is required. This is not regarded as a systematic imperfection.		
503 Excessive convexity t ≥ 0,5 mm		h ≤ 1 mm + 0,25b max 5 mm	h ≤ 1 mm + 0,15b max 4 mm	h ≤ 1 mm + 0,1b max 3 mm

(continues)

Table 8.9 Permitted weld imperfections for fillet welds [1,6]
(continued)

Imperfection designation ISO 6520-1		Weld quality level (EN ISO 5817)		
		D [a)] Moderate	C Intermediate	B Stringent
5214 Excessive throat thickness $t \geq 0{,}5$ mm		Permitted	$h \leq 1$ mm + 0,2a but max 4 mm	$h \leq 1$ mm + 0,15a but max 3 mm
5213 Insufficient throat thickness $t > 3$ mm Not applicable to processes with proof of greater depth of penetration		Short imperfections [b)] $h \leq 0{,}3$ mm + 0,1a but max 2 mm	Short imperfections [b)] $h \leq 0{,}3$ mm + 0,1a but max 1 mm	Not permitted
512 Excessive unequal leg length $t \geq 0{,}5$ mm		$h \leq 2$ mm + 0,2a	$h \leq 2$ mm + 0,15a	$h \leq 1{,}5$mm + 0,15a

a) Systematic weld imperfection permitted. Systematic weld imperfection means an imperfection that is repeatedly distributed over the weld length to be examined, the size of a single imperfection being within the specified limits.
b) Short weld imperfection means one or more imperfection(s) having the total length of not greater than 25 mm in any 100 mm weld length, or not more than 25 % of the weld length in case when the weld length is less than 100 mm long. The region having the greatest number of imperfections is chosen.

8.5.2 Welding methods

For welding of structural hollow sections, mainly two welding methods are used: manual metal-arc welding with covered electrode and gas shielded metal-arc welding. Manual metal-arc welding is used mainly for on-site weldings. Its benefits are the light-weight and easily transportable equipment as well as non-sensitiveness for wind.

In workshop fabrication, gas shielded metal-arc welding (MIG/MAG welding) is the most common welding method. Its benefits are better productivity and the possibility for the automatization of the welding process.

When fabricating the welded structures, the requirements set by EN 1090-2 for the technical skills of the welder must be considered. Furthermore, EN 1090-2 requires the welder to be qualified using a specific test, if in hollow section lattice structures the angle between the chord and the brace members is $\theta < 60°$.

Welding shall be undertaken in accordance with the relevant Parts of EN ISO 3834 or EN ISO 14554 as applicable. Depending on the execution class, the following Parts of EN ISO 3834 shall be applied:

- EXC1 : Part 4. Elementary quality requirements
- EXC2 : Part 3. Standard quality requirements
- EXC3 and EXC4 : Part 2. Comprehensive quality requirements

As part of the production planning, a welding plan shall be provided. The welding plan shall include, as relevant:

- The welding procedure specifications including welding consumable, any preheating, interpass temperature and post weld heat treatment requirements;
- Measures to be taken to avoid distortions during and after welding;
- The sequence of welding with any restrictions or acceptable locations for start and stop positions, including intermediate stop and start positions where joint geometry is such that welding cannot be executed continuously;
- Requirements for checkings during welding;
- Turning of components in the welding process, in connection with the sequence of welding;
- Details of restraints to be applied;
- Measures to be taken to avoid lamellar tearing;
- Special equipment and requirements for welding consumables (low hydrogen content, control of humidity and temperature etc.);
- Shape and finishing of the weld;
- Requirements for acceptance criteria of welds, as well as cross reference to the inspection and testing plan;
- Requirements for identification of the welds;
- Requirements for surface treatment.

Welding shall be carried out with qualified procedures using a welding procedure specification (WPS) in accordance with the applicable Part of EN ISO 15609 or EN ISO 14555 or EN ISO 15620, as relevant. The qualification procedures are presented in Table 8.10. The qualification of the welding procedure depends on the execution class, the parent metal and the mechanization grade, as presented in Table 8.10.

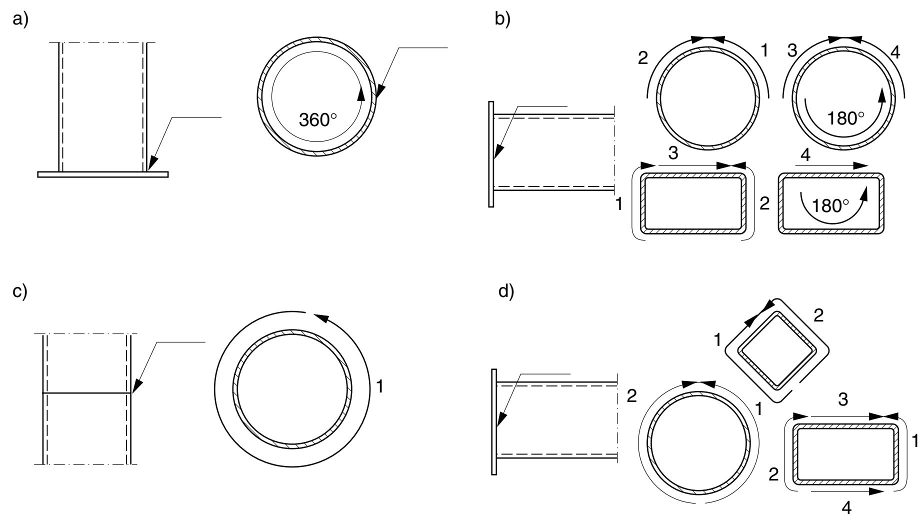
Table 8.10 Qualification of welding procedures for manual metal-arc welding, self-shielded tubular cored arc welding, submerged arc welding, MIG/MAG and TIG welding [1]

Method of qualification	Standard	Execution class		
		EXC2	EXC3	EXC4
Welding procedure test	EN ISO 15614-1	X	X	X
Pre-production welding test	EN ISO 15613	X	X	X
Standard welding procedure	EN ISO 15612	X a)	–	–
Previous welding experience	EN ISO 15611	X b)	–	–
Tested welding consumables	EN ISO 15610			
X Permitted – Not permitted				
a) Only for materials ≤S355 and only for manual welding or partly mechanized welding. b) Only for materials ≤S275 and only for manual welding or partly mechanized welding.				

8.5.3 Welding sequence

Considering the stresses generated by welding, as well as the deformations of the joined components, the correct welding sequence is important. The welding of structural hollow sections should not be started nor stopped at the corner of the hollow section. In hollow section structures, stress peaks are typically generated at corners, which the notch effect of start and stop positions increases further. Consequently, the fatigue resistance of the joint decreases.

The welding sequence depends also on the welding position and on the mobility of the hollow section. In Figure 8.10a, the hollow section can be rotated horizontally. The welding point thus stays the same and the hollow section is rotated a full circle. In Figure 8.10b, the hollow section is positioned horizontally and can also be rotated about its axis. Thus the welding direction is at first from the bottom upwards. The lower side is welded after rotating the hollow section. In Figure 8.10c, the hollow section is fixed in a vertical position. The welding is performed continuously around the entire section. In Figure 8.10d, the hollow section is fixed in a horizontal position. Now, the lower seam must be welded from below.



NOTE: On square and rectangular hollow sections, the start and stop positions for welds should not be located at the corner, but at some distance from it.

Figure 8.10 Examples of welding sequence of structural hollow section joints in different positions and situations

For welding sequence in hollow section joints, there is general guidance given in Annex E of EN 1090-2. The end-to-end joints of hollow sections (for example, in lattice structures the end-to-end joint of the chord) should be executed as follows:

- start and stop positions of welds for end-to-end joints in chords should be chosen such that they are not located under the weld between the chord and a brace member
- start and stop positions of welds between two in-line square or rectangular hollow sections should not be located at or close to the corner positions

For other joint types of hollow sections, EN 1090-2 gives the following general guidance:

- welding between hollow sections should be completed all round, even if this total weld length would not be necessary for joint resistance
- start and stop positions of welds should not be located at or close to the corner positions of a joint between a square or rectangular hollow section brace member and a hollow section chord
- start and stop positions of welds should not be located at or close to the toe position or lateral flank positions of a joint between two circular hollow sections, but in accordance with Figure 8.11b
- recommended welding sequence for welding brace-to-chord joints is presented in Figure 8.11

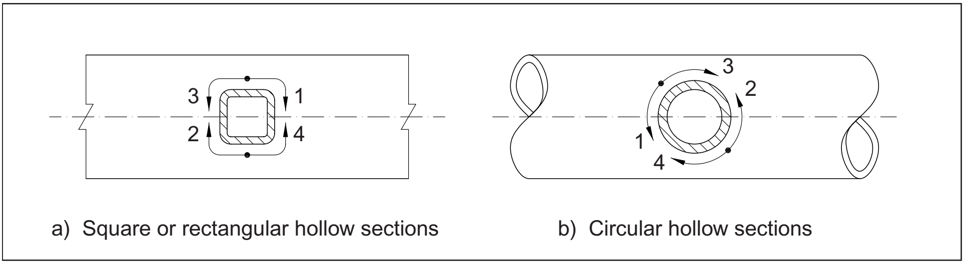


Figure 8.11 Welding sequence according to EN 1090-2 [1]

8.5.4 Joint preparation

With hollow sections, the aim is to design joints in such a manner that fillet welds can be used. The maximum throat thickness welded by one pass is 5 mm. A fillet weld is the simplest and most cost-efficient weld type, since no joint preparation is needed. However, depending on the joint geometry, in some cases a groove must be made to ensure a sufficient throat thickness of the weld. It is advisable to make the weld as symmetrical as possible to minimize the consumption of the welding consumable.

8.5.4.1 Joint preparation for hollow section end-to-end joints

According to EN 1090-2, for joints between hollow sections welded from one side, the joint preparations given in applicable Parts of EN ISO 9692 shall be used.

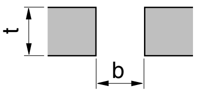
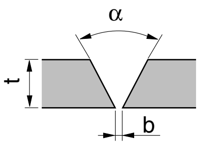
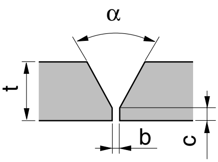
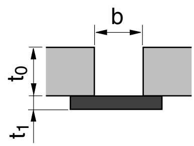
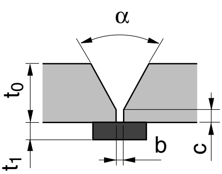
The standard EN ISO 9692 is, however, not adressed particularly for joints of structural hollow sections, but the instructions given in it for the geometrical dimensions of joint preparation are only of empirical nature for general applications. Thus, considering the dimensional range of hollow sections, the instructions given in EN ISO 9692 are partly insufficient. Based on [2], Table 8.11 presents empirical guidance for joint preparation of end-to-end joints better applicable specially for hollow sections and their dimensional range.

According to EN 1090-2, for end-to-end joints of hollow sections it is possible to use a backing ring or strip (Figure 8.12).

Figure 8.13 presents guidance given in EN 1090-2 for end-to-end joints between hollow sections having different wall thickness.

Guidance for the joint preparation and fit-up for mitre butt joints are locally the same as above for end-to-end joints of hollow sections. For a mitre butt joint, the bevel angle needs to be increased on the inside of the mitre and reduced on the outside as shown in Figure 8.14.

Table 8.11 Welded end-to-end joint of hollow sections for MIG/MAG welding, when the hollow sections have equal wall thickness [2]

Groove type	Wall thickness	α	b	c	Backing thickness
Without backing plate					
	$t \leq 3$ mm	0°	t	-	-
	$3 \leq t \leq 20$ mm	≈ 60°	$0 \leq b \leq 3$ mm	-	-
	$t \leq 20$ mm	≈ 60°	$0 \leq b \leq 4$ mm	$1{,}5 \leq c \leq 4$ mm	-
With backing plate					
	$t_0 = 3$ mm	0°	$3 \leq b \leq 5$ mm	-	$t_1 = 3$ mm
	$t_0 = 5$ mm	0°	$5 \leq b \leq 6$ mm	-	$3 < t_1 \leq 5$ mm
	$t_0 = 6$ mm	0°	$6 \leq b \leq 8$ mm	-	$3 < t_1 \leq 6$ mm
	$t_0 < 20$ mm	> 60°	$5 \leq b \leq 8$ mm	$1 \leq c \leq 2{,}5$ mm	$3 < t_1 \leq 6$ mm

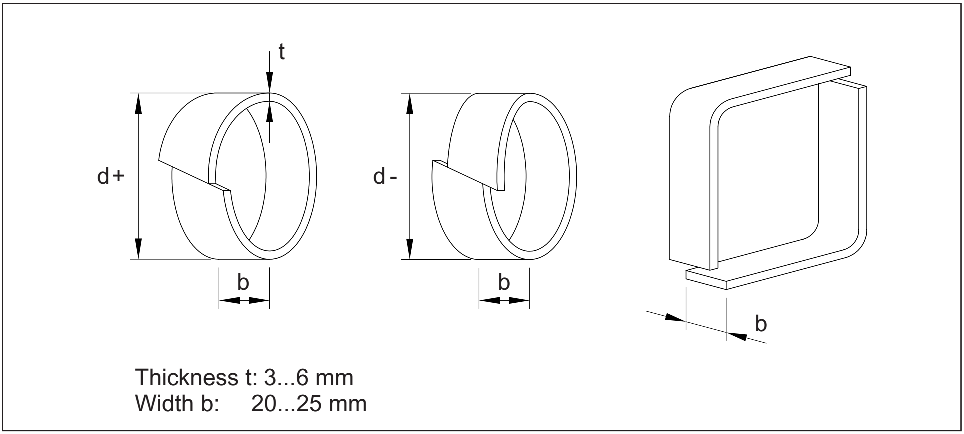


Figure 8.12 Shapes of backing rings or tapes suitable for end-to-end joints of hollow sections [1]

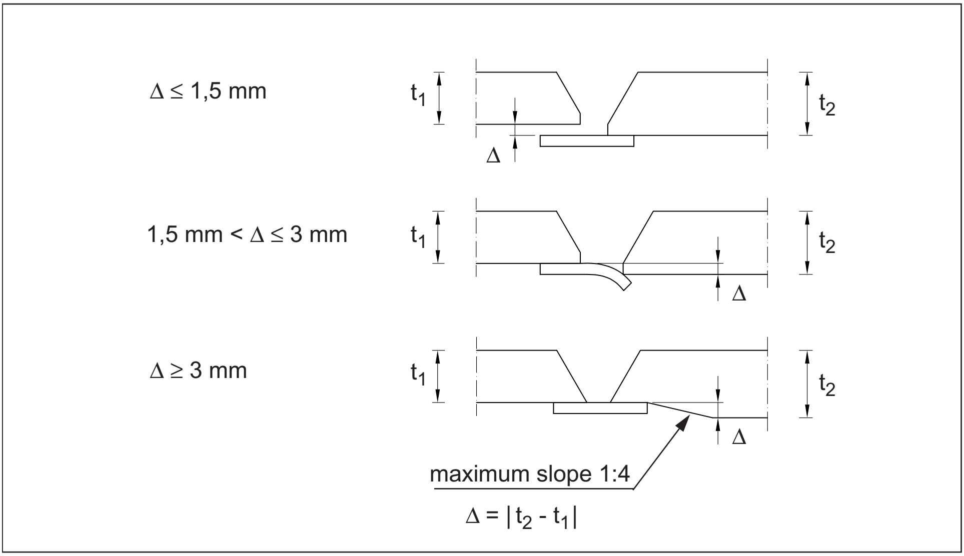


Figure 8.13 End-to-end joint using backing, when the hollow sections have different wall thickness [1]

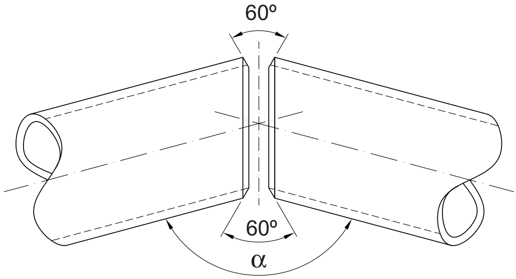


Figure 8.14 Joint preparation for hollow section mitre joints [1]

8.5.4.2 Joint preparation for hollow section lattice joints

In welded joints between brace members and chord, the local profile of the weld gap should be chosen such that a smooth transition from butt welds to the fillet welds is ensured. As butt welds, in accordance with EN 1090-2, the welds presented in Figure 8.15 (circular hollow sections) and in Figure 8.17 (square and rectangular hollow sections) are used. As fillet welds, the welds presented in Figure 8.16 (circular hollow sections) and in Figure 8.18 (square and rectangular hollow sections) are used.

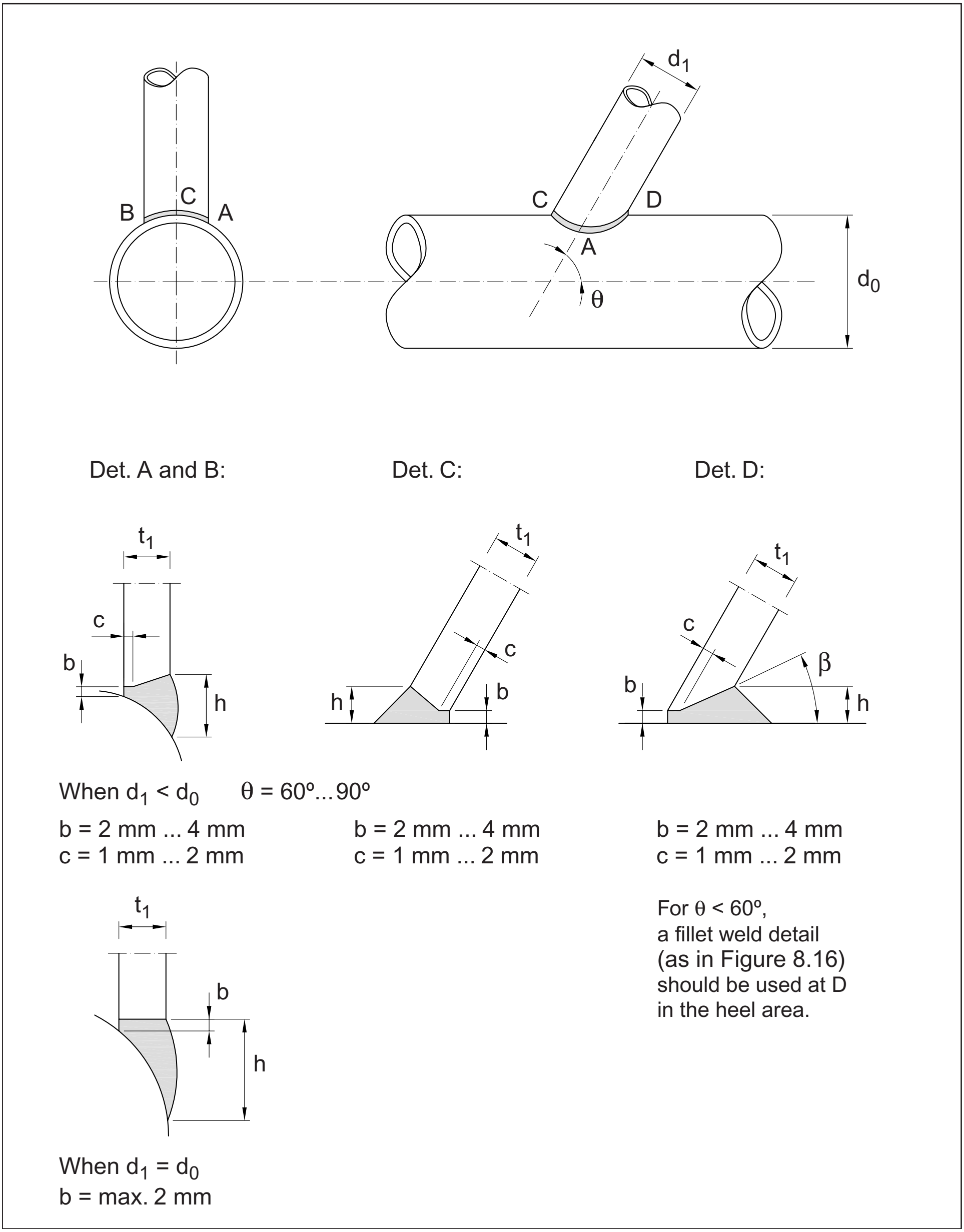


Figure 8.15 Butt welds in lattice joints of circular hollow sections. Joint preparation and fit-up [1]

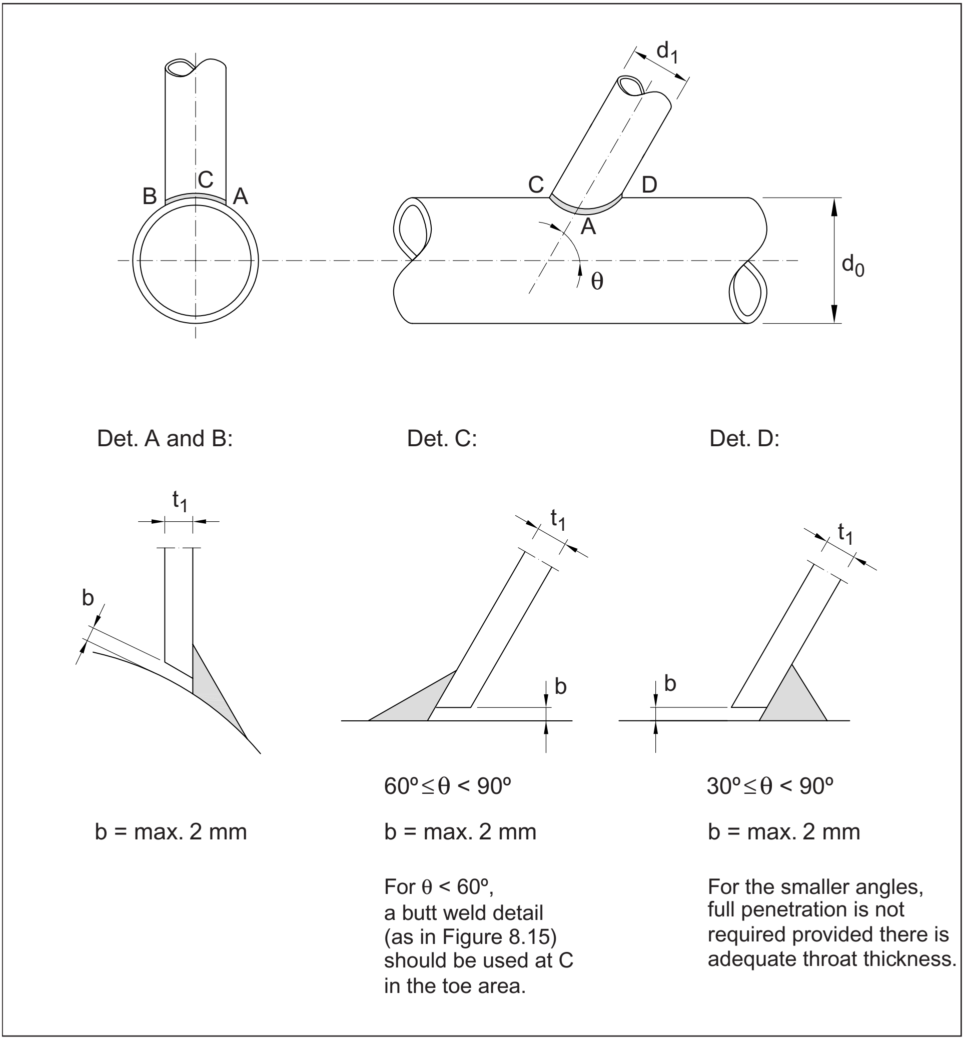


Figure 8.16 Fillet welds in lattice joints of circular hollow sections. Joint preparation and fit-up [1]

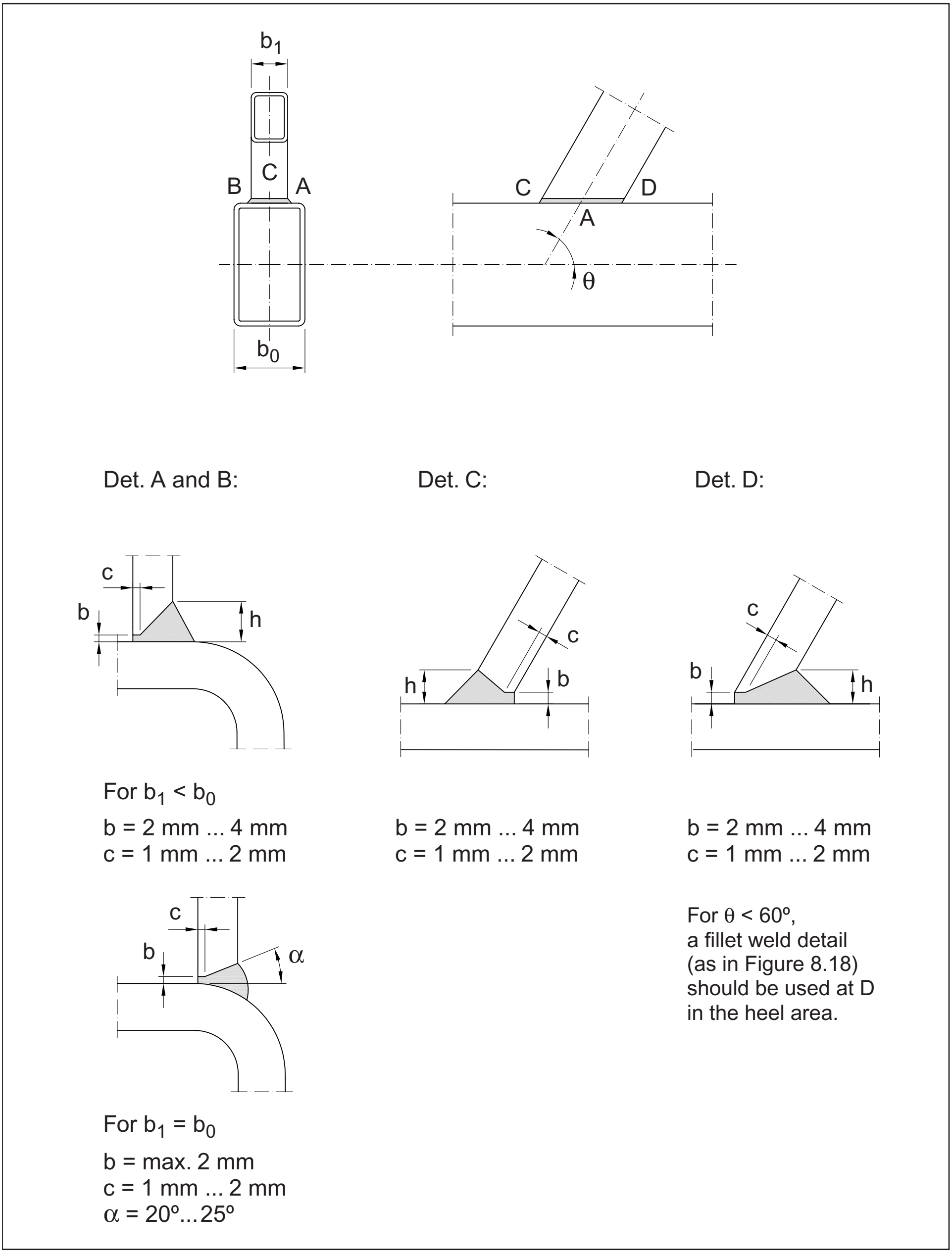


Figure 8.17 Butt welds in lattice joints of square and rectangular hollow sections. Joint preparation and fit-up [1]

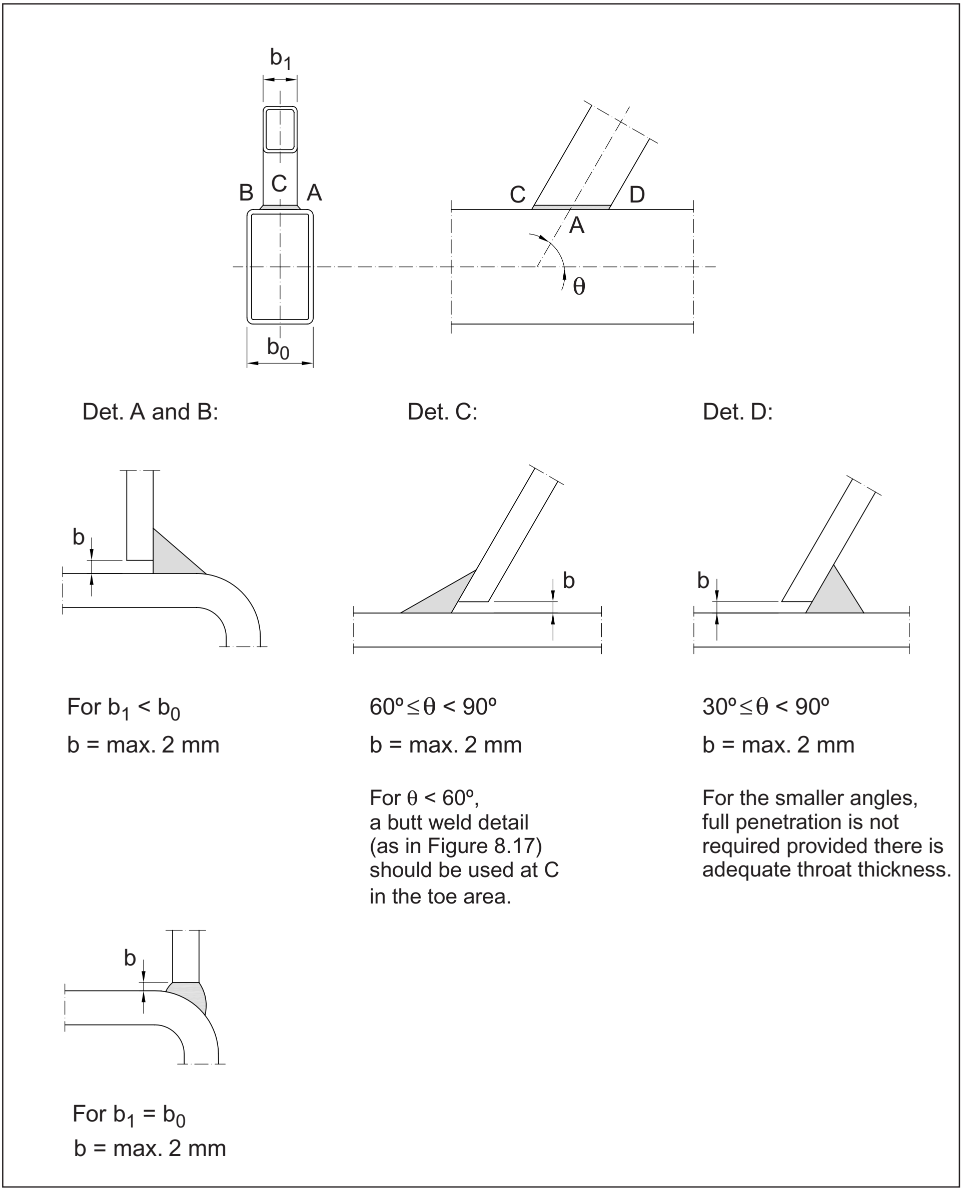


Figure 8.18 Fillet welds in lattice joints of square and rectangular hollow sections. Joint preparation and fit-up [1]

8.5.5 Preheating

The structural steels used in structural hollow sections do not normally require preheating or elevated interpass temperature. However, if the surface of the hollow section is damp or the temperature of the steel to be welded is below 5 ºC, a suitable preheating may be necessary. For steel grades higher than S355, a suitable preheating shall be provided if the temperature of the steel is below 5 ºC [1].

For thermal cutting, preheating is normally not required if the carbon equivalent of steel is less than 0,45 [7].

8.5.6 Residual stresses

In welding, residual stresses due to the temperature are always generated in the steel. This may result in permanent deformations of the component.

Deformations are generated both parallel and perpendicular to the weld direction. The magnitude of the deformation depends on the material to be welded, the number of weld passes and the rigidity of the structure. In addition to deformation, there is also angular distortion, in which the angle between welded elements changes due to the welding stresses.

Figure 8.19 illustrates the angular distortions due to welding, when deformations are allowed to take place freely.

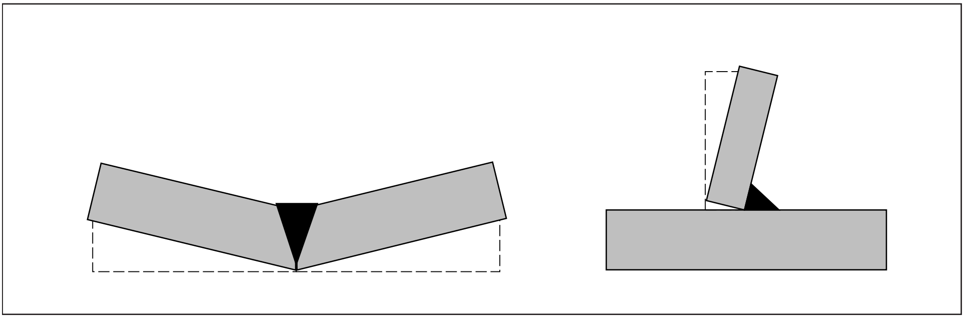


Figure 8.19 Angular distortions due to welding

In practice, the elements are often fixed/restrained during welding in such a manner that the deformations cannot take place freely, why stresses develop in the material. The deformation is significantly reduced if a fillet weld is made as double-sided, or if a double V-groove is used instead of a single V-groove. In this case, welding is performed alternately from both sides. A single-sided V-groove can also be welded by placing the welded elements in an angle which eliminates the deformation due to welding. The magnitude of the angle is estimated based on empirical knowledge.

When welding an entire structural element (for example a lattice), it is recommendable to first tack weld the joints, then make the fillet welds and finally the full penetration single-V butt welds [3].

Stress relieving using heat treatment is usually needed only in special cases. Stress relieving temperature is normally 550 - 600 °C. In this temperature range, a recommended hold time is 2 x t [minutes] (t = maximum material thickness [mm]), however at least 30 minutes [7].

The following factors, for example, increase the residual stresses caused by welding:

- large throat thickness of the weld
- great distance of the weld from the neutral axis of the hollow section
- restraints applied for the hollow sections during welding
- high rigidity of the hollow sections to be welded
- wrong welding sequence
- welding methods with high heat input.

8.5.7 Inspections of welds

Inspections before and during welding shall be presented in the inspection plan according to the requirements of EN ISO 3834.

Standard EN 1090-2 defines the general minimum hold times after welding. The hold times depend on weld size, heat input and the steel grade.

Table 8.12 Minimum hold times for welds before NDT inspection [1]

Weld size (mm) a)	Heat input Q (kJ/mm) b)	Hold time (hours) c)	
		S235 to S460	above S460
a or s ≤ 6	All	Cooling period only d)	24
6 < a or s ≤ 12	≤3	8	24
	>3	16	40
a or s > 12	≤3	16	40
	>3	24	48

a) Size applies to the nominal throat thickness *a* of a fillet weld or the nominal material thickness *s* of a full penetration weld. For individual partial penetration butt welds applies the nominal weld depth *a*, but for pairs of partial penetration butt welds it is the sum of the weld throats *a*.
b) Heat input Q to be calculated in accordance with Clause 19 of EN 1011-1:1998.
c) The time between weld completion and commencement of NDT shall be stated in the NDT report.
d) In the case of "cooling period only" this will last until the weld is cool enough for NDT to commence.

8.5.7.1 Non-destructive testing methods

Welded joints are normally tested in the workshop by using non-destructive testing (NDT) methods. These methods include [1]:

- visual inspection (VT)
- magnetic particle inspection (MT)
- penetrant testing (PT)
- ultrasonic testing (UT) and
- radiographic testing (RT).

The most common inspection methods used in workshops are visual inspection, magnetic particle inspection and ultrasonic testing.

Visual inspection (EN ISO 17637)

Standard EN ISO 17637 has superseded standard EN 970.

All welds are visually inspected along their full lengths just after completion of the welding. The welders, supervisors and inspectors take part in the inspection. Inspecting does not, for the time being, require formal qualification, but the inspector is required to have experience and knowledge on inspection of welds and welding. The requirements set for the inspector are presented in EN ISO 17637. A lamp and a mirror can be used as instruments. Visual testing includes inspection of the location, size and shape of the welds, inspection of the throat thickness of the fillet weld, inspection of the surfaces as well as detection of surface imperfections (for example undercut), stray arcs and weld spatter [8].

Magnetic particle inspection (EN ISO 17638, EN ISO 23278)

Standard EN ISO 17638 has superseded standard EN 1290, and standard EN ISO 23278 has superseded standard EN 1291.

Magnetic particle inspection is able to detect defects which are located at surface or near to it, such as gas pores, cracks and lacks of fusion. The inspector shall be qualified to perform magnetic particle inspection (EN 473, level 2).

After cleaning the weld surface, the weld is sprayed with a contrast colour. The weld is magnetized and, at the same time, magnetic powder is applied. The excess magnetic powder is blown off. The powder agglomerates around the surface defects allowing them to be seen against the contrast colour [8].

Penetrant testing (EN 571, EN ISO 23277)

Standard EN ISO 23277 has superseded standard EN 1289.

In addition to the aforementioned methods, also penetrant testing is applicable for detecting surface defects. The method is mainly used to inspect non-magnetic materials (aluminium, stainless steel). The inspector shall be qualified to perform penetrant testing (EN 473, level 2).

The weld surface is cleaned with cleaning liquid, and the penetrant colour is sprayed onto the surface. The excess penetrant colour is swept off, and developer agent is sprayed onto the weld surface. The developer agent reacts with the penetrant colour bringing it to the surface. The surface defects become coloured, because the penetrant colour gathers at the locations of discontinuity [8].

Ultrasonic testing (EN ISO 11666, EN ISO 17640, EN ISO 23279)

Standard EN ISO 11666 has superseded standard EN 1712, standard EN ISO 23279 has superseded standard EN 1713, and standard EN ISO 17640 has superseded standard EN 1714.

Ultrasonic testing can be used to detect internal defects in the weld and in the material. Internal defects include for example root defects, underbead cracks, gas pores and micro cracks. The method applies best to the inspection of full penetration weld seams. The inspector shall be qualified to perform ultrasonic testing (EN 473, level 2).

The measuring instrument is calibrated and adjusted for the material thickness to be inspected. Coupling media is spread on the surface to achieve good contact. The instrument emits sonic waves into the material, and the waves are reflected back from the locations of the defect or from the surface of the material. If the sound does not reflect before it reaches the surface again, there are not defects in the inspected area. The area to be inspected should be scanned using several different angles, because the defects which are parallel to the sonic waves may be difficult to detect.

Ultrasonic testing is not applicable for small material thicknesses. In EN ISO 17640, the minimum thickness is specified as 8 mm. Ultrasonic testing is very challenging, which is why in practice it is recommended to prefer experienced inspectors.

Radiographic testing (EN 1435, EN 12517)

Internal defects as well as surface defects can be detected also using radiographic testing. The method applies mainly for inspection of butt joints. The material thickness may vary from less than 1 mm thickness up to ca. 30 mm thickness. On thicker materials, the exposure time becomes considerably longer [8]. The inspector shall be qualified to perform radiographic testing (EN 473, level 2).

The film is set on the other side of the object to be inspected. X-ray radiation is directed from the opposite side through the object, and the image projects to the film. The film is developed, after which the possible defects can be studied. During the inspection it is not possible to work in the vicinity of the object due to the danger of radiation, and this may interrupt the production for the time of inspection.

8.5.7.2 Extent of inspection for NDT methods

For welds, the extent of inspection and requirements to be applied are presented in EN 1090-2. In execution class EXC1, only visual inspection is required, if not otherwise specified. The extent for NDT inspections other than visual inspection, is presented in Table 8.13 according to EN 1090-2.

Table 8.13 Extent of supplementary NDT [1]

Type of weld	Workshop and site welds		
	EXC2	EXC3	EXC4
Transverse butt welds and partial penetration butt welds subjected to tensile stress:			
Utilization grade for quasi-static actions: U ≥ 50 %	10 %	20 %	100 %
Utilization grade for quasi-static actions: U < 50 %	0 %	10 %	50 %
Transverse butt welds and partial penetration welds:			
In cruciform joints	10 %	20 %	100 %
In T joints	5 %	10 %	50 %
Transverse fillet welds in tension or shear:			
With a > 12 mm or t > 20 mm	5 %	10 %	20 %
With a ≤ 12 mm and t ≤ 20 mm	0 %	5 %	10 %
Full penetration longitudinal welds between web and top flange of crane girders	10 %	20 %	100 %
Other longitudinal welds and welds to stiffeners	0 %	5 %	10 %
Longitudinal welds are those made parallel to the component axis. All the others are considered to be transverse welds. Utilization grade is defined for welds for quasi-static actions. $U = E_d/R_d$, where E_d is the largest action effect of the weld and R_d is the resistance of the weld at the ultimate limit state. Terms *a* and *t* refer respectively to the throat thickness and the thickest material to be joined.			

According to EN 1090-2, the welds shall be inspected visually throughout their entire length. If surface defects are detected, the weld shall be inspected again, now using penetrant testing or magnetic particle inspection [1].

When welding according to a new welding procedure specification (WPS), the first 5 joints shall fulfill the following requirements:

- weld quality level B is required to prove the compliance of the WPS in production conditions
- the extent of the inspection shall be double compared to the values given in Table 8.13 (min. 5 %, max. 100 %)
- the minimum length to be inspected is 900 mm.

If the inspection gives non-conforming results, the reasons should be worked out and a new set of five joints shall be inspected [1].

8.5.7.3 Destructive testing

Destructive testing is mainly used when performing welding and procedure tests, so they are normally not applied in daily inspection activity in the workshop.

Samples

A sample perpendicular to the welded surface is cut from the welded component. After grinding and smoothing, the sectional surface is acidified. The parent material and the weld zone can be clearly distinguished after acidifying. The method is applied in procedure testing to define the magnitude of penetration [8].

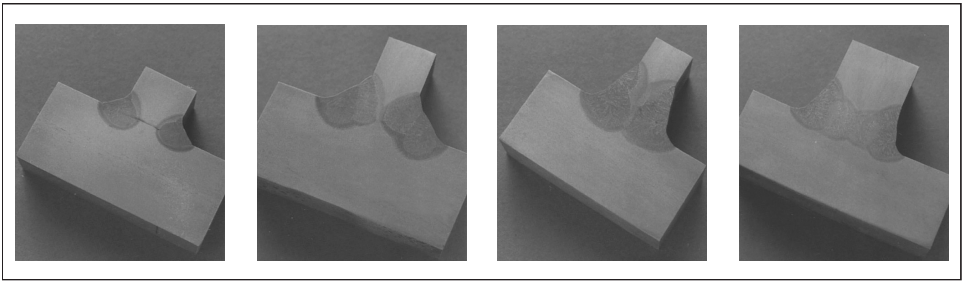


Figure 8.20 Samples of weld seam

Weld break tests

Weld break tests are used, for example, in connection with the qualification tests of welders. After welding, the specimen shall be broken through the welded area. By examining the fractured area and the surface of fracture, it is possible to study the strength of the weld metal in proportion to the parent material, or the fusion between weld and parent metal [8].

8.6 Tolerances

8.6.1 Manufacturing tolerances

Standard EN 1090-2 defines two different types of geometric tolerances for the structures and components to be manufactured:

- essential geometrical tolerances (Tables 8.14 - 8.16) and
- functional geometrical tolerances (Tables 8.17 - 8.20).

The essential geometrical tolerances secure the strength and stability of the structure, but do not secure functionality. **The functional geometrical tolerances**, which are generally divided in two tolerance classes, secure the functionality of the structure. In practice the division is not so clear. At the same time, the structure shall fulfill both the essential tolerances and also the functional tolerances, in latter the tolerance class 1 or 2 as given in the execution specification. In respect to the structures manufactured from structural hollow sections, the most substantial manufacturing tolerances are presented in Tables 8.14 - 8.20. Complete tolerance tables, see EN 1090-2.

In respect to welded constructions EN 1090-2 gives additionally a possibility to use, instead of the functional tolerances, the classes defined in EN 1090-2 according to EN ISO 13920.

The tolerance requirements differing from the tables shall be presented in the execution specification and in the manufacturing drawings. In such a case also the region where the differing requirement are to be applied, shall be presented.

Table 8.14 Essential manufacturing tolerances – Fastener holes and notches [1]

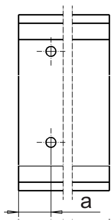
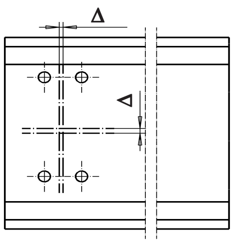
Criterion	Parameter	Permitted deviation Δ
Position of holes for fasteners:	Deviaton Δ of centreline of an individual hole from its intended position within a group of holes	Δ = ± 2 mm
Position of holes for fasteners: 	Deviation Δ in distance a between an individual hole and a cut end	- Δ = 0 mm (no positive value given)
Position of hole group: 	Deviation Δ of a hole group from its intended position	Δ = ± 2 mm

Table 8.15 Essential manufacturing tolerances – Cylindrical and conical shells [1]

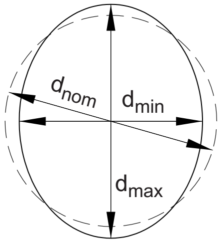
Manufacturing tolerance quality class	Out-of-roundness: permitted deviation Δ [a)] Internal diameter $d_i \leq 0{,}50$ m [b)]	Out-of-roundness: permitted deviation Δ [a)] Internal diameter $0{,}5 \text{ m} < d_i < 1{,}25$ m [b)]	Out-of-roundness: permitted deviation Δ [a)] Internal diameter $d_i \geq 1{,}25$ m [b)]
Class A	$\Delta = \pm 0{,}014$	$\Delta = \pm [0{,}007 + 0{,}0093\,(1{,}25 - d_i)]$	$\Delta = \pm 0{,}007$
Class B	$\Delta = \pm 0{,}020$	$\Delta = \pm [0{,}010 + 0{,}0133\,(1{,}25 - d_i)]$	$\Delta = \pm 0{,}010$
Class C	$\Delta = \pm 0{,}030$	$\Delta = \pm [0{,}015 + 0{,}0200\,(1{,}25 - d_i)]$	$\Delta = \pm 0{,}015$

a) Out-of-roundness:
Difference between the maximum and minimum values of the measured internal diameter, relative to the nominal internal diameter:

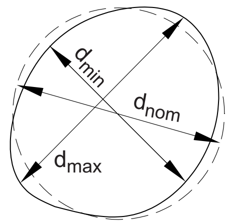
$$\Delta = \frac{d_{max} - d_{min}}{d_{nom}}$$

b) d_i is the nominal internal diameter in metres

flattening:



unsymmetrical:



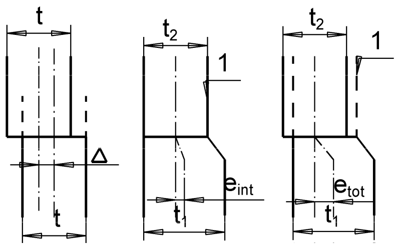
Manufacturing tolerance quality class	Misalignment: Permitted deviation Δ
Class A	$\Delta = \pm 0{,}14\,t$ but $\lvert\Delta\rvert \leq 2$ mm
Class B	$\Delta = \pm 0{,}20\,t$ but $\lvert\Delta\rvert \leq 3$ mm
Class C	$\Delta = \pm 0{,}30\,t$ but $\lvert\Delta\rvert \leq 4$ mm

Misalignment:
Non-intended eccentricity of plates at a horizontal joint.

At a change of plate thickness, the intentional part of the eccentricity is not included

At a change of plate thickness:
$t = (t_1 + t_2)/2$
$\Delta = e_{tot} - e_{int}$

where:
t_1 is the larger thickness;
t_2 is the smaller thickness.



Key: 1 = intended joint geometry

(continues)

Table 8.15 Essential manufacturing tolerances – Cylindrical and conical shells [1] *(continued)*

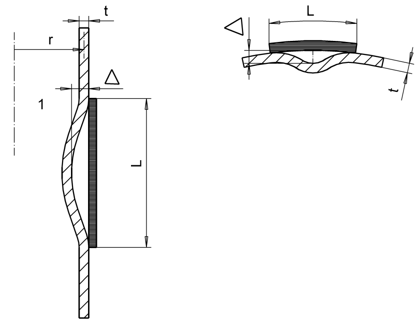
	Dents (dimples):
Manufacturing tolerance quality class	Permitted deviation **Δ**
Class A	**Δ** = ± 0,006 L
Class B	**Δ** = ± 0,010 L
Class C	Δ = ± 0,016 L
Dents (Dimples) : a) Meridionally: $L = 4\ (rt)^{0,5}$ b) Circumferentially (gauge radius = r) : $L = 4\ (rt)^{0,5}$ $L = 2,3\ (h^2 rt)^{0,25}$ but $L \leq r$ where h is the axial length of the shell segment c) Additionally, across welds: $L = 25t$ but $L \leq 500$ mm At a change of thickness: $t = t_2$	 Key: 1 = inward
Note: With reference to the manufacturing tolerance quality classes in EN 1993-1-6 , Class A = Excellent, Class B = High, and Class C = Normal.	

Table 8.16 Essential manufacturing tolerances – Lattice components [1]

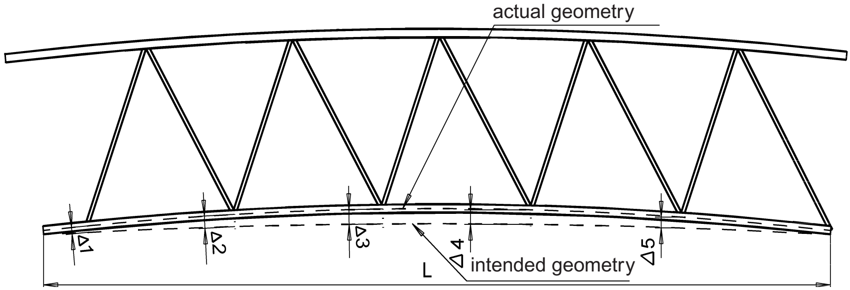
Criterion	**Parameter**	**Permitted deviation Δ**
Straightness and camber: 		
Position of the nodal points:	Deviation **Δ** of each nodal point from the intended line	**Δ** = ± L/500 but |**Δ**| ≥ 12 mm
Straightness of the brace members:	Straightness deviation on the length L_i of each brace member	**Δ** = ± L_i/750 but |**Δ**|≥6 mm
Note: Notations such as **Δ** = ± L/500 but |**Δ**| ≥ 12 mm mean that |**Δ**| is greater of the values L/500 and 12 mm.		

Table 8.17 Functional manufacturing tolerances – Components [1]

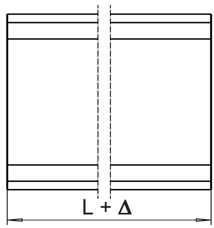
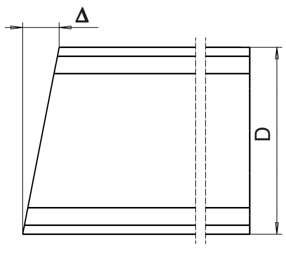
Criterion	Parameter	Permitted deviation Δ	
		Class 1	**Class 2**
Length: 	Deviation Δ of cut length measured on the centreline (or on the corner for an angle). Length L measured including welded end-plates	Δ = ± (L/5000 + 2) mm Ends ready for full contact bearing: Δ = ± 1 mm	Δ = ± (L/10000 + 2) mm Ends ready for full contact bearing: Δ = ± 1 mm
Length, where sufficient compensation with adjacent component is possible:	Cut length measured on centreline	Δ = ± 50 mm	Δ = ± 50 mm
Straightness:	See product standard EN 10219 for structural hollow sections.		
Squareness of ends: 	Deviation Δ of squareness to longitudinal axis: - ends intended for full contact bearing - ends not intended for full contact bearing	 Δ = ± D/1000 Δ = ± D/100	 Δ = ± D/1000 Δ = ± D/300 but |Δ| ≤ 10 mm
Twist:	See product standard EN 10219 for structural hollow sections.		
Note: Notations such as Δ = ± d/100 but |Δ| ≥ 5 mm mean that |Δ| is greater of the values d/100 and 5 mm.			

Table 8.18 Functional manufacturing tolerances – Fastener holes and notches [1]

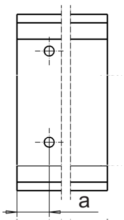
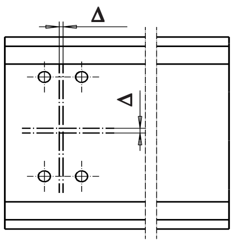
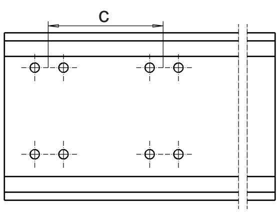
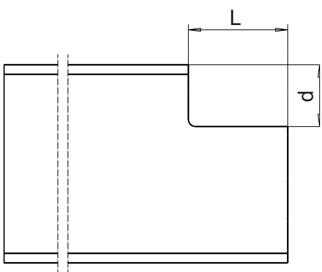
Criterion	Parameter	Permitted deviation Δ	
		Class 1	Class 2
Position of holes for fasteners:	Deviation Δ of centreline of an individual hole from its intended position within a group of holes	Δ = ± 2 mm	Δ = ± 1 mm
Position of holes for fasteners: 	Deviation Δ in distance *a* between an individual hole and a cut end	- Δ = 0 mm + Δ ≤ 3 mm	- Δ = 0 mm + Δ ≤ 2 mm
Position of hole group: 	Deviation Δ of a hole group from its intended position	Δ = ± 2 mm	Δ = ± 1 mm
Spacing of hole groups: 	Deviation Δ in spacing *c* between hole groups: - general case - where a single piece is connected by two groups of fasteners	 Δ = ± 5 mm Δ = ± 2 mm	 Δ = ± 2 mm Δ = ± 1 mm
Notches: 	Deviation Δ of notch depth and length: - depth d - length L	 - Δ = 0 mm + Δ ≤ 3 mm - Δ = 0 mm + Δ ≤ 3 mm	 - Δ = 0 mm + Δ ≤ 2 mm - Δ = 0 mm + Δ ≤ 2 mm

Table 8.19 Functional manufacturing tolerances – Column splices and base plates [1]

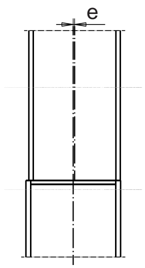
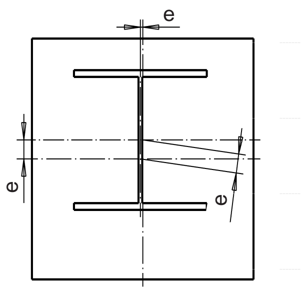
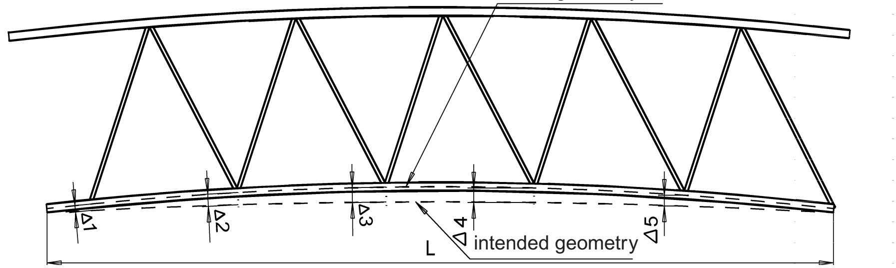
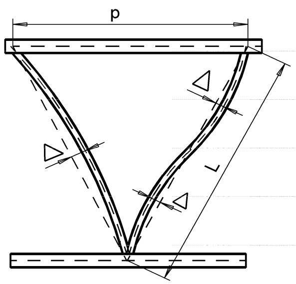
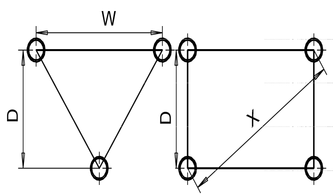
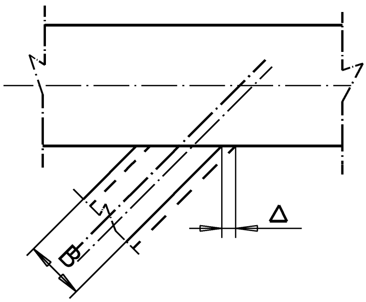
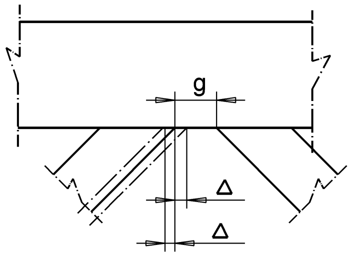
Criterion	**Parameter**	**Permitted deviation Δ**	
		Class 1	**Class 2**
Column splice: 	Non-intended eccentricity *e* (about either axis)	5 mm	3 mm
Base plate: 	Non-intended eccentricity *e* between the base plate and centreline of the column (in any direction)	5 mm	3 mm

Table 8.20 Functional manufacturing tolerances – Lattice components [1]

Criterion	Parameter	Permitted deviation Δ	
		Class 1	**Class 2**
Straightness and camber: 			
Position of the nodal points:	Deviation Δ of each nodal point from the intended line	Δ = ± L/500 but |Δ| ≥ 12 mm	Δ = ± L/500 but |Δ| ≥ 6 mm
Panel dimensions: 	Deviation Δ of individual distances *p* between intersections of centrelines at nodal points Cumulative deviation ∑p of nodal point position	Δ = ± 5 mm Δ = ± 10 mm	Δ = ± 3 mm Δ = ± 6 mm
Straightness of the brace members:	Straightness deviation on the length L_i of each brace member	Δ = ± L_i/500 but |Δ|≥6 mm	Δ = ± L_i/1000 but |Δ|≥3 mm
Cross-sectional dimensions: 	Deviation of distances D, W and X, if: s ≤300 mm: 300 < s < 1000 mm s ≥1000 mm	 Δ = ± 3 mm Δ = ± 5 mm Δ = ± 10 mm	 Δ = ± 2 mm Δ = ± 4 mm Δ = ± 6 mm

(continues)

Table 8.20 Functional manufacturing tolerances – Lattice components [1]
(continued)

Criterion	Parameter	Permitted deviation Δ	
		Class 1	**Class 2**
Intersecting joints: 	Eccentricity (relative to intended eccentricity)	Δ = ± (B/20 + 5) mm	Δ = ± (B/40 + 3) mm
Gap joints: 	Gap g between brace members: $g \geq (t_1 + t_2)$ where: t_1 and t_2 are the wall thicknesses of braces	|Δ| ≤ 5 mm	|Δ| ≤ 3 mm
Note: Notations such as Δ = ± L/500 but |Δ| ≥ 6 mm mean that |Δ| is greater of the values L/500 or and 6 mm.			

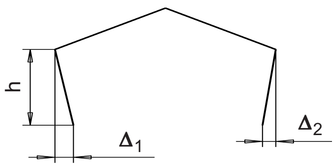
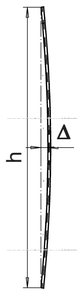
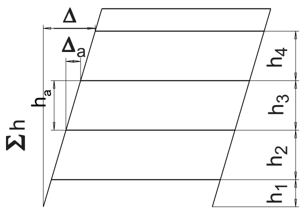
8.6.2 Erection tolerances

As the manufacturing tolerances, also erection tolerances of the structures are divided in EN 1090-2 into **essential** and **functional** geometrical tolerances:

- essential geometrical tolerances (Tables 8.21 - 8.22) and
- functional geometrical tolerances (Tables 8.23 - 8.26).

In respect to the structures manufactured from structural hollow sections, the most substantial manufacturing tolerances are presented in Tables 8.21 - 8.26. Complete tolerance tables, see EN 1090-2.

Table 8.21 Essential erection tolerances – Single storey and multi-storey columns and buildings [1].

Criterion	Parameter	Permitted deviation Δ
Inclination of columns of single storey portal frame buildings 	Overall inclination Mean inclination Δ of all the columns in the same frame [For two columns: $\Delta = (\Delta_1 + \Delta_2)/2$]	$\Delta = \pm h/300$ $\Delta = \pm h/500$
Straightness of a single storey column: 	Location of the column in plan, relative to a straight line between position points at top and bottom: - generally - structural hollow sections	 $\Delta = \pm h/750$ $\Delta = \pm h/750$
Inclination of multi-storey columns 	Inclination Δ of the column, relative to a vertical line through its centre at base level Inclination Δ_a of the column, relative to a vertical line through its centre at the next lower level Location of the column in plan, relative to a straight line between position points at adjacent storey levels	$\lvert\Delta\rvert = \Sigma h/(300\cdot\sqrt{n})$ $\Delta_a = \pm h/500$ $\Delta = \pm h/750$

(continues)

Table 8.21 *(continued)* Essential erection tolerances – Single storey and multi-storey columns and buildings [1].

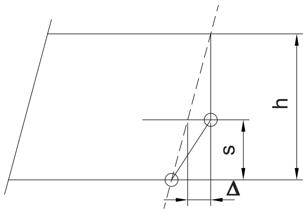
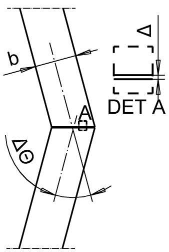
Criterion	Parameter	Permitted deviation Δ
Straightness of a spliced column between adjacent storey levels 	Location of the column in plan at the splice, relative to a straight line between position points at adjacent storey levels	$\Delta = \pm s/750$ with $s \leq h/2$
Full contact end bearing: 	Local angular misalignment $\Delta\theta$ occurring at the same time as gap Δ at point "X"	$\Delta\theta = \pm 1/500$ rad and: • Δ = 0,5 mm over at least two thirds of the area • Δ = 1,0 mm, maximum locally

Table 8.22 Essential erection tolerances – Beams subject to bending and components subject to compression [1]

Criterion	Parameter	Permitted deviation Δ
Straightness of beams subject to bending and components subject to compression if unrestrained	Deviation Δ from straightness	$\Delta = L/750$

Table 8.23 Functional erection tolerances – Concrete foundations and supports [1]

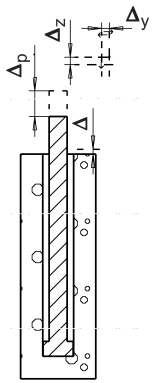
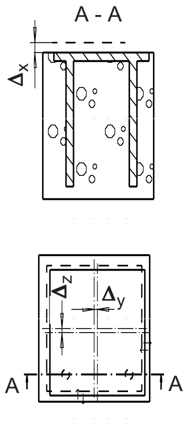
Criterion	Parameter	Permitted deviation Δ
Foundation level and foundation bolt: 	Deviation Δ from intended level Horizontal location of vertical wall at support point for steel component Adjustable foundation bolt: - location at tip - vertical protrusion Δ_p Pre-set foundation bolt where not prepared for adjustment: - location at tip - vertical protrusion Δ_p - horizontal protrusion Δ_p	$-15\ \text{mm} \leq \Delta \leq +5\ \text{mm}$ $\Delta = \pm 25$ mm Δ_y, $\Delta_z = \pm 10$ mm $-5\ \text{mm} \leq \Delta_p \leq +25\ \text{mm}$ Δ_y, $\Delta_z = \pm 3$ mm $-5\ \text{mm} \leq \Delta_p \leq +45\ \text{mm}$ $-5\ \text{mm} \leq \Delta_x \leq +45\ \text{mm}$
Steel anchor plate embedded in concrete: 	Deviations Δ_x, Δ_z, Δ_y from the intended location	Δ_x, Δ_z, $\Delta_y = \pm 10$ mm

Table 8.24 Functional erection tolerances – Positions of columns [1]

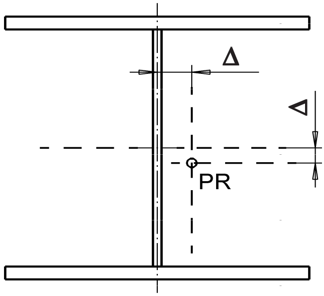
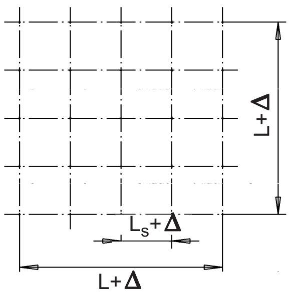
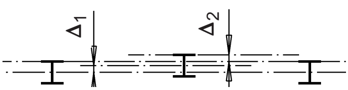
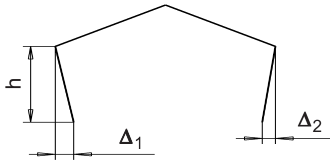
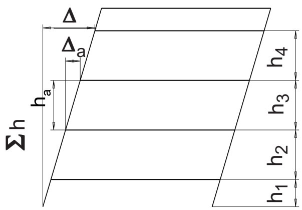
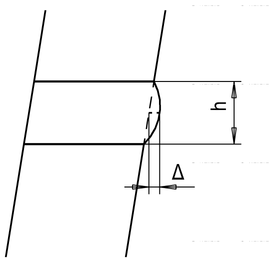
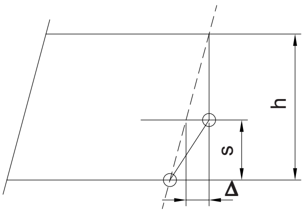
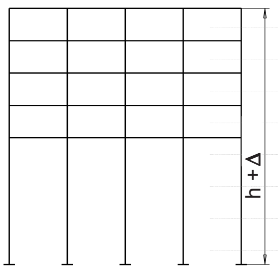
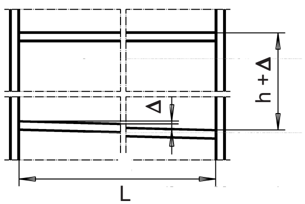
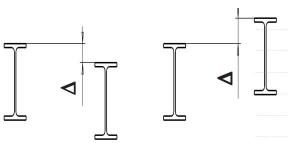
Criterion	Parameter	Permitted deviation Δ	
		Class 1	Class 2
Location: 	Location in plan of the centre of the column at the level of its base, relative to the position point of the reference (PR)	$\Delta = \pm 10$ mm	$\Delta = \pm 5$ mm
Overall length of a building and distance between columns: 	Distance between end columns in each line, at base level: $L \leq 30$ m 30 m $< L < 250$ m $L \geq 250$ m Distance between centres of adjacent columns at base level: $L_s \leq 5$ m $L_s > 5$ m	 $\Delta = \pm 20$ mm $\Delta = \pm 0{,}25 \cdot (L + 50)$ mm $\Delta = \pm 0{,}1 \cdot (L + 500)$ mm (L in metres) $\Delta = \pm 10$ mm $\Delta = \pm 0{,}2 \cdot (L + 45)$ mm (L_s in metres)	 $\Delta = \pm 16$ mm $\Delta = \pm 0{,}2 \cdot (L + 50)$ mm $\Delta = \pm 0{,}1 \cdot (L + 350)$ mm (L in metres) $\Delta = \pm 7$ mm $\Delta = \pm 0{,}2 \cdot (L + 30)$ mm (L_s in metres)
Column alignment 	Location of the centre of the column at base level, relative to the established column line (ECL) Location of the outer face of a perimeter column at base level, relative to the line joining the faces of the adjacent columns	$\Delta_1 = \pm 10$ mm $\Delta_2 = \pm 10$ mm	$\Delta_1 = \pm 7$ mm $\Delta_2 = \pm 7$ mm

Table 8.25 Functional erection tolerances – Columns of single storey and multi-storey buildings [1]

Criterion	Parameter	Permitted deviation Δ	
		Class 1	**Class 2**
Inclination of columns in single storey portal frames: 	Inclination, general case	$\Delta = \pm h/300$	$\Delta = \pm h/500$
	Inclination of each column Δ_1 or Δ_2	$\Delta = \pm h/150$	$\Delta = \pm h/300$
	Mean inclination **Δ** of all the columns in the same frame [for two columns $\Delta = (\Delta_1 + \Delta_2)/2$]	$\Delta = \pm h/500$	$\Delta = \pm h/500$
Inclination of columns, multi-storey buildings: 	Inclination **Δ** of column, relative to a vertical line through its centre at base level	$\|\Delta\| = \Sigma h/(300 \cdot \sqrt{n})$	$\|\Delta\| = \Sigma h/(500 \cdot \sqrt{n})$
	Inclination Δ_a of a column relative to a vertical line through its centre at the next lower level as well as straightness deviation between the levels	$\Delta_a = \pm h/500$	$\Delta_a = \pm h/1000$
Straightness of a continuous column between adjacent storey levels: 	Location of the column in plan, relative to a straight line between position points at adjacent storey levels	$\Delta = \pm h/750$	$\Delta = \pm h/1000$

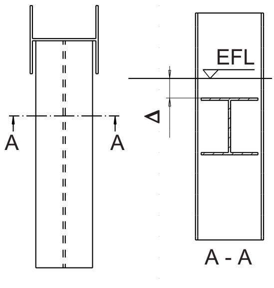
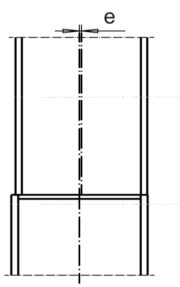
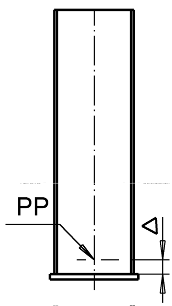
(continues)

Table 8.25 (*continued*) Functional erection tolerances – Columns of single storey and multi-storey buildings [1]

Criterion	Parameter	Permitted deviation Δ	
		Class 1	**Class 2**
Straightness of a spliced column, between adjacent storey levels: 	Location of the column in plan at the splice, relative to a straight line beween position points at adjacent storey levels	Δ = ±s/750 with s ≤h/2	Δ = ±s/1000 with s ≤h/2
Height: 	Overall height, relative to the base level: h ≤ 20 m 20 m < h < 100 m h ≥ 100 m	 Δ = ±20 mm Δ = ±0,5·(h + 20) mm Δ = ±0,2·(h + 200) mm (h in metres)	 Δ = ±10 mm Δ = ±0,25·(h + 20) mm Δ = ±0,1·(h + 200) mm (h in metres)
Storey height and slope of the horizontal beam: 	Height relative to the adjacent levels Slope of the horizontal beam (height relative to the other end of the beam)	Δ = ±10 mm Δ = ±L/500 but |Δ| ≤ 10 mm	Δ = ±5 mm Δ = ±L/1000 but |Δ| ≤ 5 mm
Relative levels: 	Levels of adjacent beams	Δ = ±10 mm	Δ = ±5 mm

(continues)

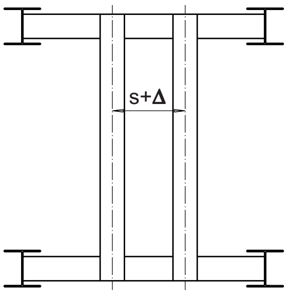
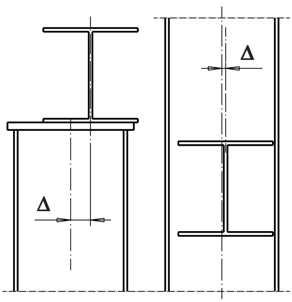
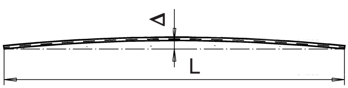
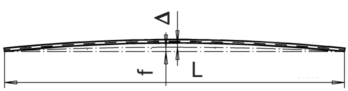
Table 8.25 (*continued*) Functional erection tolerances – Columns of single storey and multi-storey buildings [1]

Criterion	Parameter	Permitted deviation Δ	
		Class 1	**Class 2**
Connection levels: 	Level of the beam at a beam-to-column connection, measured relative to the established floor level (EFL)	$\Delta = \pm 10$ mm	$\Delta = \pm 5$ mm
Column splice 	Non-intended eccentricity e, (about either axis)	5 mm	3 mm
Column base: 	Level of bottom of column shaft, relative to specified level of its position point (PP).	$\Delta = \pm 5$ mm	$\Delta = \pm 5$ mm

Note 1: The levels of beams should be measured relative to the established floor level (the best-fit to the specified floor levels, adjusted for tolerances in the column lengths).

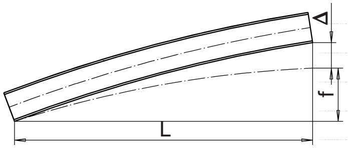
Note 2: Notations such as $\Delta = \pm L/500$ but $|\Delta| \leq 5$ mm mean that $|\Delta|$ is the smaller of $L/500$ and 5 mm.

Table 8.26 Functional erection tolerances – Beams in buildings [1].

Criterion	Parameter	Permitted deviation Δ	
		Class 1	**Class 2**
Spacing: 	Deviation Δ from distance between adjacent erected beams measured at each end	Δ = ±10 mm	Δ = ±5 mm
Location at columns: 	Deviation Δ from location of a beam-to-column connection, measured relative to the column	Δ = ±5 mm	Δ = ±3 mm
Straightness in plan 	Deviation Δ from straightness of an erected beam or cantilever of length *L*	Δ = ±*L*/500	Δ = ±*L*/1000
Camber: 	Deviation Δ at mid span from intended camber *f* of an erected beam or lattice component of length *L*	Δ = ±*L*/300	Δ = ±*L*/500

(continues)

Table 8.26 Functional erection tolerances – Beams in buildings [1] *(continued)*

Criterion	Parameter	Permitted deviation Δ	
		Class 1	Class 2
Pre-set of cantilevered part: 	Deviation Δ from intended pre-set at end of an erected cantilever of length *L*	$\Delta = \pm L/200$	$\Delta = \pm L/300$

8.7 Assembly of the truss in the workshop

In lattice assembly, an important factor is the number of lattices to be fabricated. With large series, it is worthwhile spending more time in the design and preparation of the assembly frame (or jig). Also the accuracy requirements must be taken into account when designing the jig. In regard to fabrication costs, it is important to be able to store the hollow sections close to the assembly site, since this saves time during transfer.

The jig must be made of sufficiently strong elements so that thermal and mechanical stresses generated during the assembly shall not distort it. The distances between lattice member supports shall be sufficiently short to avoid deformations in the lattice due to welding. In the jig, it is advisable to join the flange plates of the lattice to the plates welded to the assembly frame with bolts (Figure 8.21).

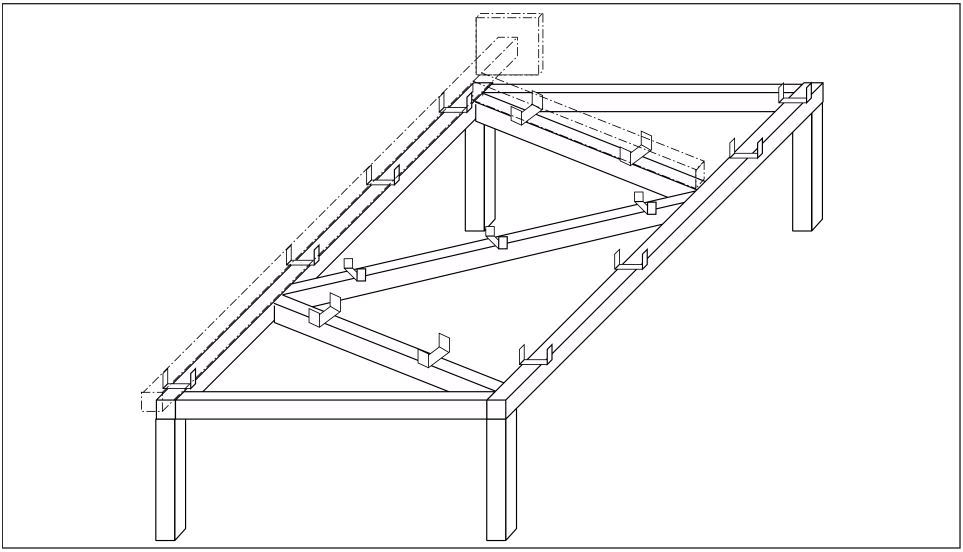


Figure 8.21 Assembly frame for a lattice

To speed up the shop fabrication phase, the members in the jig can be tack welded together, and the final welding can be done at another worksite. In such a case, attention must be paid to the firmness of the tack welding to avoid the generation of high deformations during the final welding.

In gap joints, attention must be paid to fabricate the gap as specified in the drawings and to meet the geometric tolerances given in Table 8.20. In overlap joints, the larger brace member that is overlapped shall always be welded first.

8.8 Fire protection

It is not always an advantage to prepare the fire protection in the workshop. Most fire protection materials will be damaged during transport and must then be repaired on-site. The fire protection method best suited for workshop fabrication is the fire protection painting (intumescent coating). Even then, sufficient care must be taken during transport to keep the damage to the painted surface to a minimum. The damaged spots must be repaired on-site, and it may, in extreme cases, be necessary to apply a new coat of paint to ensure the required fire resistance period. Guidelines and execution practises for fire protection painting have been presented in more details in [9].

When using concrete-filled hollow sections, the concrete infill is possible to pour already at workshop, but this increases the transport weight of the hollow sections and makes the erection on-site more difficult. Thus, pouring of the concrete infill is usually not done until on-site.

8.9 Erection

The on-site erection of hollow section structures does not differ from erection of other steel structures. However, the greater torsional stiffness of hollow sections and the good bending stiffness about both axes make them easier to lift and erect. Thus, the need for temporary lateral supports during erection is lesser on hollow section structures. Hollow section structures are also less susceptical to wind effects during erection than the open sections.

Erection of a hollow section structure shall be made according to the erection method statement. The erection method statement must take into account the site conditions in regard to the access routes for the transportation vehicles and lifting equipment. The very first frame or column shall be supported in the horizontal direction using bracing elements. After this, the erection proceeds with the next frame or column which shall be connected to the already supported structure. The erection blocks (for example the lattices) are joined with bolted connections.

Hollow section structures are light-weight. Thus, even large erection blocks can be easily lifted and installed on the site. It is advisable not to tighten the bolts to their final torque before checking the correct positions of the components of the structure. After this, the bolts are tightened to the final torque and locked, if necessary (clause 8.4.2). The assembly of large lattice structures (especially space frame trusses) is usually most easily executed at ground level, after which the finished lattice is erected to its final position.

An erection-friendly lattice is planned such that the cranes are able to be removed as quickly as possible after the lifting, since the crane-cost may be a considerable share of the total costs of the site. The blocks to be lifted should be planned such that their both ends are immediately supported after hoisting, making it possible to get the crane free for other tasks.

8.10 References

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Execution of steel structures and aluminium structures.
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Contains also amendment A1:2011.
Finnish Standards Association SFS. 210 pages.

[2] CIDECT. 1995. Design guide for structural hollow sections in mechanical applications. CIDECT Design Guide No 6. Verlag TÜV Rheinland GmbH. 157 pages.

[3] CIDECT. 1998. Design guide for fabrication, assembly and erection of hollow section structures. CIDECT Design Guide No 7.
Verlag TÜV Rheinland GmbH. 171 pages.

[4] CIDECT. 1996. Hollofast and hollobolt system for hollow section connections.
Report 6G-14(A)/96.

[5] Jonsson B. 2007. A proposal for new weld class system.
Proceedings of Fatigue Design 2007, Senlis France 2007. 9 pages.

[6] SFS-EN ISO 5817:2006. (EN ISO 5817:2003)
Welding. Fusion-welded joints in steel, nickel, titanium and their alloys (beam welding excluded). Quality levels for imperfections.
Finnish Standards Association SFS. 64 pages.

[7] Hitsaajan opas. 2003. Rautaruukki Steel. 112 pages.

[8] Hirsimäki, H. 1996. Hitsaussaumojen laadunvarmistus.
Teräsrakenteiden pääsuunnittelijan kurssi 16.-18.10.1996. Kurssiaineisto.
Teräsrakenneyhdistys ry.

[9] Teräsrakenteiden palosuojamaalaus. 2007.
Teräsrakenneyhdistys ry. 18 pages.

9. SURFACE TREATMENTS

9.1 General

Steel structures need to be protected against corrosion. Corrosion starts when the relative humidity in the air exceeds 60 %. Part EN 1993-1-1 of Eurocode does not require corrosion protection for internal building structures, if the relative humidity does not exceed 80 % [1,2,3]. In corrosion process the energy stored in the metal in steel fabrication is released and the metal tends to return to its natural stable state. The corrosion of steel can be prevented with suitable surface treatments. Anti-corrosive painting and hot-dip galvanizing are the most common protection methods of steel structures exposed to atmospheric corrosion.

The structures shall be designed such that places liable to corrosion are not formed.

With regard to surface treatment, in some cases the purchaser may have extra requirements beyond EN 1090-2, which may have an impact also to the structural design. Neglecting them may cause a need for expensive repairing work for the project.

9.2 Corrosivity categories

The atmospheric environments are classified into six atmospheric corrosivity categories according to Table 9.1. In addition, structures immersed in water or buried in soil are divided into three corrosivity categories (Im1-Im3).

Table 9.1 Atmospheric corrosivity categories according to EN ISO 12944-2 [5]

Corrosivity category	Mass loss per unit surface / thickness loss (first year of exposure)				Description of typical environment (informative only)	
	Low-carbon steel		Zinc		Exterior	Interior
	Mass loss (g/m^2)	Thickness loss (μm)	Mass loss (g/m^2)	Thickness loss (μm)		
C1 Very low	≤10	≤1,3	≤0,7	≤0,1		Heated buildings with clean atmospheres, e.g. offices, shops, schools, hotels.
C2 Low	>10-200	>1,3-25	>0,7-5	>0,1-0,7	Rural areas with low level of air pollution	Unheated buildings where condensation may occur, e.g. depots, sports halls.
C3 Medium	>200-400	>25-50	>5-15	>0,7-2,1	Urban and industrial areas with moderate sulfur dioxide pollution. Coastal areas with low salinity.	Production rooms with high humidity and some air pollution,e.g. food-processing plants, laundries, breweries, dairies.

(continues)

Table 9.1 Atmospheric corrosivity categories according to EN ISO 12944-2 [5] *(continued)*

Corrosivity category	Mass loss per unit surface / thickness loss (first year of exposure)				Description of typical environment (informative only)	
	Low-carbon steel		Zinc		Exterior	Interior
	Mass loss (g/m^2)	Thickness loss (μm)	Mass loss (g/m^2)	Thickness loss (μm)		
C4 High	>400-650	>50-80	>15-30	>2,1-4,2	Industrial areas and coastal areas with moderate salinity.	Chemical plants, swimming pools, coastal ship- and boatyards.
C5-I Very high (industrial)	>650-1500	>80-200	>30-60	>4,2-8,4	Industrial areas with high humidity and aggressive atmosphere.	Buildings or areas with permanent moisture and with high level of pollution.
C5-M Very high (marine)	>650-1500	>80-200	>30-60	>4,2-8,4	Coastal and offshore areas with high salinity.	Buildings or areas with permanent moisture and with high level of pollution.
In coastal areas in hot, humid zones, the mass or thickness losses can exceed the limits of category C5-M. Special attention must be paid to the protective paint system.						

9.3 Surface preparation

Structural hollow sections are delivered from the factory according to agreement either unprotected (delivery state ‘dry’) or with light protective oil coating. Cleaning the surface before painting is then essential for successful surface treatment. The cleaning method selected depends on the amount and quality of the impurity, as well as the shape and size of the component. If needed, the surface of steel may be roughened to provide better adhesion for the coating.

The surfaces shall be prepared in accordance with EN ISO 12944-4. The most common preparation grades are presented in Table 9.2. Shot blasting is normally used in the workshop as the surface preparation method. The most common preparation grade is Sa 2$^1/_2$. In case of hot-dip galvanizing, acid pickling treatment is carried out prior to galvanizing. Wire brushing is used on-site, if on-site welding is necessary. After welding, the surface shall be wire brushed and patch painted.

If overpainting of hot-dip galvanized structure will be carried out, the cleaning of the surface requires particular attention: the surfaces shall be cleaned (removal of dust and grease) and possibly treated with a suitable etch primer or sweep-blasting according to EN ISO 12944-4 to surface roughness ‘fine’ in accordance with EN ISO 8503-2. The pre-treatment shall be inspected before subsequent painting.

The preparation grade of welds, edges and other areas with surface imperfectionsoatings is specified as P-classes according to EN ISO 8501-3. If the expected life of the corrosion protection and corrosivity category are specified, the P-class of the surface shall be in accordance with Table 9.3. Unless otherwise specified, P1 shall apply for execution classes EXC2 - EXC4 [4].

Table 9.2 Preparation grades for steel surfaces according to EN ISO 12944-4 [6]

Preparation grade a)	Preparation method	Essential features of prepared surfaces
Sa 2	Blast-cleaning using the method specified in ISO 8504-2	Most of the mill scale, rust, paint coatings and foreign matter are removed. Any residual contamination shall be firmly adhering.
Sa $2^1/_2$		Mill scale, rust, paint coatings and foreign matter are removed. Any remaining traces of contamination shall show only as slight stains in the form of spots or stripes.
Sa 3		Mill scale, rust, paint coatings and foreign matter are removed. The surface shall have a uniform metallic colour.
St 2	Hand- or power-tool cleaning (e.g. wire brushing). More information in ISO 8504-3	Poorly adhering mill scale, rust, paint coatings and foreign matter are removed.
St 3		Poorly adhering mill scale, rust, paint coatings and foreign matter are removed. The surface is treated clearly in more careful manner than for St 2 to give a metallic sheen arising from the metal substrate.
Be	Acid pickling	Mill scale, rust and residues of paint coatings are removed completely. Paint coatings shall be removed prior to acid pickling by suitable means.
a) Key to symbols used: Sa = Blast-cleaning (ISO 8501-1) St = hand-tool or power-tool cleaning (ISO 8501-1) Be = acid pickling		

Table 9.3 Preparation grade of welds, edges and other areas with surface imperfection-soatings according to EN ISO 8501-3 due to the expected life of the corrosion protection [4]

Expected life of the corrosion protection a)	Atmospheric corrosivity category b)	Preparation grade
> 15 years	C1	P1
	C2...C3	P2
	Over C3	P2 or P3 as specified
5...15 years	C1...C3	P1
	Over C3	P2
< 5 years	C1...C4	P1
	C5...lm	P2
a,b) Expected life of the corrosion protection and corrosivity category are referenced in EN ISO 12944 and EN ISO 14713-1 as relevant		

9.4 Anti-corrosive painting

Anti-corrosive painting is the most common corrosion protection method for steel structures. Paint coating system consists of primer, intermediate coat and top coat. When choosing the paint system, it should be considered not only the immediate painting costs, but also the maintenance costs during the whole lifetime of the structure, which depend on the environmental stresses applied to the structure (climate, corrosive substances, mechanical stress).

The metal surfaces to be painted should be as smooth and round-cornered as possible. Hollow sections apply in this respect excellently also for spray painting, since they are by nature round-cornered. Weld spatters and other irregularities must be removed before painting. Also the welds should be as smooth-surfaced as possible to avoid spots remaining non-painted. Joints should be designed such that the structure can be painted all over. When painting with a brush, the space between the splice plates must be at least as wide as the brush. Threaded parts shall not be painted, so they need to be protected for the time of painting. The corrosion resistance of bolts and nuts must be at least as high as that of the fastened materials.

Painting of hollow sections internally is not possible in practice. On the other hand, in normal circumstances the inside corrosion of hollow sections is rather small. According to [9], there is no risk to inside corrosion if the hollow section is closed at both ends. It should be ensured that the rain water cannot get inside the hollow section. However, in practice this is not always possible. For example, water can get inside the hollow section even through the holes made by self-drilling screws, no matter that they might be supposed water-tight due to the method they are executed. Rain water, or melt water from snow, can also get inside the hollow section for instance during the erection phase, when the skeleton is still exposed to rain. By a longer time period, condensing water may be produced inside the hollow section as a consequence of temperature fluctuations. **Due to above reasons, it is very important to provide hollow section structures with draining holes for water, especially if there is a risk for the water to get frozen inside the hollow section structure.**

Paint systems applicable in different atmospheric corrosivity categories as well as their expected service life have been presented in EN ISO 12944-5 [8]. To select a suitable paint system, tables provided by the paint producers can be utilised. Properties of different paints according to the former revision [7] of EN ISO 12944-5 are presented in Table 9.4. In the new revision of the standard the table is removed.

Paint systems used for steel structures in Finland include, for example, alkyd, epoxy, polyurethane, acrylic, zinc epoxy and zinc silicate systems. In steel skeletons, alkyd and epoxy systems are the most commonly used. Information about paint systems is available in the system tables provided by the paint producers.

Table 9.4 Properties of paints [7]

	AY	**AK**	**PUR,** aliphatic	**EP**	**ESI**
Gloss retention	3	2	3	1	0
Colour retention	3	2	3	1	0
Resistance to chemicals: water immersion rain/condensation solvents splashes of solvents acids splashes of acids alkalis splashes of alkalis	 1 3 1 1 1 2 3 2	 1 2 1 2 1 1 1 1	 2 / 1 [a)] 3 1 3 / 2 [a)] 1 2 / 1 [a)] 1 3	 3 3 2 3 1 2 3 3	 2 3 3 3 1 1 1 1
Resistance to dry heat: up to 70 ºC 70 ºC...120 ºC 120 ºC...150 ºC over 150 ºC	 3 3 / 2 [a)] 1 1	 3 2 1 1	 3 3 2 1	 3 3 2 1	 3 3 3 3
Physical properties: Abrasion resistance Impact resistance Flexibility Hardness	 1 2 2 2	 2 1 1 3	 2 3 3 2	 3 2 3 / 2 [a)] 3 / 2 [a)]	 3 1 1 3
AY = Acrylic AK = Alcyd PUR = Polyurethane EP = Epoxy ESI = Ethyl zinc silicate			3 = Excellent 2 = Good 1 = Poor 0 = Not relevant		
a) The ratings may vary within the same paint type depending on its composition. Both symbols may apply.					

The paint system is designated in EN ISO 12944-5 [8] by specifying the number of the paint system after the standard, eg. EN ISO 12944-5/A1.20. The binder used in the primer and top coat, if that may vary, shall be given in the identifier, eg. EN ISO 12944-5/A1.20-EP/PUR. In the former revision of EN ISO 12944-5, the marking was supplemented in Finland in such a way that the paint type, nominal thickness of the total coating, number of the paint layers, substrate, and preparation grade of the substrate, were marked in paranthesis as follows [7]:

e.g. **SFS-EN ISO 12944-5/S2.10-AK/AY (AKAY 160/3 - FeSa $2\frac{1}{2}$)**

designation in the standard
paint system
priming and top coat(s) binder
national supplement

Due to its informativeness, it is recommendable to still use the above mentioned old way of marking according to [7].

If the paint system is none of those specified in EN ISO 12944-5, a complete specification shall be given (eg. surface preparation, type of coating, number of layers, etc.) in the way presented in [8].

As the method of painting, normally spray painting is used. A brush or a roller are normally used for patch painting only. For this reason, the structures should be designed such that they are paintable by using merely spray painting as the painting method.

Shop primer is used as a temporary corrosion protection for the blast-cleaned surfaces during fabrication, transport, erection and storage. The paint is applied by spraying an even layer with a film thickness of 15-30 μm. The shop primer must not adversely affect the fabrication processes, such as welding or thermal cutting. Normally the shop primer is not part of the paint system and it may be necessary to remove it [8].

If the corrosivity category is C3 or higher and the expected life of the corrosion protection is more than five years, the edges shall be rounded or chamfered in accordance with EN ISO 12944-3, and the edges shall be protected by an extra stripe of coating, reaching to 25 mm on both sides of the edge. The thickness of the extra stripe shall be applied to a nominal thickness appropriate to the coating system [4].

The success of the painting work is highly dependant on the preparation of the surface to be painted as well as on the painting conditions (temperature, relative humidity in the air, purity of air). The painting conditions have to fulfill the requirements set by the paint producer for the paint system in question. The temperature of the surfaces to be painted shall be at least 3 ºC above the dew point, unless otherwise specified by the paint producer [4].

When planning the painting to be done, it is advisable to check the VOC requirements (Volatile Organic Compound) of the country in question, as these may have influence on the paint system to be chosen.

9.5 Hot-dip galvanizing

The protective characteristics of zinc coating are based on the fact that zinc as a more electronegative metal corrodes prior to steel. Effectiveness of the protection is directly proportional to the thickness of the zinc coating. On the other hand, a thick zinc coating is sensitive to mechanical damage. The thickness of the coating is normally 50-100 μm, on high-silicon steels even 250 μm.

As an additional benefit, zinc coating protects by cathodic means (i.e. zinc corrodes instead of steel) also the revealed steel surface in scratches and in cut edges.

The most common and most effective zinc coating method is hot-dip galvanizing. The method is covered in EN ISO 1461 [10] and EN ISO 14713 [11,12]. The component to be galvanized is dipped into a zinc bath, and the zinc adheres to the surface of the steel. The component has to be designed such a way that zinc is able to reach every surface in it. Moreover, the size of the zinc bathtub limits the maximum size of the component to be galvanized. The maximum dimensions have to be checked with the fabricating workshop.

In hot-dip galvanizing, the form and dimensions of the steel structure affect the way the dipping will be executed, and by that way also the final outcome. The steel grade has no direct influence to the visual appearance or thickness of the zinc coating.

In regard with steel structures, there are two factors that have the most essential impact to the **visual appearance of the zinc surface, thickness of the coating and the zinc's adherence to the steel surface**:

- Si+P content of the steel to be used and
- dipping time (see Chapter 1, Figure 1.4).

Low-silicon steel (Si + P ≤ 0,04 %) (corresponds to EN 10025-2, class 1)**:**
If the visual appearance of the zinc-coated steel structure is important, or if the structure will be painted after hot-dip galvanizing, it is recommended to select a low-silicon steel, where Si + P ≤ 0,04 %. In this case the zinc coating will be shiny, even-coloured and well-adhesive. The thickness of the zinc coating is typically < 90 µm. If a larger coating thickness is needed, a medium-silicon or a high-silicon steel should be chosen.

Medium-silicon steel (Si = 0,15 - 0,25 %) (corresponds to EN 10025-2, class 3)**:**
When corrosion resistance and hence the thickness of the zinc coating is a decisive factor, it is recommended to select a medium-silicon steel (Si = 0,15 - 0,25 %). With medium-silicon steels, the thickness of the zinc coating is thicker than with low-silicon steels, but the adhesion of the coating is weaker and there may be colour variations and darker regions in the coating. The structures are even now paintable on the zinc coating, but a good surface quality is more difficult to achieve than with low-silicon steels. In medium-silicon steels, the small amount of phosphorus (P) has no influence on the hot-dip galvanizing.

High-silicon steel (Si = 0,25 - 0,35 %) (corresponds to EN 10025-2, class 2)**:**
If a specially thick zinc coating is wanted (eg. immersed structures), a high-silicon steel shall be chosen (Si = 0,25 - 0,35 %). The zinc coatings will be thick, coarse, and brittle. Furthermore, they will develop a dark appearance over time quickly. A good surface preparation level prior to painting as well as a good-quality outcome of painting are difficult to achieve.

Since the silicon content of steel has a high impact on the suitability for hot-dip galvanizing, this shall be taken into account already when placing an order for the steel material. In Table 9.5, the classes for the suitability for hot-dip galvanizing are presented in accordance with the flat steel standard EN 10025-2, based on the silicon and phosphorus contents of steel. In Table 9.6, based on the composition of the steel material nearly equally, the typical characteristics of the finished zinc coating are presented according to the hot-dip galvanizing standard EN 14713-2.

Table 9.5 Classes for the suitability for hot-dip galvanizing according to the flat steel standard EN 10025-2 [13]

Classes	Elements (% by mass)		
	Si	Si + 2,5 P	P
Class 1	≤ 0,030	≤ 0,090	–
Class 2 a)	≤ 0,35	–	–
Class 3	0,14 ≤ Si ≤ 0,25	–	≤ 0,035
a) Class 2 applies only for special zinc alloys.			

Table 9.6 Properties of zinc coating depending on the steel composition according to the hot-dip galvanizing standard EN ISO 14713-2 [12]

Category	Typical levels of reactive elements	Additional information	Typical coating characteristics
A	Si ≤ 0,04 % and P < 0,02 %	See Note 1.	Coating has a shiny appearance with a finer texture. Coating structure includes the outer zinc layer.
B	0,14 % ≤ Si ≤ 0,25 %	Iron/zink alloy may extend through to the coating surface. Coating thickness increases with increasing silicon content. Other elements may also affect steel reactivity. Especially phosphorus levels greater than 0,035 % will increase the reactivity.	
C	0,04 % < Si ≤ 0,14 %	Too large coating thicknesses may be formed.	Coating has a darker appearance with a coarser texture. Iron/zinc alloys dominate the coating structure and often extend to the coating surface, with reduced resistance to handling damage.
D	Si > 0,25 %	Coating thickness increases with increasing silicon content.	

Note 1:
Steels having the chemical composition within (Si + 2,5 P) ≤ 0,09 % have also these characteristics. For cold rolled steels, these characteristics apply when the composition of steel is within (Si + 2,5 P) ≤ 0,04 %.

Note 2:
The presence of alloying elements (eg. nickel) in the zinc bath can have a significant effect on the coating characteristics presented in this table. This table does not provide relevant guidance for high-temperature galvanizing (i.e. dipping in zinc melt at temperature 530....560 °C).
(Comment by the author: In normal hot-dip galvanizing the temperature of the zinc bath is about 460 °C.)

Note 3:
The steel compositions in this table will vary under the influence of other factors and the boundaries of each range will vary accordingly.

On SSAB's standard steel grade for structural hollow sections, SSAB Domex Tube Double Grade, the silicon content is guaranteed at Si 0,15 - 0,25 %. With an appropriate galvanizing procedure this silicon content enables a zinc coating thickness of over 100 μm (see Chapter 1, Figure 1.4), being sufficient for most applications. The silicon content of other steel grades depends on each case. On some steel grades, silicon and phosphorus content is Si + P ≤0,04 %. With an appropriate galvanizing procedure this silicon content enables a thin zinc coating (below 100 μm). Should there be specific requirements regarding silicon and phosphorus content, these shall be agreed when placing the inquiry and order.

SSAB Domex Tube structural hollow sections in grades S235 - S460 apply well to hot-dip galvanizing. Regarding hot-dip galvanizing of structural hollow sections of higher grades, it is recommendable to discuss first with the manufacturer.

Hot-dip galvanized hollow section structures must be designed such that zinc can flow freely into the hollow section and out of it. Closed spaces in the components are not permitted, since any residual moisture which may exist inside the component evaporates into steam during hot-dip galvanizing, and the component may explode. To avoid explosion, the component has to be provided with holes for the hot-dip galvanizing process, when necessary. The holes for the hot-dip galvanizing process also serve to ensure that the zinc is effectively applied over the entire surface of the steel to be protected and the excessive zinc runs out from the component. The latter is important because the price setting is often based on the weighted mass prior to and after the zinc coating. After hot-dip galvanizing, the holes must be checked to ensure that they are not blocked by excessive zinc. In this case it may occur that the hollow section could collect water, which by freezing can split the hollow section.

Related to the hot-dip galvanizing process, so-called LMAC-cracking (Liquid Metal Assisted Cracking) may occur in hot-dip galvanized structures. In addition to the micro structure and surface quality of the steel, factors affecting the cracking susceptibility are primarily the following:

- material thicknesses applied in the structure, and their mutual relations (t_{max} / t_{min})
- residual stresses in the structure (for example due to welding)
- stresses due to uneven temperature distribution in the structure during hot-dip galvanizing
- chemical composition of the zinc bath
- dipping time

The subject is presented in more details in [14,15].

Hot-dip galvanizing, and the pickling process before that, do not normally cause hydrogen embrittlement (hydrogen cracking) on structural steels. Possible hydrogen residuals of the hydrogen that is absorbed in the pickling process, will normally disappear in the hot zinc bath. However, if the hardness of the steel exceeds 340 HV (ultimate tensile strength $R_m > 1100$ MPa), special attention should be paid to minimize the amount of hydrogen which is absorbed during pickling [12].

Hot-dip galvanized steel surface may require painting, when [16]:

- the hot-dip galvanizing alone is not sufficient for corrosion protection
- the colour of zinc coating is not suited for the environment or
- the structure must have a specific colour.

The combination of anti-corrosive painting and hot-dip galvanizing is an effective corrosion protection, because the corrosion products of zinc under the paint coating fill in the cracks where the paint coating fails or cracks. The corrosion products of zinc do not remove the paint, since they do not require as much additional volume as rust [15]. The hot-dip galvanized surface is suitable for paint substrate, when the zinc-coated surface is carefully cleaned from corrosion products of zinc and from other impurities immediately prior to painting. A suitable cleaning method is sweep-blasting with non-metallic cleaning abrasives [8]. Paint coating systems suitable for hot-dip galvanized surface are presented in EN ISO 12944-5 [8].

9.6 Inspections of surface treatment

Cleanliness and roughness of prepared steel surfaces shall be checked in accordance with EN ISO 8501 and EN ISO 8503-2.

Each layer of the paint coating shall be checked in accordance with ISO 19840, or if zinc coating is used as a primary corrosion protection, the paint coating shall be checked in accordance with EN ISO 2808. Visual inspection shall be applied to check that the painting fulfills the requirements of EN ISO 12944-7.

Hot-dip galvanizing shall be checked according to EN ISO 1461. According to EN 1090-2, galvanized components shall always be checked due to the risk of Liquid Metal Assisted Cracking (LMAC), if not otherwise specified in the execution specification.

Reference areas to be used in different corrosivity categories shall be specified in the execution specification in accordance with EN ISO 12944-7 to establish the minimum acceptable standard for the work. Unless otherwise specified, reference areas shall be specified for corrosion protection systems in corrosivity categories C1 - C3 and Im1 - Im3.

The subject of the inspection is the number of films and thicknesses of the single films as well as other items related to the surface treatment. Reduction of costs through spreading of overly thick films is not profitable, because the inspector may detect it easily, and a new surface treatment may be required. Hence the aimed cost saving may finally turn to distinct extra costs.

9.7 References

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10. TRANSPORT AND STORAGE

During the structural design phase, the transport of structural components and sub-assemblies must also be taken into account. Large pre-assembled blocks make the erection quicker but may increase transport costs, especially if special-transport is needed. When planning special-transport, it is necessary to take into account the limits to the size of the blocks set by the bridges and roads at the proximity of the site. In international projects, it must be considered that national regulations concerning maximum weight and dimensions of the transports vary country by country. Road transport is the simplest mode for transferring material when the workshop and the site are located relatively close to each other.

In transport of hollow section structures, at least the following issues shall be considered [1]:

- the tarpaulins used to cover the load must be dry, clean, undamaged and sufficiently large
- hollow sections shall be placed on the platform of the vehicle so that they are not subjected to knocks, chafing or any other risk of damage
- when loading hollow sections on top of each other, the products with the thickest walls and the greatest length and weight shall be placed the lowermost
- heavy products which might cause damage to hollow sections must not be piled on them
- the platform must be clean, dry and even-bottomed
- **the load shall be tied so that it cannot shift during the transport**
- the load shall be tied with straps so that it does not touch the side or end posts of the platform
- the fastening straps or chains must not be tightened to the degree that they cause dents on the hollow sections; if necessary, the contact points shall be furnished with appropriate protection
- during transport, suitable skids must be used under the hollow sections

In storing of hollow sections, the following issues shall be considered [1]:

- the storage space must be clean, dry and properly ventilated
- in the store building, the entry of condensation water to the hollow sections must be prevented
- in the store, a sufficient amount of skids and dunnage below and within the stack must be used

10.1 Different modes of transport

In Finland, steel structures are transported from the workshop to the site normally by road transport. Because of the limitations of the road network, as an alternative to road transport, for example, railway transport can be used. In addition to road or railway transport, the structures can be transported abroad also by sea or air transport. However, air transport is expensive and it applies seldom for transporting large structures.

Steel structures can be so large, that for example in road transport it is necessary to use special-transport (i.e. oversized or overweight transport), even if the constructor aims to design the structures according to the dimensional and weight limits of a normal transport. The limits for the dimensions and weights of road and rail transports for normal and special-transport vary country by country.

10.2 Dimensional and weight limits for road transport; transport licences

A road transport may either stay within the limits generally permitted on the roads, or it may be a special-transport within non-licenced dimensional limits, or a special-transport (i.e. oversized/ overweight transport) based on a specific licence. The dimensional limits for a road transport, when using an articulated vehicle, are as follows (dimensions in loaded trip):

	YL (m)	EK (m)
Height:	4,2	4,4
Width:	2,55	4,0
Length:		
• lorry with a semi-trailer	16,5	30,0
• lorry with a trailer in Finland	25,25	27,0

YL = maximum dimensions commonly permitted on the road
EK = maximum non-licenced dimensions permitted on the road for an oversized transport (separate licence not required)

In special-transports, it shall be considered for example that when the width exceeds 3,5 m, an alerting vehicle is required in front of the transport, which increases clearly the transport costs. Thus, in structural design, in addition to resistance, manufacture and erection, also the subjects due to transport of the structure or its components shall be taken into account.

For a non-licenced special-transport, the load overhang in length at the rear end is not permitted to be more than 4 m for a lorry combined with a trailer, and 6 m for a lorry combined with a semi-trailer. For a special-transport exceeding the size of non-licenced special-transport, a specific licence may be granted for the public roads and the roads of communes (excluding the province of Åland) by the ELY-center at Pirkanmaa since beginning of the year 2010 for the present. For a private road, only the acceptance by the road owner is required. The police no longer grants the licences for special-transports.

All overweight transports need a special-transport licence. The licence may be granted, if the load consists of one object only. The most important weight limits commonly permitted on roads are:

• single axle	10 to
• single drive shaft	11,5 to
• two-axle bogie (depending on the wheelbase)	11,5, 16 or 18 to
• three-axle bogie (depending on the wheelbase)	21 or 24 to
• vehicle on two axles	18 to
• vehicle on three axles	25 or 26 to
• vehicle on four axles	31 or 32 to
• vehicle on five axles	38 to
• articulated vehicle, lorry with a semi-trailer	48 to
• articulated vehicle on four axles	36 to
• articulated vehicle on five axles	44 to
• articulated vehicle on six axles	53 to
• articulated vehicle on seven or more axles	60 to

The weights of articulated vehicles other than those registered or taken in use in an EU or ETA country are at the most 40 tonnes. Detailed information can be found in the Decree on the Use of Vehicles on the Road [2].

10.3 Railway transport

The freight wagons shall be loaded complying the guidelines given in the Guide on Goods Loading (TKO), published by VR Transpoint (formerly VR Cargo) and placed on web site [www.transpoint.fi]. If the guide cannot be applied, the transport will be considered as special-transport, which requires a transport licence. The licence is drawn up by the domestic freight office of VR Transpoint [3].

The railway transport is covered in more details in [3], which contains a presentation of the freight wagons available, as well as their loading areas and maximum loads. The type of the freight wagon to be chosen for the transport depends on the form and weight of the structure.

10.4 References

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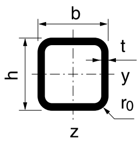
11. ANNEXES

Annex 11.1 Cross-sectional properties and resistance values for steel grades S355J2H and S420MH

Table	Steel grade		Shape
11.1.1	S355J2H		Square
11.1.2	S355J2H		Rectangular
11.1.3	S355J2H		Circular longitudinally welded
11.1.4	S355J2H		Circular spirally welded
11.1.5		S420MH	Square
11.1.6		S420MH	Rectangular
11.1.7		S420MH	Circular longitudinally welded
11.1.8		S420MH	Circular spirally welded

Table 11.1.1 Cross-sectional properties and resistance values for square hollow sections of steel grade S355J2H (f_y = 355 N/mm^2)



M = weight
A = cross-section area
A_u = external area
A_m/V = cross-section factor in fire design
I_t = St. Venant torsional constant
W_t = torsional section modulus

I = second moment of area
W_{el} = elastic section modulus
W_{pl} = plastic section modulus (shall be used only for CL1...2)
i = radius of gyration

CL = cross-section Class at uniform compression
$N_{c.Rd}$ = design compression resistance (without buckling)
$M_{c.Rd}$ = design bending resistance (respective to cross-section Class)
$V_{pl.Rd}$ = design plastic shear resistance (without shear buckling)

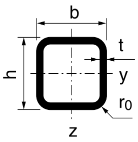
r_0 = 2,0 x t when t ≤ 6,0 mm
r_0 = 2,5 x t when 6,0 mm < t ≤ 10,0 mm
r_0 = 3,0 x t when t > 10,0 mm

The calculated resistance values are design values (see Chapter 2) based on recommended partial safety factor values **γ_{M0} = 1,0** and **γ_{M1} = 1,0 (for Class 4 circular hollow sections γ_{M1} = 1,1)** as given in Eurocode 3 (EN 1993). Partial safety factor values may differ in each country. National values must be checked from the National Annex of the relevant country.

h mm	b mm	t mm	M kg/m	A mm^2 x 10^2	A_u m^2/m	A_m/V 1/m	I_t mm^4 x 10^4	W_t mm^3 x 10^3	I mm^4 x 10^4	W_{el} mm^3 x 10^3	W_{pl} mm^3 x 10^3	i mm x 10	CL	$N_{c.Rd}$ kN	$M_{c.Rd}$ kNm	$V_{pl.Rd}$ kN
25	25	2	1,36	1,74	0,093	536	2,53	1,80	1,48	1,19	1,47	0,92	1	61,66	0,52	17,80
25	25	2,5	1,64	2,09	0,091	438	2,97	2,07	1,69	1,35	1,71	0,90	1	74,16	0,61	21,41
25	25	3	1,89	2,41	0,090	372	3,33	2,27	1,84	1,47	1,91	0,87	1	85,49	0,68	24,68
30	30	2	1,68	2,14	0,113	529	4,54	2,75	2,72	1,82	2,21	1,13	1	75,86	0,78	21,90
30	30	2,5	2,03	2,59	0,111	430	5,40	3,20	3,16	2,10	2,61	1,10	1	91,91	0,93	26,53
30	30	3	2,36	3,01	0,110	365	6,15	3,58	3,50	2,34	2,96	1,08	1	106,8	1,05	30,83
40	40	2	2,31	2,94	0,153	521	11,28	5,23	6,94	3,47	4,13	1,54	1	104,3	1,47	30,10
40	40	2,5	2,82	3,59	0,151	422	13,61	6,21	8,22	4,11	4,97	1,51	1	127,4	1,76	36,78
40	40	3	3,30	4,21	0,150	356	15,75	7,07	9,32	4,66	5,72	1,49	1	149,4	2,03	43,13
40	40	4	4,20	5,35	0,146	273	19,44	8,48	11,07	5,54	7,01	1,44	1	189,9	2,49	54,81
50	50	2	2,93	3,74	0,193	517	22,63	8,51	14,15	5,66	6,66	1,95	1	132,7	2,37	38,30
50	50	2,5	3,60	4,59	0,191	417	27,53	10,22	16,94	6,78	8,07	1,92	1	162,9	2,87	47,03
50	50	3	4,25	5,41	0,190	351	32,13	11,76	19,47	7,79	9,39	1,90	1	192,0	3,33	55,42
50	50	4	5,45	6,95	0,186	268	40,42	14,43	23,74	9,49	11,73	1,85	1	246,7	4,16	71,20
50	50	5	6,56	8,36	0,183	219	47,46	16,56	27,04	10,82	13,70	1,80	1	296,6	4,86	85,63
60	60	2	3,56	4,54	0,233	514	39,79	12,59	25,14	8,38	9,79	2,35	2	161,1	3,48	46,49
60	60	2,5	4,39	5,59	0,231	414	48,66	15,22	30,34	10,11	11,93	2,33	1	198,4	4,24	57,28
60	60	3	5,19	6,61	0,230	348	57,09	17,65	35,13	11,71	13,95	2,31	1	234,6	4,95	67,72
60	60	4	6,71	8,55	0,226	265	72,64	21,97	43,55	14,52	17,64	2,26	1	303,5	6,26	87,60
60	60	5	8,13	10,36	0,223	215	86,42	25,61	50,49	16,83	20,88	2,21	1	367,6	7,41	106,1
70	70	2,5	5,17	6,59	0,271	412	78,49	21,22	49,41	14,12	16,54	2,74	1	233,9	5,87	67,52
70	70	3	6,13	7,81	0,270	345	92,42	24,74	57,53	16,44	19,42	2,71	1	277,2	6,89	80,02
70	70	4	7,97	10,15	0,266	262	118,5	31,11	72,12	20,61	24,76	2,67	1	360,3	8,79	104,0
70	70	5	9,70	12,36	0,263	213	142,2	36,65	84,63	24,18	29,56	2,62	1	438,6	10,49	126,6

h mm	b mm	t mm	M kg/m	A mm² x 10²	A_u m²/m	A_m/V 1/m	I_t mm⁴ x 10⁴	W_t mm³ x 10³	I mm⁴ x 10⁴	W_{el} mm³ x 10³	W_{pl} mm³ x 10³	i mm x 10	CL	$N_{c.Rd}$ kN	$M_{c.Rd}$ kNm	$V_{pl.Rd}$ kN
80	80	2,5	5,96	7,59	0,311	410	118,5	28,22	75,15	18,79	21,90	3,15	2	269,4	7,77	77,77
80	80	3	7,07	9,01	0,310	344	139,9	33,02	87,84	21,96	25,78	3,12	1	319,8	9,15	92,32
80	80	4	9,22	11,75	0,306	261	180,4	41,84	111,0	27,76	33,07	3,07	1	417,1	11,74	120,4
80	80	5	11,3	14,36	0,303	211	217,8	49,68	131,4	32,86	39,74	3,03	1	509,6	14,11	147,1
80	80	6	13,2	16,83	0,299	178	252,1	56,59	149,2	37,29	45,79	2,98	1	597,6	16,25	172,5
90	90	2,5	6,74	8,59	0,351	409	170,3	36,23	108,6	24,12	28,00	3,56	3	304,9	8,56	88,02
90	90	3	8,01	10,21	0,350	343	201,4	42,51	127,3	28,29	33,04	3,53	2	362,4	11,73	104,6
90	90	4	10,5	13,35	0,346	259	260,8	54,17	161,9	35,98	42,58	3,48	1	473,9	15,12	136,8
90	90	5	12,8	16,36	0,343	210	316,3	64,70	192,9	42,87	51,41	3,43	1	580,6	18,25	167,6
90	90	6	15,1	19,23	0,339	176	367,8	74,16	220,5	49,00	59,54	3,39	1	682,8	21,14	197,1
100	100	3	8,96	11,41	0,390	342	278,7	53,19	177,0	35,41	41,21	3,94	2	405,0	14,63	116,9
100	100	4	11,7	14,95	0,386	258	362,0	68,10	226,4	45,27	53,30	3,89	1	530,7	18,92	153,2
100	100	5	14,4	18,36	0,383	209	440,5	81,72	271,1	54,22	64,59	3,84	1	651,6	22,93	188,1
100	100	6	17,0	21,63	0,379	175	514,2	94,12	311,5	62,29	75,10	3,79	1	768,0	26,66	221,7
100	100	7,1	19,4	24,65	0,370	150	589,2	105,6	340,1	68,03	83,59	3,71	1	875,2	29,67	252,6
100	100	8	21,4	27,24	0,366	134	644,5	114,2	365,9	73,19	91,05	3,67	1	967,1	32,32	279,2
100	100	10	25,6	32,57	0,357	110	749,8	130,1	411,1	82,22	105,3	3,55	1	1156	37,36	333,7
110	110	2,5	8,31	10,59	0,431	407	314,9	55,23	202,4	36,80	42,47	4,37	4	320,5	11,86	108,5
110	110	3	9,90	12,61	0,430	341	373,5	65,07	238,3	43,33	50,27	4,35	3	447,6	15,38	129,2
110	110	4	13,0	16,55	0,426	258	486,5	83,63	305,9	55,62	65,21	4,30	1	587,5	23,15	169,6
110	110	5	16,0	20,36	0,423	208	593,6	100,7	367,9	66,90	79,27	4,25	1	722,6	28,14	208,6
110	110	6	18,9	24,03	0,419	175	694,9	116,5	424,6	77,19	92,46	4,20	1	853,2	32,82	246,3
120	120	3	10,8	13,81	0,470	340	487,7	78,15	312,3	52,06	60,24	4,76	4	445,7	17,44	141,5
120	120	4	14,2	18,15	0,466	257	636,6	100,8	402,3	67,05	78,33	4,71	2	644,3	27,81	186,0
120	120	5	17,5	22,36	0,463	207	778,5	121,8	485,5	80,91	95,45	4,66	1	793,6	33,88	229,1
120	120	5,6	19,5	24,82	0,461	186	860,3	133,6	532,3	88,71	105,3	4,63	1	881,0	37,37	254,3
120	120	6	20,7	26,43	0,459	174	913,5	141,2	562,2	93,69	111,6	4,61	1	938,4	39,62	270,9
120	120	7,1	23,8	30,33	0,450	148	1056	160,1	623,5	103,9	125,7	4,53	1	1077	44,62	310,9
120	120	8	26,4	33,64	0,446	132	1163	174,6	676,9	112,8	137,8	4,49	1	1194	48,92	344,8
120	120	8,8	28,6	36,48	0,442	121	1252	186,5	719,9	120,0	147,9	4,44	1	1295	52,50	373,9
120	120	10	31,8	40,57	0,437	108	1376	202,5	776,8	129,5	161,8	4,38	1	1440	57,44	415,7
140	140	4	16,8	21,35	0,546	256	1023	139,8	651,6	93,09	108,2	5,52	3	757,9	33,05	218,8
140	140	5	20,7	26,36	0,543	206	1256	169,8	790,6	112,9	132,3	5,48	1	935,6	46,97	270,1
140	140	5,6	23,0	29,30	0,541	185	1391	186,9	869,6	124,2	146,3	5,45	1	1040	51,92	300,2
140	140	6	24,5	31,23	0,539	173	1479	197,9	920,4	131,5	155,3	5,43	1	1109	55,14	320,1
140	140	7,1	28,3	36,01	0,530	147	1719	226,0	1032	147,4	176,3	5,35	1	1278	62,59	369,1
140	140	8	31,4	40,04	0,526	131	1901	247,7	1127	161,0	194,2	5,30	1	1422	68,93	410,4
140	140	8,8	34,2	43,52	0,522	120	2055	265,8	1205	172,1	209,2	5,26	1	1545	74,28	446,0
140	140	10	38,1	48,57	0,517	106	2274	290,9	1312	187,4	230,4	5,20	1	1724	81,79	497,7

Table 11.1.1 Cross-sectional properties and resistance values for square hollow sections of steel grade S355J2H (f_y = 355 N/mm^2) *continued*



M = weight
A = cross-section area
A_u = external area
A_m/V = cross-section factor in fire design
I_t = St. Venant torsional constant
W_t = torsional section modulus

I = second moment of area
W_{el} = elastic section modulus
W_{pl} = plastic section modulus (shall be used only for CL1...2)
i = radius of gyration

CL = cross-section Class at uniform compression
$N_{c.Rd}$ = design compression resistance (without buckling)
$M_{c.Rd}$ = design bending resistance (respective to cross-section Class)
$V_{pl.Rd}$ = design plastic shear resistance (without shear buckling)

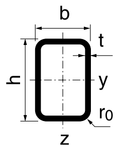
$r_0 = 2{,}0 \times t$ when $t \le 6{,}0$ mm
$r_0 = 2{,}5 \times t$ when $6{,}0$ mm $< t \le 10{,}0$ mm
$r_0 = 3{,}0 \times t$ when $t > 10{,}0$ mm

The calculated resistance values are design values (see Chapter 2) based on recommended partial safety factor values **γ_{M0} = 1,0** and **γ_{M1} = 1,0 (for Class 4 circular hollow sections γ_{M1} = 1,1)** as given in Eurocode 3 (EN 1993). Partial safety factor values may differ in each country. National values must be checked from the National Annex of the relevant country.

h mm	b mm	t mm	M kg/m	A mm^2 x 10^2	A_u m^2/m	A_m/V 1/m	I_t mm^4 x 10^4	W_t mm^3 x 10^3	I mm^4 x 10^4	W_{el} mm^3 x 10^3	W_{pl} mm^3 x 10^3	i mm x 10	CL	$N_{c.Rd}$ kN	$M_{c.Rd}$ kNm	$V_{pl.Rd}$ kN
150	150	4	18,0	22,95	0,586	255	1265	161,7	807,8	107,7	124,9	5,93	4	771,4	36,97	235,2
150	150	5	22,3	28,36	0,583	206	1554	196,8	982,1	130,9	153,0	5,89	2	1007	54,31	290,6
150	150	6	26,4	33,63	0,579	172	1833	229,8	1146	152,8	179,9	5,84	1	1194	63,86	344,7
150	150	7,1	30,5	38,85	0,570	147	2134	263,2	1290	172,0	204,8	5,76	1	1379	72,71	398,2
150	150	8	33,9	43,24	0,566	131	2364	289,0	1412	188,2	226,0	5,71	1	1535	80,21	443,2
150	150	8,8	36,9	47,04	0,562	120	2560	310,7	1513	201,7	243,9	5,67	1	1670	86,57	482,1
150	150	10	41,3	52,57	0,557	106	2839	341,0	1653	220,3	269,2	5,61	1	1866	95,55	538,7
150	150	12,5	48,7	62,04	0,536	86	3321	389,3	1817	242,3	305,6	5,41	1	2203	108,5	635,8
160	160	4	19,3	24,55	0,626	255	1542	185,3	987,2	123,4	142,8	6,34	4	792,3	41,33	251,6
160	160	5	23,8	30,36	0,623	205	1896	225,8	1202	150,3	175,2	6,29	2	1078	62,18	311,1
160	160	6	28,3	36,03	0,619	172	2239	264,2	1405	175,7	206,2	6,25	1	1279	73,22	369,3
160	160	7,1	32,7	41,69	0,610	146	2611	303,2	1587	198,4	235,5	6,17	1	1480	83,59	427,3
160	160	8	36,5	46,44	0,606	130	2897	333,6	1741	217,7	260,1	6,12	1	1649	92,35	475,9
160	160	8,8	39,7	50,56	0,602	119	3141	359,2	1870	233,7	281,1	6,08	1	1795	99,80	518,2
160	160	10	44,4	56,57	0,597	106	3490	395,1	2048	256,0	311,0	6,02	1	2008	110,4	579,7
160	160	12,5	52,6	67,04	0,576	86	4114	454,6	2275	284,4	355,7	5,83	1	2380	126,3	687,1
180	180	5	27,0	34,36	0,703	205	2724	289,8	1737	193,0	224,0	7,11	3	1220	68,51	352,1
180	180	6	32,1	40,83	0,699	171	3223	340,1	2037	226,3	264,4	7,06	2	1450	93,85	418,5
180	180	7,1	37,2	47,37	0,690	146	3768	391,7	2313	257,0	303,1	6,99	1	1682	107,6	485,5
180	180	8	41,5	52,84	0,686	130	4189	432,2	2546	282,9	335,7	6,94	1	1876	119,2	541,5
180	180	8,8	45,2	57,60	0,682	118	4551	466,6	2742	304,6	363,6	6,90	1	2045	129,1	590,3
180	180	10	50,7	64,57	0,677	105	5074	515,3	3017	335,2	403,5	6,84	1	2292	143,3	661,7
180	180	12,5	60,5	77,04	0,656	85	6050	600,1	3406	378,5	467,1	6,65	1	2735	165,8	789,5

h	b	t	M	A	A_u	A_m/V	I_t	W_t	I	W_{el}	W_{pl}	i	CL	$N_{c.Rd}$	$M_{c.Rd}$	$V_{pl.Rd}$
mm	mm	mm	kg/m	mm^2	m^2/m	1/m	mm^4	mm^3	mm^4	mm^3	mm^3	mm		kN	kNm	kN
				$x\ 10^2$			$x\ 10^4$	$x\ 10^3$	$x\ 10^4$	$x\ 10^3$	$x\ 10^3$	x 10				
200	200	5	30,1	38,36	0,783	204	3763	361,8	2410	241,0	278,9	7,93	4	1238	80,72	393,1
200	200	6	35,8	45,63	0,779	171	4459	425,5	2833	283,3	329,7	7,88	2	1620	117,0	467,6
200	200	7,1	41,6	53,05	0,770	145	5223	491,6	3232	323,2	379,3	7,81	1	1883	134,7	543,7
200	200	8	46,5	59,24	0,766	129	5815	543,6	3566	356,6	420,9	7,76	1	2103	149,4	607,1
200	200	8,8	50,7	64,64	0,762	118	6328	588,1	3850	385,0	456,6	7,72	1	2295	162,1	662,5
200	200	10	57,0	72,57	0,757	104	7072	651,5	4251	425,1	508,1	7,65	1	2576	180,4	743,7
200	200	12,5	68,3	87,04	0,736	85	8502	765,5	4859	485,9	593,5	7,47	1	3090	210,7	892,0
220	220	6	39,6	50,43	0,859	170	5976	520,6	3813	346,7	402,2	8,70	3	1790	123,1	516,8
220	220	7,1	46,1	58,73	0,850	145	7010	602,9	4367	397,0	464,0	8,62	2	2085	164,7	601,9
220	220	8	51,5	65,64	0,846	129	7815	667,9	4828	438,9	515,6	8,58	1	2330	183,1	672,7
220	220	8,8	56,3	71,68	0,842	117	8514	723,6	5221	474,7	560,2	8,53	1	2545	198,9	734,6
220	220	10	63,2	80,57	0,837	104	9533	803,6	5782	525,7	624,7	8,47	1	2860	221,8	825,6
220	220	12,5	76,2	97,04	0,816	84	11530	950,8	6674	606,7	734,9	8,29	1	3445	260,9	994,5
250	250	6	45,2	57,63	0,979	170	8843	681,2	5672	453,8	524,5	9,92	4	1811	149,6	590,6
250	250	7,1	52,8	67,25	0,970	144	10387	791,0	6523	521,8	607,1	9,85	3	2387	185,2	689,2
250	250	8	59,1	75,24	0,966	128	11598	878,2	7229	578,3	675,8	9,80	2	2671	239,9	771,1
250	250	8,8	64,6	82,24	0,962	117	12653	953,3	7835	626,8	735,3	9,76	1	2920	261,0	842,8
250	250	10	72,7	92,57	0,957	103	14197	1062	8707	696,5	822,0	9,70	1	3286	291,8	948,6
250	250	12,5	88,0	112,0	0,936	84	17283	1266	10161	812,9	975,2	9,52	1	3978	346,2	1148
260	260	6	47,1	60,03	1,019	170	9970	739,5	6405	492,7	568,8	10,33	4	1836	159,8	615,2
260	260	7,1	55,0	70,09	1,010	144	11716	859,4	7374	567,2	659,0	10,26	3	2488	201,4	718,3
260	260	8	61,6	78,44	1,006	128	13087	954,7	8178	629,1	734,0	10,21	2	2785	260,6	803,9
260	260	8,8	67,3	85,76	1,002	117	14283	1037	8869	682,2	799,0	10,17	1	3045	283,6	878,9
260	260	10	75,8	96,57	0,997	103	16035	1156	9865	758,8	893,8	10,11	1	3428	317,3	989,6
260	260	12,5	91,9	117,0	0,976	83	19553	1381	11548	888,3	1063	9,93	1	4155	377,3	1199
300	300	6	54,7	69,63	1,179	169	15434	996,8	9964	664,2	764,2	11,96	4	1921	203,0	713,6
300	300	7,1	63,9	81,45	1,170	144	18160	1161	11516	767,7	888,0	11,89	4	2533	251,5	834,7
300	300	8	71,6	91,24	1,166	128	20312	1293	12801	853,4	990,7	11,84	4	3066	292,9	935,1
300	300	8,8	78,4	99,84	1,162	116	22195	1406	13911	927,4	1080	11,80	3	3544	329,2	1023
300	300	10	88,4	112,6	1,157	103	24966	1572	15519	1035	1211	11,74	2	3996	429,9	1154
300	300	12,5	108,0	137,0	1,136	83	30601	1892	18348	1223	1451	11,57	1	4865	515,0	1404

Table 11.1.2 Cross-sectional properties and resistance values for rectangular hollow sections of steel grade S355J2H (f_y = 355 N/mm^2)



M = weight
A = cross-section area
A_u = external area
A_m/V = cross-section factor in fire design
I_t = St. Venant torsional constant
W_t = torsional section modulus

I = second moment of area
W_{el} = elastic section modulus
W_{pl} = plastic section modulus (shall be used only for CL1...2)
i = radius of gyration

CL = cross-section Class at uniform compression
$N_{c.Rd}$ = design compression resistance (without buckling)
$M_{c.Rd}$ = design bending resistance (respective to cross-section Class)
$V_{pl.Rd}$ = design plastic shear resistance (without shear buckling)

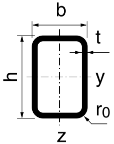
$r_0 = 2{,}0 \times t$ when $t \leq 6{,}0$ mm
$r_0 = 2{,}5 \times t$ when $6{,}0$ mm $< t \leq 10{,}0$ mm
$r_0 = 3{,}0 \times t$ when $t > 10{,}0$ mm

The calculated resistance values are design values (see Chapter 2) based on recommended partial safety factor values **γ_{M0} = 1,0** and **γ_{M1} = 1,0 (for Class 4 circular hollow sections γ_{M1} = 1,1)** as given in Eurocode 3 (EN 1993). Partial safety factor values may differ in each country. National values must be checked from the National Annex of the relevant country.

h	b	t	M	A	A_u	A_m/V	I_t	W_t	I_y	$W_{el.y}$	$W_{pl.y}$	i_y	I_z	$W_{el.z}$	$W_{pl.z}$	i_z	CL		$N_{c.Rd}$	$M_{c.y.Rd}$	$M_{c.z.Rd}$	$V_{pl.y.Rd}$	$V_{pl.z.Rd}$
mm	mm	mm	kg/m	mm^2 x 10^2	m^2/m	1/m	mm^4 x 10^4	mm^3 x 10^3	mm^4 x 10^4	mm^3 x 10^3	mm^3 x 10^3	mm x 10	mm^4 x 10^4	mm^3 x 10^3	mm^3 x 10^3	mm x 10	h	b	kN	kNm	kNm	kN	kN
40	20	2	1,68	2,14	0,113	529	3,45	2,36	4,05	2,02	2,61	1,38	1,34	1,34	1,60	0,79	1	1	75,86	0,93	0,57	14,60	29,20
40	20	2,5	2,03	2,59	0,111	430	4,06	2,72	4,69	2,35	3,09	1,35	1,54	1,54	1,88	0,77	1	1	91,91	1,10	0,67	17,69	35,38
40	20	3	2,36	3,01	0,110	365	4,57	3,00	5,21	2,60	3,50	1,32	1,68	1,68	2,12	0,75	1	1	106,8	1,24	0,75	20,55	41,10
40	30	2	1,99	2,54	0,133	525	7,07	3,79	5,49	2,75	3,37	1,47	3,51	2,34	2,77	1,18	1	1	90,06	1,20	0,98	22,28	29,71
40	30	2,5	2,42	3,09	0,131	425	8,47	4,46	6,45	3,23	4,03	1,45	4,10	2,74	3,30	1,15	1	1	109,7	1,43	1,17	27,13	36,18
40	30	3	2,83	3,61	0,130	359	9,72	5,03	7,27	3,63	4,61	1,42	4,60	3,07	3,77	1,13	1	1	128,1	1,64	1,34	31,69	42,26
50	30	2	2,31	2,94	0,153	521	9,77	4,84	9,54	3,81	4,74	1,80	4,29	2,86	3,33	1,21	1	1	104,3	1,68	1,18	22,57	37,62
50	30	2,5	2,82	3,59	0,151	422	11,74	5,72	11,30	4,52	5,70	1,77	5,05	3,37	3,98	1,19	1	1	127,4	2,02	1,41	27,59	45,98
50	30	3	3,30	4,21	0,150	356	13,53	6,49	12,83	5,13	6,57	1,75	5,70	3,80	4,58	1,16	1	1	149,4	2,33	1,63	32,34	53,91
50	30	4	4,20	5,35	0,146	273	16,53	7,71	15,25	6,10	8,05	1,69	6,69	4,46	5,58	1,12	1	1	189,9	2,86	1,98	41,10	68,51
60	40	2	2,93	3,74	0,193	517	20,70	8,12	18,41	6,14	7,47	2,22	9,83	4,92	5,65	1,62	2	1	132,7	2,65	2,01	30,64	45,96
60	40	2,5	3,60	4,59	0,191	417	25,14	9,72	22,07	7,36	9,06	2,19	11,74	5,87	6,84	1,60	1	1	162,9	3,21	2,43	37,62	56,43
60	40	3	4,25	5,41	0,190	351	29,28	11,17	25,38	8,46	10,53	2,17	13,44	6,72	7,94	1,58	1	1	192,0	3,74	2,82	44,34	66,51
60	40	4	5,45	6,95	0,186	268	36,67	13,65	30,99	10,33	13,16	2,11	16,28	8,14	9,89	1,53	1	1	246,7	4,67	3,51	56,96	85,44
60	40	5	6,56	8,36	0,183	219	42,85	15,60	35,33	11,78	15,38	2,06	18,43	9,21	11,52	1,48	1	1	296,6	5,46	4,09	68,51	102,8
70	50	2	3,56	4,54	0,233	514	37,45	12,20	31,48	8,99	10,80	2,63	18,76	7,50	8,58	2,03	3	1	161,1	3,83	2,66	38,75	54,24
70	50	2,5	4,39	5,59	0,231	414	45,75	14,72	38,01	10,86	13,16	2,61	22,59	9,04	10,45	2,01	1	1	198,4	4,67	3,71	47,73	66,82
70	50	3	5,19	6,61	0,230	348	53,62	17,06	44,05	12,59	15,40	2,58	26,10	10,44	12,21	1,99	1	1	234,6	5,47	4,33	56,43	79,01
70	50	4	6,71	8,55	0,226	265	68,07	21,19	54,67	15,62	19,48	2,53	32,22	12,89	15,41	1,94	1	1	303,5	6,91	5,47	73,00	102,2
70	50	5	8,13	10,36	0,223	215	80,77	24,64	63,46	18,13	23,06	2,48	37,20	14,88	18,20	1,90	1	1	367,6	8,19	6,46	88,44	123,8
80	40	2	3,56	4,54	0,233	514	30,88	11,00	37,36	9,34	11,61	2,87	12,72	6,36	7,17	1,67	4	1	151,2	4,12	2,10	31,00	61,99
80	40	2,5	4,39	5,59	0,231	414	37,58	13,24	45,11	11,28	14,15	2,84	15,26	7,63	8,72	1,65	2	1	198,4	5,02	3,09	38,18	76,37
80	40	3	5,19	6,61	0,230	348	43,88	15,28	52,25	13,06	16,54	2,81	17,56	8,78	10,16	1,63	1	1	234,6	5,87	3,61	45,15	90,29
80	40	4	6,71	8,55	0,226	265	55,24	18,84	64,79	16,20	20,91	2,75	21,49	10,74	12,77	1,59	1	1	303,5	7,42	4,54	58,40	116,8
80	40	5	8,13	10,36	0,223	215	64,97	21,74	75,11	18,78	24,74	2,69	24,59	12,30	15,02	1,54	1	1	367,6	8,78	5,33	70,75	141,5

h mm	b mm	t mm	M kg/m	A mm^2 x 10^2	A_u m^2/m	A_m/V 1/m	I_t mm^4 x 10^4	W_t mm^3 x 10^3	I_y mm^4 x 10^4	$W_{el.y}$ mm^3 x 10^3	$W_{pl.y}$ mm^3 x 10^3	i_y mm x 10	I_z mm^4 x 10^4	$W_{el.z}$ mm^3 x 10^3	$W_{pl.z}$ mm^3 x 10^3	i_z mm x 10	CL h	CL b	$N_{c.Rd}$ kN	$M_{c.y.Rd}$ kNm	$M_{c.z.Rd}$ kNm	$V_{pl.y.Rd}$ kN	$V_{pl.z.Rd}$ kN
80	60	2,5	5,17	6,59	0,271	412	75,07	20,73	60,13	15,03	18,02	3,02	38,61	12,87	14,81	2,42	2	1	233,9	6,40	5,26	57,88	77,17
80	60	3	6,13	7,81	0,270	345	88,35	24,14	70,05	17,51	21,16	3,00	44,89	14,96	17,37	2,40	1	1	277,2	7,51	6,17	68,59	91,45
80	60	4	7,97	10,15	0,266	262	113,1	30,32	87,92	21,98	26,99	2,94	56,12	18,71	22,12	2,35	1	1	360,3	9,58	7,85	89,14	118,9
80	60	5	9,70	12,36	0,263	213	135,5	35,67	103,3	25,82	32,24	2,89	65,66	21,89	26,38	2,31	1	1	438,6	11,44	9,36	108,5	144,7
90	50	2,5	5,17	6,59	0,271	412	65,30	19,24	70,26	15,61	19,25	3,27	28,24	11,29	12,82	2,07	3	1	233,9	6,83	4,01	48,23	86,82
90	50	3	6,13	7,81	0,270	345	76,67	22,36	81,85	18,19	22,60	3,24	32,74	13,10	15,03	2,05	2	1	277,2	8,02	5,33	57,16	102,9
90	50	4	7,97	10,15	0,266	262	97,70	27,96	102,7	22,82	28,82	3,18	40,71	16,28	19,09	2,00	1	1	360,3	10,23	6,78	74,28	133,7
90	50	5	9,70	12,36	0,263	213	116,5	32,75	120,6	26,80	34,41	3,12	47,37	18,95	22,70	1,96	1	1	438,6	12,22	8,06	90,45	162,8
100	40	2,5	5,17	6,59	0,271	412	50,52	16,76	79,32	15,86	20,23	3,47	18,78	9,39	10,59	1,69	4	1	218,5	7,18	3,09	38,59	96,46
100	40	3	6,13	7,81	0,270	345	59,05	19,39	92,34	18,47	23,75	3,44	21,67	10,84	12,38	1,67	2	1	277,2	8,43	4,40	45,72	114,3
100	40	4	7,97	10,15	0,266	262	74,53	24,04	115,7	23,14	30,26	3,38	26,69	13,35	15,65	1,62	1	1	360,3	10,74	5,56	59,43	148,6
100	40	5	9,70	12,36	0,263	213	87,92	27,90	135,6	27,12	36,09	3,31	30,76	15,38	18,52	1,58	1	1	438,6	12,81	6,58	72,36	180,9
100	50	2,5	5,56	7,09	0,291	411	75,39	21,49	91,20	18,24	22,67	3,59	31,06	12,42	14,01	2,09	4	1	236,2	8,05	4,11	48,43	96,86
100	50	3	6,60	8,41	0,290	345	88,56	25,01	106,5	21,29	26,66	3,56	36,06	14,42	16,44	2,07	2	1	298,5	9,46	5,84	57,44	114,9
100	50	4	8,59	10,95	0,286	261	113,0	31,35	134,1	26,83	34,10	3,50	44,95	17,98	20,93	2,03	1	1	388,7	12,11	7,43	74,80	149,6
100	50	5	10,5	13,36	0,283	212	134,9	36,80	158,2	31,64	40,84	3,44	52,45	20,98	24,95	1,98	1	1	474,1	14,50	8,86	91,25	182,5
100	50	6	12,3	15,63	0,279	179	154,2	41,43	178,8	35,75	46,90	3,38	58,67	23,47	28,52	1,94	1	1	555,0	16,65	10,12	106,8	213,6
100	60	2,5	5,96	7,59	0,311	410	103,3	26,23	103,1	20,62	25,11	3,69	46,88	15,63	17,68	2,49	4	1	254,0	8,91	5,18	58,33	97,22
100	60	3	7,07	9,01	0,310	344	121,7	30,64	120,6	24,11	29,57	3,66	54,65	18,22	20,79	2,46	2	1	319,8	10,50	7,38	69,24	115,4
100	60	4	9,22	11,75	0,306	261	156,3	38,68	152,6	30,52	37,94	3,60	68,68	22,89	26,60	2,42	1	1	417,1	13,47	9,44	90,29	150,5
100	60	5	11,3	14,36	0,303	211	187,9	45,75	180,8	36,15	45,59	3,55	80,83	26,94	31,88	2,37	1	1	509,6	16,18	11,32	110,3	183,9
100	60	6	13,2	16,83	0,299	178	216,4	51,92	205,3	41,06	52,54	3,49	91,20	30,40	36,64	2,33	1	1	597,6	18,65	13,01	129,4	215,6
100	80	2,5	6,74	8,59	0,351	409	165,8	35,73	126,9	25,37	29,98	3,84	90,17	22,54	25,77	3,24	4	2	289,5	10,64	7,52	78,24	97,80
100	80	3	8,01	10,21	0,350	343	196,1	41,91	148,8	29,76	35,39	3,82	105,6	26,41	30,40	3,22	2	1	362,4	12,56	10,79	92,99	116,2
100	80	4	10,5	13,35	0,346	259	253,8	53,38	189,5	37,89	45,62	3,77	134,2	33,54	39,15	3,17	1	1	473,9	16,19	13,90	121,6	152,0
100	80	5	12,8	16,36	0,343	210	307,6	63,72	225,9	45,19	55,09	3,72	159,6	39,90	47,24	3,12	1	1	580,6	19,56	16,77	149,0	186,2
100	80	6	15,1	19,23	0,339	176	357,4	72,98	258,4	51,68	63,82	3,67	182,1	45,53	54,67	3,08	1	1	682,8	22,66	19,41	175,2	219,0
120	40	2,5	5,96	7,59	0,311	410	63,77	20,27	126,7	21,12	27,32	4,09	22,30	11,15	12,47	1,71	4	1	228,5	9,70	3,31	38,89	116,7
120	40	3	7,07	9,01	0,310	344	74,56	23,51	148,0	24,67	32,16	4,05	25,79	12,89	14,60	1,69	4	1	297,5	11,42	4,24	46,16	138,5
120	40	4	9,22	11,75	0,306	261	94,23	29,24	186,9	31,15	41,21	3,99	31,90	15,95	18,53	1,65	2	1	417,1	14,63	6,58	60,20	180,6
120	40	5	11,3	14,36	0,303	211	111,4	34,05	220,8	36,80	49,45	3,92	36,93	18,46	22,02	1,60	1	1	509,6	17,55	7,82	73,56	220,7
120	40	6	13,2	16,83	0,299	178	126,0	38,02	250,0	41,66	56,89	3,85	40,97	20,49	25,08	1,56	1	1	597,6	20,20	8,90	86,25	258,8
120	50	2,5	6,35	8,09	0,331	410	96,03	26,01	144,0	23,99	30,26	4,22	36,70	14,68	16,39	2,13	4	1	246,2	10,74	4,39	48,76	117,0
120	50	3	7,54	9,61	0,330	343	112,9	30,32	168,6	28,10	35,67	4,19	42,69	17,08	19,26	2,11	4	1	318,8	12,66	5,63	57,92	139,0
120	50	4	9,85	12,55	0,326	260	144,2	38,13	213,8	35,64	45,85	4,13	53,43	21,37	24,61	2,06	2	1	445,5	16,28	8,74	75,64	181,5
120	50	5	12,1	15,36	0,323	210	172,4	44,92	253,9	42,32	55,20	4,07	62,62	25,05	29,45	2,02	1	1	545,1	19,60	10,46	92,57	222,2
120	50	6	14,2	18,03	0,319	177	197,6	50,76	289,0	48,16	63,73	4,00	70,36	28,14	33,80	1,98	1	1	640,2	22,63	12,00	108,7	260,9
120	60	2,5	6,74	8,59	0,351	409	132,6	31,75	161,2	26,87	33,20	4,33	55,15	18,38	20,56	2,53	4	1	264,0	11,79	5,54	58,68	117,4
120	60	3	8,01	10,21	0,350	343	156,3	37,14	189,1	31,52	39,18	4,30	64,40	21,47	24,21	2,51	4	1	340,1	13,91	7,10	69,74	139,5
120	60	4	10,5	13,35	0,346	259	201,1	47,05	240,7	40,12	50,49	4,25	81,25	27,08	31,08	2,47	2	1	473,9	17,92	11,03	91,19	182,4
120	60	5	12,8	16,36	0,343	210	242,2	55,85	287,0	47,83	60,95	4,19	95,99	32,00	37,38	2,42	1	1	580,6	21,64	13,27	111,7	223,5
120	60	6	15,1	19,23	0,339	176	279,7	63,60	328,0	54,67	70,57	4,13	108,8	36,26	43,12	2,38	1	1	682,8	25,05	15,31	131,4	262,8

Table 11.1.2 Cross-sectional properties and resistance values for rectangular hollow sections of steel grade S355J2H (f_y = 355 N/mm^2) *continued*



M = weight
A = cross-section area
A_u = external area
A_m/V = cross-section factor in fire design
I_t = St. Venant torsional constant
W_t = torsional section modulus

I = second moment of area
W_{el} = elastic section modulus
W_{pl} = plastic section modulus (shall be used only for CL1...2)
i = radius of gyration

CL = cross-section Class at uniform compression
$N_{c.Rd}$ = design compression resistance (without buckling)
$M_{c.Rd}$ = design bending resistance (respective to cross-section Class)
$V_{pl.Rd}$ = design plastic shear resistance (without shear buckling)

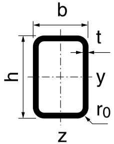
$r_0 = 2,0 \times t$ when $t \le 6,0$ mm
$r_0 = 2,5 \times t$ when $6,0 \text{ mm} < t \le 10,0$ mm
$r_0 = 3,0 \times t$ when $t > 10,0$ mm

The calculated resistance values are design values (see Chapter 2) based on recommended partial safety factor values **γ_{M0} = 1,0** and **γ_{M1} = 1,0 (for Class 4 circular hollow sections γ_{M1} = 1,1)** as given in Eurocode 3 (EN 1993). Partial safety factor values may differ in each country. National values must be checked from the National Annex of the relevant country.

h mm	b mm	t mm	M kg/m	A mm^2 x 10^2	A_u m^2/m	A_m/V 1/m	I_t mm^4 x 10^4	W_t mm^3 x 10^3	I_y mm^4 x 10^4	$W_{el.y}$ mm^3 x 10^3	$W_{pl.y}$ mm^3 x 10^3	i_y mm x 10	I_z mm^4 x 10^4	$W_{el.z}$ mm^3 x 10^3	$W_{pl.z}$ mm^3 x 10^3	i_z mm x 10	CL h	CL b	$N_{c.Rd}$ kN	$M_{c.y.Rd}$ kNm	$M_{c.z.Rd}$ kNm	$V_{pl.y.Rd}$ kN	$V_{pl.z.Rd}$ kN
120	80	2,5	7,53	9,59	0,391	408	215,8	43,23	195,8	32,63	39,07	4,52	105,2	26,30	29,65	3,31	4	2	299,5	13,87	8,02	78,61	117,9
120	80	3	8,96	11,41	0,390	342	255,5	50,80	230,2	38,37	46,20	4,49	123,4	30,86	35,02	3,29	4	1	382,7	16,40	10,25	93,53	140,3
120	80	4	11,7	14,95	0,386	258	331,2	64,93	294,6	49,10	59,77	4,44	157,3	39,32	45,23	3,24	2	1	530,7	21,22	16,06	122,6	183,8
120	80	5	14,4	18,36	0,383	209	402,3	77,77	353,1	58,86	72,45	4,39	187,8	46,94	54,74	3,20	1	1	651,6	25,72	19,43	150,5	225,7
120	80	6	17,0	21,63	0,379	175	468,5	89,40	406,1	67,68	84,25	4,33	215,0	53,76	63,55	3,15	1	1	768,0	29,91	22,56	177,4	266,0
120	80	7,1	19,4	24,65	0,369	150	535,1	99,99	442,1	73,68	93,59	4,24	234,4	58,60	70,68	3,08	1	1	875,0	33,22	25,09	202,1	303,1
120	80	8	21,4	27,24	0,366	134	584,0	108,0	475,8	79,31	102,0	4,18	251,7	62,92	76,93	3,04	1	1	967,1	36,20	27,31	223,3	335,0
120	80	8,8	23,1	29,44	0,362	123	623,5	114,3	501,8	83,63	108,8	4,13	264,8	66,21	81,97	3,00	1	1	1045	38,61	29,10	241,4	362,1
120	80	10	25,6	32,57	0,357	110	675,6	122,4	534,1	89,02	117,8	4,05	281,1	70,29	88,68	2,94	1	1	1156	41,82	31,48	267,0	400,5
120	100	2,5	8,31	10,59	0,431	407	309,4	54,73	230,3	38,38	44,95	4,66	174,4	34,88	39,73	4,06	4	4	319,5	12,90	10,75	98,65	118,4
120	100	3	9,90	12,61	0,430	341	367,0	64,47	271,3	45,21	53,22	4,64	205,3	41,06	47,03	4,04	4	2	425,3	18,89	13,70	117,5	141,0
120	100	4	13,0	16,55	0,426	258	477,8	82,83	348,4	58,07	69,05	4,59	263,2	52,65	60,98	3,99	2	1	587,5	24,51	21,65	154,2	185,0
120	100	5	16,0	20,36	0,423	208	582,9	99,75	419,3	69,88	83,95	4,54	316,3	63,25	74,09	3,94	1	1	722,6	29,80	26,30	189,6	227,6
120	100	6	18,9	24,03	0,419	175	682,0	115,3	484,1	80,68	97,93	4,49	364,6	72,91	86,38	3,89	1	1	853,2	34,77	30,67	223,9	268,7
140	60	2,5	7,53	9,59	0,391	408	162,7	37,26	236,6	33,79	42,29	4,97	63,43	21,14	23,43	2,57	4	1	271,0	15,01	5,81	58,96	137,6
140	60	3	8,96	11,41	0,390	342	191,9	43,64	278,1	39,73	49,98	4,94	74,16	24,72	27,63	2,55	4	1	352,5	17,74	7,52	70,15	163,7
140	60	4	11,7	14,95	0,386	258	247,1	55,42	355,6	50,80	64,63	4,88	93,81	31,27	35,56	2,51	3	1	530,7	22,95	11,10	91,91	214,5
140	60	5	14,4	18,36	0,383	209	298,0	65,94	425,9	60,84	78,30	4,82	111,2	37,05	42,88	2,46	1	1	651,6	27,80	15,22	112,9	263,4
140	60	6	17,0	21,63	0,379	175	344,5	75,29	489,2	69,88	91,01	4,76	126,3	42,11	49,60	2,42	1	1	768,0	32,31	17,61	133,0	310,4
140	70	2,5	7,92	10,09	0,411	408	213,1	44,00	260,2	37,17	45,72	5,08	89,30	25,51	28,35	2,98	4	1	288,7	16,23	7,07	68,93	137,9
140	70	3	9,43	12,01	0,410	341	252,0	51,66	306,2	43,75	54,09	5,05	104,7	29,91	33,49	2,95	4	1	373,8	19,20	9,15	82,04	164,1
140	70	4	12,4	15,75	0,406	258	326,0	65,94	392,6	56,09	70,07	4,99	133,2	38,05	43,24	2,91	3	1	559,1	24,88	13,51	107,6	215,2
140	70	5	15,2	19,36	0,403	208	395,1	78,88	471,5	67,35	85,05	4,94	158,7	45,35	52,31	2,86	1	1	687,1	30,19	18,57	132,2	264,5
140	70	6	17,9	22,83	0,399	175	459,1	90,54	543,1	77,59	99,05	4,88	181,4	51,84	60,71	2,82	1	1	810,6	35,16	21,55	156,0	312,0
140	80	3	9,90	12,61	0,430	341	317,1	59,69	334,4	47,77	58,20	5,15	141,2	35,31	39,64	3,35	4	1	395,1	20,66	10,85	93,97	164,5
140	80	4	13,0	16,55	0,426	258	411,6	76,48	429,6	61,37	75,51	5,10	180,4	45,10	51,31	3,30	3	1	587,5	26,81	16,01	123,3	215,8
140	80	5	16,0	20,36	0,423	208	500,5	91,83	517,1	73,87	91,80	5,04	215,9	53,99	62,24	3,26	1	1	722,6	32,59	22,09	151,7	265,5
140	80	6	18,9	24,03	0,419	175	583,8	105,8	597,0	85,29	107,1	4,98	248,0	61,99	72,43	3,21	1	1	853,2	38,02	25,71	179,1	313,5

h	b	t	M	A	A_u	A_m/V	I_t	W_t	I_y	$W_{el.y}$	$W_{pl.y}$	i_y	I_z	$W_{el.z}$	$W_{pl.z}$	i_z	CL		$N_{c.Rd}$	$M_{c.y.Rd}$	$M_{c.z.Rd}$	$V_{pl.y.Rd}$	$V_{pl.z.Rd}$
mm	mm	mm	kg/m	mm^2 x 10^2	m^2/m	1/m	mm^4 x 10^4	mm^3 x 10^3	mm^4 x 10^4	mm^3 x 10^3	mm^3 x 10^3	mm x 10	mm^4 x 10^4	mm^3 x 10^3	mm^3 x 10^3	mm x 10	h	b	kN	kNm	kNm	kN	kN
150	50	2,5	7,53	9,59	0,391	408	127,7	32,78	254,1	33,88	43,52	5,15	45,17	18,07	19,95	2,17	4	1	256,0	15,45	4,69	49,13	147,4
150	50	3	8,96	11,41	0,390	342	150,2	38,28	298,6	39,81	51,43	5,12	52,65	21,06	23,49	2,15	4	1	336,1	18,26	6,11	58,46	175,4
150	50	4	11,7	14,95	0,386	258	192,1	48,30	381,4	50,85	66,47	5,05	66,16	26,47	30,13	2,10	4	1	509,0	23,60	8,99	76,59	229,8
150	50	5	14,4	18,36	0,383	209	230,1	57,11	456,3	60,84	80,48	4,99	77,87	31,15	36,20	2,06	2	1	651,6	28,57	12,85	94,06	282,2
150	50	6	17,0	21,63	0,379	175	264,0	64,77	523,5	69,80	93,48	4,92	87,89	35,16	41,72	2,02	1	1	768,0	33,19	14,81	110,9	332,5
150	100	3	11,3	14,41	0,490	340	507,2	81,40	460,6	61,42	73,48	5,65	247,6	49,53	55,76	4,15	4	2	442,6	26,08	14,81	118,1	177,2
150	100	4	14,9	18,95	0,486	257	661,6	104,9	594,6	79,28	95,67	5,60	318,6	63,71	72,50	4,10	4	1	651,0	33,96	21,77	155,3	233,0
150	100	5	18,3	23,36	0,483	207	808,7	126,8	719,2	95,89	116,7	5,55	384,0	76,80	88,34	4,05	2	1	829,1	41,44	31,36	191,5	287,2
150	100	6	21,7	27,63	0,479	173	948,3	147,1	834,7	111,3	136,7	5,50	444,2	88,84	103,3	4,01	1	1	981,0	48,52	36,67	226,5	339,8
150	100	7,1	24,9	31,75	0,470	148	1096	166,8	926,9	123,6	154,1	5,40	493,6	98,72	116,6	3,94	1	1	1127	54,70	41,38	260,3	390,5
150	100	8	27,7	35,24	0,466	132	1206	181,9	1008	134,4	169,2	5,35	535,7	107,1	127,9	3,90	1	1	1251	60,05	45,39	288,9	433,4
150	100	8,8	30,0	38,24	0,462	121	1298	194,2	1074	143,2	181,7	5,30	569,5	113,9	137,3	3,86	1	1	1358	64,51	48,72	313,5	470,3
150	100	10	33,4	42,57	0,457	107	1426	211,0	1162	154,9	199,2	5,22	614,4	122,9	150,3	3,80	1	1	1511	70,70	53,34	349,0	523,5
160	70	4	13,6	17,35	0,446	257	389,7	75,90	549,0	68,63	86,62	5,63	150,6	43,04	48,52	2,95	4	1	576,3	30,75	14,20	108,2	247,4
160	70	5	16,8	21,36	0,443	207	472,5	90,96	661,6	82,70	105,4	5,57	179,9	51,39	58,81	2,90	2	1	758,1	37,42	20,88	133,2	304,5
160	70	6	19,8	25,23	0,439	174	549,6	104,6	764,8	95,60	123,1	5,51	206,1	58,88	68,39	2,86	1	1	895,8	43,69	24,28	157,4	359,8
160	70	8	25,2	32,04	0,426	133	683,7	126,7	908,7	113,6	150,7	5,33	243,3	69,50	83,55	2,76	1	1	1138	53,50	29,66	199,9	456,9
160	80	3	10,8	13,81	0,470	340	380,3	68,59	463,8	57,98	71,41	5,80	159,0	39,76	44,26	3,39	4	1	404,2	25,35	11,32	94,34	188,7
160	80	4	14,3	18,15	0,466	257	494,1	88,03	597,7	74,71	92,86	5,74	203,5	50,89	57,39	3,35	4	1	604,7	32,97	16,82	124,0	248,0
160	80	5	17,6	22,36	0,463	207	601,3	105,9	721,7	90,21	113,2	5,68	244,1	61,03	69,74	3,30	2	1	793,6	40,17	24,76	152,7	305,5
160	80	6	20,8	26,43	0,459	174	702,1	122,3	836,0	104,5	132,3	5,62	280,9	70,22	81,31	3,26	1	1	938,4	46,97	28,86	180,6	361,2
160	80	7,1	23,8	30,33	0,449	148	805,5	137,8	922,6	115,3	148,6	5,52	310,1	77,53	91,39	3,20	1	1	1077	52,74	32,44	207,2	414,4
160	80	8	26,4	33,64	0,446	132	882,3	149,5	1001	125,2	162,9	5,46	335,0	83,74	99,97	3,16	1	1	1194	57,81	35,49	229,8	459,7
160	80	10	31,8	40,57	0,437	108	1031	171,6	1146	143,3	191,0	5,32	379,8	94,95	116,7	3,06	1	1	1440	67,79	41,42	277,2	554,3
160	90	3	11,3	14,41	0,490	340	465,4	77,82	500,8	62,60	76,12	5,90	206,8	45,95	51,31	3,79	4	2	425,5	27,02	13,17	106,3	189,0
160	90	4	14,9	18,95	0,486	257	606,2	100,2	646,4	80,80	99,10	5,84	265,5	59,01	66,66	3,74	4	1	633,1	35,18	19,54	139,8	248,6
160	90	5	18,3	23,36	0,483	207	739,7	120,9	781,8	97,72	120,9	5,79	319,5	71,00	81,16	3,70	2	1	829,1	42,92	28,81	172,3	306,4
160	90	6	21,7	27,63	0,479	173	866,0	140,0	907,2	113,4	141,6	5,73	368,9	81,98	94,82	3,65	1	1	981,0	50,25	33,66	203,9	362,5
160	90	7,1	24,9	31,75	0,470	148	997,9	158,4	1006	125,8	159,5	5,63	409,3	90,96	106,9	3,59	1	1	1127	56,61	37,96	234,3	416,5
160	90	8	27,7	35,24	0,466	132	1097	172,4	1094	136,7	175,0	5,57	443,5	98,55	117,2	3,55	1	1	1251	62,13	41,60	260,0	462,3
160	90	8,8	30,0	38,24	0,462	121	1178	183,9	1165	145,6	188,0	5,52	470,8	104,6	125,7	3,51	1	1	1358	66,73	44,63	282,2	501,7
180	100	4	16,8	21,35	0,546	256	853,9	127,1	926,0	102,9	125,9	6,59	373,9	74,78	84,02	4,18	4	1	678,6	44,69	23,38	156,3	281,3
180	100	5	20,7	26,36	0,543	206	1045	153,9	1124	124,9	154,0	6,53	451,8	90,35	102,6	4,14	3	1	935,6	54,68	32,08	192,9	347,3
180	100	5,6	23,0	29,30	0,541	185	1155	169,1	1237	137,4	170,3	6,50	495,7	99,14	113,3	4,11	2	1	1040	60,44	40,21	214,5	386,0
180	100	6	24,5	31,23	0,539	173	1227	178,9	1310	145,5	180,8	6,48	523,8	104,8	120,2	4,10	2	1	1109	64,20	42,68	228,6	411,5
180	100	7,1	28,3	36,01	0,530	147	1420	203,5	1464	162,7	204,9	6,38	585,7	117,1	136,4	4,03	1	1	1278	72,75	48,41	263,6	474,5
180	100	8	31,4	40,04	0,526	131	1565	222,5	1598	177,6	225,6	6,32	637,5	127,5	149,9	3,99	1	1	1422	80,10	53,23	293,1	527,6
180	100	8,8	34,2	43,52	0,522	120	1688	238,2	1709	189,9	243,1	6,27	679,7	135,9	161,3	3,95	1	1	1545	86,28	57,27	318,6	573,5
180	100	10	38,1	48,57	0,517	106	1859	259,6	1859	206,6	267,5	6,19	736,4	147,3	177,3	3,89	1	1	1724	94,97	62,92	355,5	639,9

Table 11.1.2 Cross-sectional properties and resistance values for rectangular hollow sections of steel grade S355J2H (f_y = 355 N/mm^2) *continued*



M = weight
A = cross-section area
A_u = external area
A_m/V = cross-section factor in fire design
I_t = St. Venant torsional constant
W_t = torsional section modulus

I = second moment of area
W_{el} = elastic section modulus
W_{pl} = plastic section modulus (shall be used only for CL1...2)
i = radius of gyration

CL = cross-section Class at uniform compression
$N_{c.Rd}$ = design compression resistance (without buckling)
$M_{c.Rd}$ = design bending resistance (respective to cross-section Class)
$V_{pl.Rd}$ = design plastic shear resistance (without shear buckling)

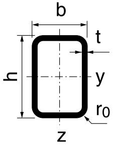
r_0 = 2,0 x t when t ≤ 6,0 mm
r_0 = 2,5 x t when 6,0 mm < t ≤ 10,0 mm
r_0 = 3,0 x t when t > 10,0 mm

The calculated resistance values are design values (see Chapter 2) based on recommended partial safety factor values **γ_{M0} = 1,0** and **γ_{M1} = 1,0 (for Class 4 circular hollow sections γ_{M1} = 1,1)** as given in Eurocode 3 (EN 1993). Partial safety factor values may differ in each country. National values must be checked from the National Annex of the relevant country.

h	b	t	M	A	A_u	A_m/V	I_t	W_t	I_y	$W_{el.y}$	$W_{pl.y}$	i_y	I_z	$W_{el.z}$	$W_{pl.z}$	i_z	CL		$N_{c.Rd}$	$M_{c.y.Rd}$	$M_{c.z.Rd}$	$V_{pl.y.Rd}$	$V_{pl.z.Rd}$
mm	mm	mm	kg/m	mm^2 x 10^2	m^2/m	1/m	mm^4 x 10^4	mm^3 x 10^3	mm^4 x 10^4	mm^3 x 10^3	mm^3 x 10^3	mm x 10	mm^4 x 10^4	mm^3 x 10^3	mm^3 x 10^3	mm x 10	h	b	kN	kNm	kNm	kN	kN
180	120	4	18,0	22,95	0,586	255	1160	154,6	1050	116,7	140,0	6,76	563,8	93,97	106,2	4,96	4	2	735,4	49,69	29,56	188,1	282,2
180	120	5	22,3	28,36	0,583	206	1424	187,8	1277	141,9	171,5	6,71	684,0	114,0	130,0	4,91	3	1	1007	60,89	40,47	232,5	348,7
180	120	6	26,4	33,63	0,579	172	1677	219,1	1491	165,7	201,7	6,66	796,3	132,7	152,7	4,87	2	1	1194	71,61	54,19	275,7	413,6
180	120	7,1	30,5	38,85	0,570	147	1949	250,5	1676	186,3	229,5	6,57	895,4	149,2	173,8	4,80	1	1	1379	81,46	61,69	318,5	477,8
180	120	8	34,0	43,24	0,566	131	2156	274,8	1835	203,9	253,1	6,51	978,4	163,1	191,6	4,76	1	1	1535	89,86	68,01	354,5	531,8
180	120	8,8	36,9	47,04	0,562	120	2332	295,1	1967	218,6	273,2	6,47	1047	174,5	206,6	4,72	1	1	1670	96,98	73,35	385,7	578,5
180	120	10	41,3	52,57	0,557	106	2582	323,3	2149	238,8	301,5	6,39	1141	190,1	227,8	4,66	1	1	1866	107,0	80,87	431,0	646,4
180	120	12,5	48,7	62,04	0,536	86	3001	367,5	2352	261,4	341,5	6,16	1252	208,7	258,5	4,49	1	1	2203	121,2	91,75	508,7	763,0
200	80	4	16,8	21,35	0,546	256	663,6	111,1	1046	104,6	132,4	7,00	249,8	62,45	69,55	3,42	4	1	635,4	46,99	18,24	125,0	312,5
200	80	5	20,7	26,36	0,543	206	808,4	134,1	1269	126,9	161,9	6,94	300,4	75,11	84,74	3,38	4	1	873,8	57,46	24,74	154,3	385,9
200	80	6	24,5	31,23	0,539	173	944,8	155,2	1477	147,7	190,0	6,88	346,7	86,69	99,07	3,33	2	1	1109	67,44	35,17	182,9	457,3
200	80	7,1	28,3	36,01	0,529	147	1086	175,6	1646	164,6	214,9	6,76	385,8	96,45	112,1	3,27	1	1	1278	76,29	39,79	210,9	527,1
200	80	8	31,4	40,04	0,526	131	1192	191,1	1796	179,6	236,5	6,70	418,2	104,6	123,0	3,23	1	1	1422	83,97	43,67	234,5	586,2
200	80	8,8	34,2	43,52	0,522	120	1279	203,8	1918	191,8	254,7	6,64	444,2	111,1	132,1	3,19	1	1	1545	90,41	46,89	254,9	637,2
200	80	10	38,1	48,57	0,517	106	1399	220,8	2083	208,3	280,1	6,55	478,5	119,6	144,7	3,14	1	1	1724	99,43	51,36	284,4	711,0
200	100	4	18,0	22,95	0,586	255	985,4	141,8	1200	120,0	148,00	7,23	410,8	82,16	91,70	4,23	4	1	692,2	52,55	24,22	156,8	313,6
200	100	5	22,3	28,36	0,583	206	1206	171,9	1459	145,9	181,40	7,17	496,9	99,39	112,1	4,19	4	1	944,8	64,39	32,85	193,7	387,5
200	100	6	26,4	33,63	0,579	172	1417	200,1	1703	170,3	213,30	7,12	576,9	115,4	131,5	4,14	2	1	1194	75,71	46,68	229,8	459,6
200	100	7,1	30,5	38,85	0,570	147	1641	228,1	1911	191,1	242,40	7,01	647,1	129,4	149,6	4,08	1	1	1379	86,03	53,09	265,4	530,9
200	100	8	34,0	43,24	0,566	131	1811	249,6	2091	209,1	267,30	6,95	705,4	141,1	164,7	4,04	1	1	1535	94,88	58,45	295,4	590,9
200	100	8,8	36,9	47,04	0,562	120	1954	267,5	2240	224,0	288,30	6,90	753,1	150,6	177,4	4,00	1	1	1670	102,4	62,97	321,4	642,8
200	100	10	41,3	52,57	0,557	106	2154	292,1	2444	244,4	318,10	6,82	817,7	163,6	195,3	3,94	1	1	1866	112,9	69,31	359,1	718,3
200	100	12,5	48,7	62,04	0,536	86	2474	328,8	2659	265,9	359,10	6,55	892,2	178,4	220,8	3,79	1	1	2203	127,5	78,38	423,9	847,8

h mm	b mm	t mm	M kg/m	A mm^2 x 10^2	A_u m^2/m	A_m/V 1/m	I_t mm^4 x 10^4	W_t mm^3 x 10^3	I_y mm^4 x 10^4	$W_{el.y}$ mm^3 x 10^3	$W_{pl.y}$ mm^3 x 10^3	i_y mm x 10	I_z mm^4 x 10^4	$W_{el.z}$ mm^3 x 10^3	$W_{pl.z}$ mm^3 x 10^3	i_z mm x 10	CL h	CL b	$N_{c.Rd}$ kN	$M_{c.y.Rd}$ kNm	$M_{c.z.Rd}$ kNm	$V_{pl.y.Rd}$ kN	$V_{pl.z.Rd}$ kN
200	120	5	23,8	30,36	0,623	205	1652	209,9	1649	164,9	200,90	7,37	750,1	125,0	141,5	4,97	4	1	1016	71,31	41,45	233,3	388,9
200	120	6	28,3	36,03	0,619	172	1947	245,1	1929	192,9	236,60	7,32	874,4	145,7	166,3	4,93	2	1	1279	83,97	59,05	277,0	461,6
200	120	7,1	32,7	41,69	0,610	146	2265	280,7	2175	217,5	269,70	7,22	986,0	164,3	189,8	4,86	1	1	1480	95,76	67,39	320,5	534,1
200	120	8	36,5	46,44	0,606	130	2507	308,3	2386	238,6	298,00	7,17	1079	179,8	209,5	4,82	1	1	1649	105,8	74,37	357,0	594,9
200	120	8,8	39,7	50,56	0,602	119	2714	331,4	2562	256,2	322,00	7,12	1156	192,7	226,2	4,78	1	1	1795	114,3	80,29	388,6	647,7
200	120	10	44,4	56,57	0,597	106	3007	363,7	2806	280,6	356,10	7,04	1262	210,4	249,8	4,72	1	1	2008	126,4	88,68	434,8	724,6
200	120	12,5	52,6	67,04	0,576	86	3514	415,7	3099	309,9	406,00	6,80	1397	232,9	285,3	4,57	1	1	2380	144,1	101,3	515,3	858,8
220	120	5	25,4	32,36	0,663	205	1885	231,9	2082	189,3	232,20	8,02	816,3	136,1	153,0	5,02	4	1	1038	82,44	43,00	234,1	429,1
220	120	6	30,2	38,43	0,659	172	2222	271,1	2439	221,7	273,80	7,97	952,4	158,7	180,0	4,98	3	1	1364	97,19	56,35	278,0	509,7
220	120	7,1	35,0	44,53	0,650	146	2586	310,8	2757	250,7	312,90	7,87	1077	179,4	205,9	4,92	2	1	1581	111,1	73,08	322,1	590,6
220	120	8	39,0	49,64	0,646	130	2864	341,7	3029	275,4	346,00	7,81	1179	196,6	227,4	4,87	1	1	1762	122,8	80,73	359,1	658,4
220	120	8,8	42,5	54,08	0,642	119	3102	367,7	3258	296,1	374,30	7,76	1265	210,9	245,8	4,84	1	1	1920	132,9	87,24	391,2	717,3
220	120	10	47,5	60,57	0,637	105	3440	404,1	3576	325,1	414,70	7,68	1383	230,6	271,8	4,78	1	1	2150	147,2	96,49	438,1	803,2
250	100	5	26,2	33,36	0,683	205	1620	217,1	2554	204,3	258,50	8,75	609,9	122,0	135,8	4,28	4	1	992,7	91,77	35,62	195,3	488,3
250	100	6	31,1	39,63	0,679	171	1905	253,2	2992	239,4	304,90	8,69	709,6	141,9	159,7	4,23	4	1	1289	108,2	45,79	232,1	580,2
250	100	8	40,2	51,24	0,666	130	2439	317,4	3714	297,1	385,40	8,51	875,1	175,0	201,5	4,13	2	1	1819	136,8	71,51	300,1	750,2
250	100	10	49,1	62,57	0,657	105	2910	373,3	4384	350,7	462,00	8,37	1021	204,2	240,3	4,04	1	1	2221	164,0	85,29	366,4	916,0
250	150	5	30,1	38,36	0,783	204	3285	336,9	3304	264,3	319,80	9,28	1508	201,1	225,5	6,27	4	2	1170	113,5	59,79	294,8	491,3
250	150	6	35,8	45,63	0,779	171	3886	395,7	3886	310,8	378,10	9,23	1768	235,8	266,3	6,23	4	1	1502	134,2	76,70	350,7	584,6
250	150	7,1	41,7	53,05	0,770	145	4543	456,3	4428	354,2	434,60	9,14	2015	268,7	306,3	6,16	3	1	1883	154,3	95,39	407,8	679,6
250	150	8	46,5	59,24	0,766	129	5050	504,0	4886	390,9	482,20	9,08	2219	295,9	339,6	6,12	2	1	2103	171,2	120,5	455,3	758,9
250	150	8,8	50,8	64,64	0,762	118	5488	544,5	5274	422,0	523,10	9,03	2392	318,9	368,1	6,08	1	1	2295	185,7	130,7	496,9	828,1
250	150	10	57,0	72,57	0,757	104	6121	602,1	5825	466,0	582,00	8,96	2634	351,2	409,2	6,02	1	1	2576	206,6	145,3	557,7	929,6
250	150	12,5	68,3	87,04	0,736	85	7315	704,1	6633	530,6	678,30	8,73	3002	400,3	477,5	5,87	1	1	3090	240,8	169,5	669,0	1115
260	140	6	35,8	45,63	0,779	171	3646	382,5	4082	314,0	385,90	9,46	1567	223,9	251,8	5,86	4	1	1473	137,0	71,27	327,4	607,9
260	140	7,1	41,7	53,05	0,770	145	4258	440,8	4648	357,6	443,50	9,36	1785	255,0	289,6	5,80	3	1	1883	157,5	90,52	380,6	706,8
260	140	8	46,5	59,24	0,766	129	4731	486,5	5129	394,5	492,00	9,30	1964	280,6	320,9	5,76	2	1	2103	174,7	113,9	425,0	789,3
260	140	8,8	50,8	64,64	0,762	118	5138	525,3	5536	425,9	533,70	9,25	2115	302,2	347,8	5,72	1	1	2295	189,5	123,5	463,7	861,2
260	140	10	57,0	72,57	0,757	104	5724	580,4	6113	470,2	593,80	9,18	2328	332,5	386,4	5,66	1	1	2576	210,8	137,2	520,6	966,8
260	140	12,5	68,3	87,04	0,736	85	6821	677,2	6950	534,6	691,50	8,94	2648	378,3	450,5	5,52	1	1	3090	245,5	159,9	624,4	1160
260	180	6	39,6	50,43	0,859	170	5566	501,4	4856	373,5	446,90	9,81	2763	307,1	347,9	7,40	4	2	1643	158,6	98,45	422,9	610,8
260	180	7,1	46,1	58,73	0,850	145	6522	580,3	5557	427,5	515,30	9,73	3163	351,4	401,3	7,34	3	1	2085	182,9	124,8	492,5	711,3
260	180	8	51,5	65,64	0,846	129	7267	642,4	6145	472,7	572,70	9,68	3493	388,1	445,8	7,29	2	1	2330	203,3	158,3	550,4	795,0
260	180	8,8	56,3	71,68	0,842	117	7912	695,6	6647	511,3	622,20	9,63	3774	419,4	484,1	7,26	1	1	2545	220,9	171,9	601,0	868,2
260	180	10	63,2	80,57	0,837	104	8850	771,9	7363	566,4	693,80	9,56	4174	463,8	539,5	7,20	1	1	2860	246,3	191,5	675,5	975,8

Table 11.1.2 Cross-sectional properties and resistance values for rectangular hollow sections of steel grade S355J2H (f_y = 355 N/mm^2) *continued*



M = weight
A = cross-section area
A_u = external area
A_m/V = cross-section factor in fire design
I_t = St. Venant torsional constant
W_t = torsional section modulus

I = second moment of area
W_{el} = elastic section modulus
W_{pl} = plastic section modulus (shall be used only for CL1...2)
i = radius of gyration

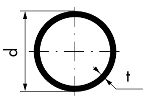
CL = cross-section Class at uniform compression
$N_{c.Rd}$ = design compression resistance (without buckling)
$M_{c.Rd}$ = design bending resistance (respective to cross-section Class)
$V_{pl.Rd}$ = design plastic shear resistance (without shear buckling)

$r_0 = 2{,}0 \times t$ when $t \le 6{,}0$ mm
$r_0 = 2{,}5 \times t$ when $6{,}0$ mm $< t \le 10{,}0$ mm
$r_0 = 3{,}0 \times t$ when $t > 10{,}0$ mm

The calculated resistance values are design values (see Chapter 2) based on recommended partial safety factor values **γ_{M0} = 1,0** and **γ_{M1} = 1,0 (for Class 4 circular hollow sections γ_{M1} = 1,1)** as given in Eurocode 3 (EN 1993). Partial safety factor values may differ in each country. National values must be checked from the National Annex of the relevant country.

h mm	b mm	t mm	M kg/m	A mm² x 10²	A_u m²/m	A_m/V 1/m	I_t mm⁴ x 10⁴	W_t mm³ x 10³	I_y mm⁴ x 10⁴	$W_{el.y}$ mm³ x 10³	$W_{pl.y}$ mm³ x 10³	i_y mm x 10	I_z mm⁴ x 10⁴	$W_{el.z}$ mm³ x 10³	$W_{pl.z}$ mm³ x 10³	i_z mm x 10	CL h	CL b	$N_{c.Rd}$ kN	$M_{c.y.Rd}$ kNm	$M_{c.z.Rd}$ kNm	$V_{pl.y.Rd}$ kN	$V_{pl.z.Rd}$ kN
300	100	5	30,1	38,36	0,783	204	2044	262,2	4065	271,0	348,2	10,29	722,8	144,6	159,6	4,34	4	1	1024	123,6	37,56	196,5	589,6
300	100	6	35,8	45,63	0,779	171	2403	306,2	4777	318,5	411,4	10,23	842,4	168,5	187,9	4,30	4	1	1344	146,1	48,87	233,8	701,5
300	100	7,1	41,7	53,05	0,770	145	2787	350,8	5424	361,6	472,1	10,11	954,1	190,8	215,5	4,24	4	1	1704	167,6	60,85	271,8	815,5
300	100	8	46,5	59,24	0,766	129	3080	385,2	5978	398,5	523,5	10,05	1045	209,0	238,3	4,20	4	1	2017	185,8	70,93	303,6	910,7
300	100	8,8	50,8	64,64	0,762	118	3329	414,2	6446	429,7	567,6	9,99	1120	224,0	257,6	4,16	3	1	2295	201,5	79,53	331,2	993,7
300	100	10	57,0	72,57	0,757	104	3681	454,5	7106	473,7	630,9	9,90	1224	244,9	285,3	4,11	2	1	2576	224,0	101,3	371,8	1115
300	100	12,5	68,3	87,04	0,736	85	4292	521,2	8010	534,0	731,9	9,59	1374	274,8	330,2	3,97	1	1	3090	259,8	117,2	446,0	1338
300	150	6	40,5	51,63	0,879	170	4988	478,6	6074	404,9	499,6	10,85	2080	277,3	309,5	6,35	4	1	1557	177,4	81,75	352,8	705,5
300	150	7,1	47,2	60,15	0,870	145	5834	553,0	6947	463,1	576,1	10,75	2378	317,1	357,0	6,29	4	1	1956	204,5	101,9	411,0	821,9
300	150	8	52,8	67,24	0,866	129	6491	611,5	7684	512,2	640,3	10,69	2623	349,7	396,4	6,25	4	1	2301	227,3	119,1	459,4	918,8
300	150	10	64,8	82,57	0,857	104	7879	732,8	9209	614,0	775,9	10,56	3125	416,7	479,2	6,15	2	1	2931	275,5	170,1	564,1	1128
300	150	12,5	78,1	99,54	0,836	84	9452	861,8	10590	706,3	911,5	10,32	3595	479,3	563,4	6,01	1	1	3534	323,6	200,0	680,1	1360
300	200	6	45,2	57,63	0,979	170	8115	651,2	7370	491,4	587,8	11,31	3962	396,2	446,1	8,29	4	2	1770	208,7	118,5	472,5	708,7
300	200	7,1	52,8	67,25	0,970	144	9524	755,7	8470	564,7	680,1	11,22	4554	455,4	516,3	8,23	4	1	2208	241,4	147,4	551,4	827,0
300	200	8	59,1	75,24	0,966	128	10630	838,4	9389	626,0	757,1	11,17	5042	504,2	574,5	8,19	4	1	2585	268,8	172,2	616,9	925,3
300	200	8,8	64,6	82,24	0,962	117	11590	909,5	10178	678,6	823,8	11,12	5459	545,9	624,9	8,15	3	1	2920	292,5	193,8	674,3	1011
300	200	10	72,7	92,57	0,957	103	12990	1012	11313	754,2	920,9	11,05	6058	605,8	698,1	8,09	2	1	3286	326,9	247,8	758,9	1138
300	200	12,5	88,0	112,0	0,936	84	15770	1204	13179	878,6	1091	10,85	7060	706,0	827,9	7,94	1	1	3978	387,4	293,9	918,6	1378
400	200	6	54,7	69,63	1,179	169	12069	877,1	14789	739,5	906,0	14,57	5092	509,2	562,5	8,55	4	2	1837	321,6	128,2	475,7	951,5
400	200	7,1	63,9	81,45	1,170	144	14169	1020	17070	853,5	1052	14,48	5875	587,5	653,2	8,49	4	1	2321	373,4	162,0	556,5	1113
400	200	8	71,6	91,24	1,166	128	15820	1133	18974	948,7	1173	14,42	6517	651,7	728,1	8,45	4	1	2749	416,5	191,9	623,4	1247
400	200	8,8	78,4	99,84	1,162	116	17260	1231	20619	1031	1279	14,37	7069	706,9	793,1	8,41	4	1	3143	454,1	218,9	682,1	1364
400	200	10	88,4	112,6	1,157	103	19368	1373	23003	1150	1434	14,30	7864	786,4	888,1	8,36	4	1	3749	509,0	259,8	769,1	1538
400	200	12,5	108	137,0	1,136	83	23594	1644	27101	1355	1714	14,06	9260	926,1	1062	8,22	2	1	4865	608,5	377,1	936,3	1873

Table 11.1.3 Cross-sectional properties and resistance values for circular longitudinally welded hollow sections of steel grade S355J2H (f_y = 355 N/mm^2)



M = weight
A = cross-section area
A_u = external area
A_m/V = cross-section factor in fire design
I_t = St. Venant torsional constant
W_t = torsional section modulus

I = second moment of area
W_{el} = elastic section modulus
W_{pl} = plastic section modulus (shall be used only for CL1...2)
i = radius of gyration

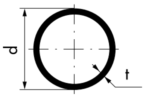
CL = cross-section Class at uniform compression
$N_{c.Rd}$ = design compression resistance (without buckling)
$M_{c.Rd}$ = design bending resistance (respective to cross-section Class)
$V_{pl.Rd}$ = design plastic shear resistance (without shear buckling)

The calculated resistance values are design values (see Chapter 2) based on recommended partial safety factor values **γ_{M0} = 1,0** and **γ_{M1} = 1,0 (for Class 4 circular hollow sections γ_{M1} = 1,1)** as given in Eurocode 3 (EN 1993). Partial safety factor values may differ in each country. National values must be checked from the National Annex of the relevant country.

d mm	t mm	M kg/m	A mm^2 x 10^2	A_u m²/m	A_m/V 1/m	I_t mm^4 x 10^4	W_t mm^3 x 10^3	I mm^4 x 10^4	W_{el} mm^3 x 10^3	W_{pl} mm^3 x 10^3	i mm x 10	CL	$N_{c.Rd}$ kN	$M_{c.Rd}$ kNm	$V_{pl.Rd}$ kN
26,9	2	1,23	1,56	0,085	540	2,44	1,81	1,22	0,91	1,24	0,88	1	55,54	0,44	20,41
26,9	2,5	1,50	1,92	0,085	441	2,88	2,14	1,44	1,07	1,49	0,87	1	68,03	0,53	25,01
26,9	2,6	1,56	1,98	0,085	426	2,96	2,20	1,48	1,10	1,54	0,86	1	70,46	0,55	25,90
33,7	2	1,56	1,99	0,106	532	5,02	2,98	2,51	1,49	2,01	1,12	1	70,71	0,71	25,99
33,7	2,5	1,92	2,45	0,106	432	6,00	3,56	3,00	1,78	2,44	1,11	1	86,99	0,87	31,97
33,7	2,6	1,99	2,54	0,106	417	6,19	3,67	3,09	1,84	2,52	1,10	1	90,18	0,89	33,15
33,7	3	2,27	2,89	0,106	366	6,88	4,08	3,44	2,04	2,84	1,09	1	102,7	1,01	37,75
33,7	3,2	2,41	3,07	0,106	345	7,21	4,28	3,60	2,14	2,99	1,08	1	108,9	1,06	40,01
42,4	2	1,99	2,54	0,133	525	10,38	4,90	5,19	2,45	3,27	1,43	1	90,11	1,16	33,12
42,4	2,5	2,46	3,13	0,133	425	12,52	5,91	6,26	2,95	3,99	1,41	1	111,3	1,41	40,89
42,4	2,6	2,55	3,25	0,133	410	12,93	6,10	6,46	3,05	4,12	1,41	1	115,4	1,46	42,42
42,4	2,9	2,82	3,60	0,133	370	14,11	6,66	7,06	3,33	4,53	1,40	1	127,8	1,61	46,96
42,4	3	2,91	3,71	0,133	359	14,49	6,84	7,25	3,42	4,67	1,40	1	131,8	1,66	48,45
42,4	3,2	3,09	3,94	0,133	338	15,24	7,19	7,62	3,59	4,93	1,39	1	139,9	1,75	51,42
42,4	4	3,79	4,83	0,133	276	17,98	8,48	8,99	4,24	5,92	1,36	1	171,3	2,10	62,96
48,3	2	2,28	2,91	0,152	522	15,62	6,47	7,81	3,23	4,29	1,64	1	103,3	1,52	37,96
48,3	2,5	2,82	3,60	0,152	422	18,92	7,83	9,46	3,92	5,25	1,62	1	127,7	1,86	46,94
48,3	2,6	2,93	3,73	0,152	406	19,55	8,10	9,78	4,05	5,44	1,62	1	132,5	1,93	48,71
48,3	3	3,35	4,27	0,152	355	22,00	9,11	11,00	4,55	6,17	1,61	1	151,6	2,19	55,71
48,3	3,2	3,56	4,53	0,152	335	23,17	9,59	11,59	4,80	6,52	1,60	1	161,0	2,31	59,16
48,3	4	4,37	5,57	0,152	273	27,54	11,40	13,77	5,70	7,87	1,57	1	197,6	2,79	72,64

d mm	t mm	M kg/m	A mm^2 $\times 10^2$	A_u m^2/m	A_m/V 1/m	I_t mm^4 $\times 10^4$	W_t mm^3 $\times 10^3$	I mm^4 $\times 10^4$	W_{el} mm^3 $\times 10^3$	W_{pl} mm^3 $\times 10^3$	i mm x 10	CL	$N_{c.Rd}$ kN	$M_{c.Rd}$ kNm	$V_{pl.Rd}$ kN
60,3	2	2,88	3,66	0,189	517	31,16	10,34	15,58	5,17	6,80	2,06	1	130,0	2,41	47,80
60,3	2,5	3,56	4,54	0,189	417	37,99	12,60	18,99	6,30	8,36	2,05	1	161,2	2,97	59,23
60,3	2,9	4,11	5,23	0,189	362	43,18	14,32	21,59	7,16	9,56	2,03	1	185,7	3,39	68,24
60,3	3	4,24	5,40	0,189	351	44,45	14,74	22,22	7,37	9,86	2,03	1	191,7	3,50	70,47
60,3	3,2	4,51	5,74	0,189	330	46,94	15,57	23,47	7,78	10,44	2,02	1	203,8	3,71	74,90
60,3	4	5,55	7,07	0,189	268	56,35	18,69	28,17	9,34	12,70	2,00	1	251,2	4,51	92,31
60,3	5	6,82	8,69	0,189	218	66,95	22,21	33,48	11,10	15,33	1,96	1	308,4	5,44	113,3
76,1	2	3,65	4,66	0,239	513	63,96	16,81	31,98	8,40	10,98	2,62	2	165,3	3,90	60,75
76,1	2,5	4,54	5,78	0,239	414	78,37	20,60	39,19	10,30	13,55	2,60	1	205,2	4,81	75,43
76,1	2,9	5,24	6,67	0,239	358	89,48	23,52	44,74	11,76	15,55	2,59	1	236,8	5,52	87,02
76,1	3	5,41	6,89	0,239	347	92,19	24,23	46,10	12,11	16,04	2,59	1	244,6	5,69	89,90
76,1	4	7,11	9,06	0,239	264	118,1	31,04	59,06	15,52	20,81	2,55	1	321,6	7,39	118,2
76,1	5	8,77	11,17	0,239	214	141,8	37,28	70,92	18,64	25,32	2,52	1	396,5	8,99	145,7
76,1	6	10,4	13,21	0,239	181	163,5	42,97	81,76	21,49	29,56	2,49	1	469,1	10,49	172,4
76,1	6,3	10,8	13,81	0,239	173	169,6	44,58	84,82	22,29	30,78	2,48	1	490,4	10,93	180,3
88,9	2	4,29	5,46	0,279	512	103,1	23,20	51,57	11,60	15,11	3,07	2	193,8	5,36	71,24
88,9	2,5	5,33	6,79	0,279	412	126,8	28,51	63,37	14,26	18,67	3,06	2	240,9	6,63	88,54
88,9	3	6,36	8,10	0,279	345	149,5	33,64	74,76	16,82	22,15	3,04	1	287,4	7,86	105,6
88,9	3,2	6,76	8,62	0,279	324	158,4	35,64	79,21	17,82	23,51	3,03	1	305,9	8,35	112,4
88,9	4	8,38	10,67	0,279	262	192,7	43,35	96,34	21,67	28,85	3,00	1	378,7	10,24	139,2
88,9	5	10,4	13,18	0,279	212	232,8	52,36	116,4	26,18	35,24	2,97	1	467,9	12,51	172,0
88,9	6	12,3	15,63	0,279	179	269,9	60,72	134,9	30,36	41,31	2,94	1	554,7	14,66	203,9
88,9	6,3	12,8	16,35	0,279	171	280,5	63,10	140,2	31,55	43,07	2,93	1	580,4	15,29	213,3
101,6	2	4,91	6,26	0,319	510	155,3	30,56	77,63	15,28	19,84	3,52	3	222,2	5,43	81,66
101,6	2,5	6,11	7,78	0,319	410	191,2	37,64	95,61	18,82	24,56	3,50	2	276,3	8,72	101,6
101,6	3	7,29	9,29	0,319	343	226,1	44,50	113,0	22,25	29,17	3,49	2	329,9	10,36	121,3
101,6	3,6	8,70	11,08	0,319	288	266,5	52,46	133,2	26,23	34,59	3,47	1	393,5	12,28	144,6
101,6	4	9,63	12,26	0,319	260	292,6	57,59	146,3	28,80	38,12	3,45	1	435,4	13,53	160,0
101,6	5	11,9	15,17	0,319	210	354,9	69,87	177,5	34,93	46,70	3,42	1	538,7	16,58	198,0
101,6	6	14,2	18,02	0,319	177	413,4	81,37	206,7	40,68	54,91	3,39	1	639,7	19,49	235,1
101,6	6,3	14,8	18,86	0,319	169	430,1	84,67	215,1	42,34	57,30	3,38	1	669,6	20,34	246,1
108	2	5,23	6,66	0,339	509	187,2	34,66	93,58	17,33	22,47	3,75	3	236,4	6,15	86,90
108	2,5	6,50	8,29	0,339	409	230,7	42,72	115,4	21,36	27,83	3,73	2	294,2	9,88	108,1
108	3	7,77	9,90	0,339	343	273,0	50,55	136,5	25,28	33,08	3,71	2	351,3	11,74	129,1
108	3,6	9,27	11,81	0,339	287	322,1	59,65	161,1	29,83	39,25	3,69	1	419,2	13,93	154,1
108	4	10,3	13,07	0,339	260	353,9	65,54	177,0	32,77	43,29	3,68	1	464,0	15,37	170,5
108	5	12,7	16,18	0,339	210	430,1	79,65	215,1	39,83	53,09	3,65	1	574,4	18,85	211,1
108	6	15,1	19,23	0,339	176	501,8	92,93	250,9	46,46	62,50	3,61	1	682,5	22,19	250,9
108	6,3	15,8	20,13	0,339	169	522,5	96,75	261,2	48,38	65,24	3,60	1	714,6	23,16	262,6

Table 11.1.3 Cross-sectional properties and resistance values for circular longitudinally welded hollow sections of steel grade S355J2H (f_y = 355 N/mm^2) *cont.*



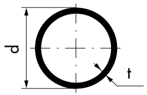
M = weight
A = cross-section area
A_u = external area
A_m/V = cross-section factor in fire design
I_t = St. Venant torsional constant
W_t = torsional section modulus
I = second moment of area
W_{el} = elastic section modulus
W_{pl} = plastic section modulus (shall be used only for CL1...2)
i = radius of gyration
CL = cross-section Class at uniform compression
$N_{c.Rd}$ = design compression resistance (without buckling)
$M_{c.Rd}$ = design bending resistance (respective to cross-section Class)
$V_{pl.Rd}$ = design plastic shear resistance (without shear buckling)

The calculated resistance values are design values (see Chapter 2) based on recommended partial safety factor values **γ_{M0} = 1,0** and **γ_{M1} = 1,0 (for Class 4 circular hollow sections γ_{M1} = 1,1)** as given in Eurocode 3 (EN 1993). Partial safety factor values may differ in each country. National values must be checked from the National Annex of the relevant country.

d mm	t mm	M kg/m	A mm^2 x 10^2	A_u m²/m	A_m/V 1/m	I_t mm^4 x 10^4	W_t mm^3 x 10^3	I mm^4 x 10^4	W_{el} mm^3 x 10^3	W_{pl} mm^3 x 10^3	i mm x 10	CL	$N_{c.Rd}$ kN	$M_{c.Rd}$ kNm	$V_{pl.Rd}$ kN
114,3	2	5,54	7,06	0,359	509	222,5	38,94	111,3	19,47	25,23	3,97	3	250,5	6,91	92,07
114,3	2,5	6,89	8,78	0,359	409	274,5	48,03	137,3	24,02	31,25	3,95	2	311,7	11,09	114,6
114,3	3	8,23	10,49	0,359	342	325,1	56,88	162,6	28,44	37,17	3,94	2	372,4	13,20	136,9
114,3	3,6	9,83	12,52	0,359	287	384,0	67,19	192,0	33,59	44,13	3,92	1	444,5	15,67	163,4
114,3	4	10,9	13,86	0,359	259	422,1	73,86	211,1	36,93	48,69	3,90	1	492,1	17,28	180,9
114,3	5	13,5	17,17	0,359	209	513,8	89,91	256,9	44,96	59,77	3,87	1	609,5	21,22	224,0
114,3	6	16,0	20,41	0,359	176	600,4	105,1	300,2	52,53	70,45	3,83	1	724,7	25,01	266,4
114,3	6,3	16,8	21,38	0,359	168	625,4	109,4	312,7	54,72	73,57	3,82	1	758,8	26,12	278,9
127	2	6,17	7,85	0,399	508	306,9	48,33	153,4	24,16	31,25	4,42	4	223,7	6,88	102,5
127	2,5	7,68	9,78	0,399	408	379,1	59,70	189,5	29,85	38,76	4,40	3	347,1	10,60	127,6
127	3	9,17	11,69	0,399	341	449,5	70,79	224,8	35,39	46,14	4,39	2	414,9	16,38	152,5
127	4	12,1	15,46	0,399	258	585,2	92,16	292,6	46,08	60,54	4,35	1	548,7	21,49	201,7
127	5	15,0	19,16	0,399	208	714,3	112,5	357,1	56,24	74,46	4,32	1	680,3	26,43	250,1
127	6	17,9	22,81	0,399	175	836,9	131,8	418,4	65,90	87,92	4,28	1	809,7	31,21	297,6
127	6,3	18,8	23,89	0,399	167	872,4	137,4	436,2	68,70	91,86	4,27	1	848,1	32,61	311,7
133	2	6,46	8,23	0,418	508	353,2	53,11	176,6	26,56	34,32	4,63	4	232,7	7,51	107,4
133	2,5	8,05	10,25	0,418	408	436,5	65,64	218,3	32,82	42,58	4,61	3	363,9	11,65	133,7
133	3	9,62	12,25	0,418	341	517,9	77,88	259,0	38,94	50,71	4,60	2	435,0	18,00	159,9
133	4	12,7	16,21	0,418	258	675,1	101,5	337,5	50,76	66,59	4,56	2	575,5	23,64	211,5
133	5	15,8	20,11	0,418	208	824,8	124,0	412,4	62,02	81,96	4,53	1	713,8	29,10	262,4
133	6	18,8	23,94	0,418	175	967,4	145,5	483,7	72,74	96,85	4,50	1	849,8	34,38	312,4
133	6,3	19,7	25,08	0,418	167	1009	151,7	504,4	75,85	101,2	4,49	1	890,2	35,93	327,2

d mm	t mm	M kg/m	A mm^2 x 10^2	A_u m^2/m	A_m/V 1/m	I_t mm^4 x 10^4	W_t mm^3 x 10^3	I mm^4 x 10^4	W_{el} mm^3 x 10^3	W_{pl} mm^3 x 10^3	i mm x 10	CL	$N_{c.Rd}$ kN	$M_{c.Rd}$ kNm	$V_{pl.Rd}$ kN
139,7	3	10,1	12,88	0,439	341	602,2	86,21	301,1	43,11	56,07	4,83	3	457,4	15,30	168,1
139,7	4	13,4	17,05	0,439	257	785,7	112,5	392,9	56,24	73,68	4,80	2	605,4	26,16	222,5
139,7	5	16,6	21,16	0,439	207	961,1	137,6	480,5	68,80	90,76	4,77	1	751,1	32,22	276,1
139,7	6	19,8	25,20	0,439	174	1129	161,6	564,3	80,78	107,3	4,73	1	894,7	38,10	328,8
139,7	6,3	20,7	26,40	0,439	166	1177	168,5	588,6	84,27	112,2	4,72	1	937,3	39,83	344,5
139,7	8	26,0	33,10	0,439	133	1441	206,2	720,3	103,1	138,9	4,66	1	1175	49,32	431,9
139,7	10	32,0	40,75	0,439	108	1724	246,8	861,9	123,4	168,6	4,60	1	1447	59,84	531,7
152,4	3	11,1	14,08	0,479	340	786,0	103,2	393,0	51,58	66,97	5,28	3	499,9	18,31	183,7
152,4	4	14,6	18,65	0,479	257	1027	134,8	513,7	67,42	88,11	5,25	2	662,0	31,28	243,3
152,4	5	18,2	23,15	0,479	207	1259	165,2	629,5	82,62	108,7	5,21	1	822,0	38,58	302,1
152,4	6	21,7	27,60	0,479	173	1481	194,4	740,6	97,19	128,7	5,18	1	979,7	45,68	360,1
152,4	6,3	22,7	28,92	0,479	166	1546	202,9	773,0	101,4	134,6	5,17	1	1027	47,77	377,3
159	3	11,5	14,70	0,500	340	894,8	112,6	447,4	56,28	73,02	5,52	3	521,9	19,98	191,8
159	4	15,3	19,48	0,500	256	1171	147,3	585,3	73,63	96,12	5,48	2	691,5	34,12	254,2
159	5	19,0	24,19	0,500	206	1436	180,6	717,9	90,30	118,6	5,45	1	858,8	42,11	315,6
159	6	22,6	28,84	0,500	173	1690	212,6	845,2	106,3	140,5	5,41	1	1024	49,89	376,3
159	6,3	23,7	30,22	0,500	165	1765	222,0	882,4	111,0	147,0	5,40	1	1073	52,18	394,4
168,3	3	12,2	15,58	0,529	339	1065	126,5	532,3	63,25	81,98	5,85	3	553,1	22,46	203,3
168,3	3,2	13,0	16,60	0,529	319	1131	134,5	565,7	67,23	87,24	5,84	3	589,2	23,87	216,6
168,3	4	16,2	20,65	0,529	256	1394	165,7	697,1	82,84	108,0	5,81	2	733,0	38,34	269,4
168,3	4,5	18,2	23,16	0,529	228	1554	184,7	777,2	92,36	120,8	5,79	2	822,1	42,87	302,2
168,3	5	20,1	25,65	0,529	206	1712	203,4	855,9	101,7	133,4	5,78	2	910,6	47,35	334,7
168,3	6	24,0	30,59	0,529	173	2017	239,7	1009	119,9	158,1	5,74	1	1086	56,13	399,2
168,3	6,3	25,2	32,06	0,529	165	2107	250,4	1053	125,2	165,4	5,73	1	1138	58,72	418,4
168,3	8	31,6	40,29	0,529	131	2595	308,3	1297	154,2	205,7	5,67	1	1430	73,04	525,7
168,3	10	39,0	49,73	0,529	106	3128	371,7	1564	185,9	250,9	5,61	1	1765	89,08	648,9
193,7	4	18,7	23,84	0,609	255	2146	221,5	1073	110,8	144,0	6,71	3	846,3	39,32	311,1
193,7	5	23,3	29,64	0,609	205	2640	272,6	1320	136,3	178,1	6,67	2	1052	63,22	386,8
193,7	6	27,8	35,38	0,609	172	3119	322,1	1560	161,1	211,5	6,64	1	1256	75,07	461,7
193,7	6,3	29,1	37,09	0,609	164	3260	336,6	1630	168,3	221,3	6,63	1	1317	78,57	484,0
193,7	8	36,6	46,67	0,609	130	4031	416,2	2016	208,1	276,1	6,57	1	1657	98,00	609,0
193,7	10	45,3	57,71	0,609	105	4883	504,2	2442	252,1	337,8	6,50	1	2049	119,9	753,0
193,7	12,5	55,9	71,16	0,609	86	5869	606,0	2934	303,0	411,1	6,42	1	2526	145,9	928,5

Table 11.1.3 Cross-sectional properties and resistance values for circular longitudinally welded hollow sections of steel grade S355J2H (f_y = 355 N/mm²) *cont.*

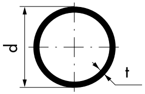


M = weight
A = cross-section area
A_u = external area
A_m/V = cross-section factor in fire design
I_t = St. Venant torsional constant
W_t = torsional section modulus
I = second moment of area
W_{el} = elastic section modulus
W_{pl} = plastic section modulus (shall be used only for CL1...2)
i = radius of gyration
CL = cross-section Class at uniform compression
$N_{c.Rd}$ = design compression resistance (without buckling)
$M_{c.Rd}$ = design bending resistance (respective to cross-section Class)
$V_{pl.Rd}$ = design plastic shear resistance (without shear buckling)

The calculated resistance values are design values (see Chapter 2) based on recommended partial safety factor values **γ_{M0} = 1,0** and **γ_{M1} = 1,0 (for Class 4 circular hollow sections γ_{M1} = 1,1)** as given in Eurocode 3 (EN 1993). Partial safety factor values may differ in each country. National values must be checked from the National Annex of the relevant country.

d mm	t mm	M kg/m	A mm² x 10²	A_u m²/m	A_m/V 1/m	I_t mm⁴ x 10⁴	W_t mm³ x 10³	I mm⁴ x 10⁴	W_{el} mm³ x 10³	W_{pl} mm³ x 10³	i mm x 10	CL	$N_{c.Rd}$ kN	$M_{c.Rd}$ kNm	$V_{pl.Rd}$ kN
219,1	4	21,2	27,03	0,688	255	3128	285,5	1564	142,8	185,1	7,61	3	959,6	50,68	352,7
219,1	4,5	23,8	30,34	0,688	227	3494	319,0	1747	159,5	207,3	7,59	3	1077	56,62	395,9
219,1	5	26,4	33,63	0,688	205	3856	352,0	1928	176,0	229,2	7,57	2	1194	81,38	438,8
219,1	6	31,5	40,17	0,688	171	4564	416,6	2282	208,3	272,5	7,54	2	1426	96,75	524,1
219,1	6,3	33,1	42,12	0,688	163	4772	435,6	2386	217,8	285,4	7,53	2	1495	101,3	549,6
219,1	8	41,7	53,06	0,688	130	5919	540,3	2960	270,2	356,7	7,47	1	1883	126,6	692,3
219,1	10	51,6	65,69	0,688	105	7197	657,0	3598	328,5	437,6	7,40	1	2332	155,3	857,1
219,1	12,5	63,7	81,13	0,688	85	8689	793,2	4345	396,6	534,2	7,32	1	2880	189,6	1059
244,5	6	35,3	44,96	0,768	171	6397	523,3	3199	261,6	341,4	8,43	2	1596	121,2	586,6
244,5	8	46,7	59,44	0,768	129	8321	680,7	4160	340,3	447,6	8,37	1	2110	158,9	775,6
244,5	10	57,8	73,67	0,768	104	10150	830,0	5073	415,0	550,2	8,30	1	2615	195,3	961,3
244,5	12,5	71,5	91,11	0,768	84	12290	1006	6147	502,9	673,5	8,21	1	3234	239,1	1189
273	4	26,5	33,80	0,858	254	6117	448,1	3058	224,1	289,5	9,51	4	951,9	63,09	441,1
273	5	33,1	42,10	0,858	204	7562	554,0	3781	277,0	359,2	9,48	3	1494	98,33	549,3
273	6	39,5	50,33	0,858	170	8974	657,5	4487	328,7	427,8	9,44	2	1787	151,9	656,7
273	6,3	41,4	52,79	0,858	162	9392	688,0	4696	344,0	448,2	9,43	2	1874	159,1	688,8
273	8	52,3	66,60	0,858	129	11703	857,4	5852	428,7	562,0	9,37	2	2364	199,5	869,0
273	10	64,9	82,62	0,858	104	14308	1048	7154	524,1	692,0	9,31	1	2933	245,7	1078
273	12,5	80,3	102,3	0,858	84	17395	1274	8697	637,2	848,9	9,22	1	3632	301,4	1335
323,9	4	31,6	40,20	1,018	253	10286	635,2	5143	317,6	409,4	11,31	4	1099	86,80	524,5
323,9	5	39,3	50,09	1,018	203	12739	786,6	6369	393,3	508,5	11,28	4	1422	111,7	653,6
323,9	6	47,0	59,92	1,018	170	15145	935,2	7572	467,6	606,4	11,24	3	2127	166,0	781,9
323,9	6,3	49,3	62,86	1,018	162	15858	979,2	7929	489,6	635,6	11,23	3	2232	173,8	820,2
323,9	8	62,3	79,39	1,018	128	19820	1224	9910	611,9	798,5	11,17	2	2819	283,5	1036
323,9	10	77,4	98,61	1,018	103	24317	1501	12158	750,8	985,7	11,10	1	3501	349,9	1287
323,9	12,5	96,0	122,3	1,018	83	29693	1833	14847	916,7	1213	11,02	1	4341	430,5	1596

Table 11.1.4 Cross-sectional properties and resistance values for circular spirally welded hollow sections of steel grade S355J2H (f_y = 355 N/mm^2)
(Technical delivery conditions to be agreed when ordering)



M = weight
A = cross-section area
A_u = external area
A_m/V = cross-section factor in fire design
I_t = St. Venant torsional constant
W_t = torsional section modulus

I = second moment of area
W_{el} = elastic section modulus
W_{pl} = plastic section modulus (shall be used only for CL1...2)
i = radius of gyration

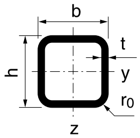
CL = cross-section Class at uniform compression
$N_{c.Rd}$ = design compression resistance (without buckling)
$M_{c.Rd}$ = design bending resistance (respective to cross-section Class)
$V_{pl.Rd}$ = design plastic shear resistance (without shear buckling)

The calculated resistance values are design values (see Chapter 2) based on recommended partial safety factor values **γ_{M0} = 1,0** and **γ_{M1} = 1,0 (for Class 4 circular hollow sections γ_{M1} = 1,1)** as given in Eurocode 3 (EN 1993). Partial safety factor values may differ in each country. National values must be checked from the National Annex of the relevant country.

d mm	t mm	M kg/m	A mm^2 x 10^2	A_u m²/m	A_m/V 1/m	I_t mm^4 x 10^4	W_t mm^3 x 10^3	I mm^4 x 10^4	W_{el} mm^3 x 10^3	W_{pl} mm^3 x 10^3	i mm x 10	CL	$N_{c.Rd}$ kN	$M_{c.Rd}$ kNm	$V_{pl.Rd}$ kN
406,4	6,3	62,2	79,19	1,277	161	31699	1560	15849	780,0	1009	14,15	4	2250	221,6	1033
406,4	8	78,6	100,1	1,277	128	39748	1956	19874	978,1	1270	14,09	3	3555	347,2	1306
406,4	10	97,8	124,5	1,277	103	48952	2409	24476	1205	1572	14,02	2	4421	557,9	1625
406,4	12,5	121	154,7	1,277	83	60061	2956	30031	1478	1940	13,93	1	5491	688,7	2018
457	6,3	70,0	89,20	1,436	161	45308	1983	22654	991,4	1280	15,94	4	2487	276,4	1164
457	8	88,6	112,9	1,436	127	56893	2490	28446	1245	1613	15,88	3	4006	442,0	1472
457	10	110	140,4	1,436	102	70183	3071	35091	1536	1998	15,81	2	4985	709,4	1832
457	12	132	167,8	1,436	86	83113	3637	41556	1819	2377	15,74	2	5956	843,8	2189
457	12,5	137	174,6	1,436	82	86290	3776	43145	1888	2470	15,72	2	6197	877,0	2278
508	6,3	78,0	99,30	1,596	161	62493	2460	31246	1230	1586	17,74	4	2716	336,5	1296
508	8	98,7	125,7	1,596	127	78560	3093	39280	1546	2000	17,68	4	3579	440,4	1640
508	10	123	156,5	1,596	102	97040	3820	48520	1910	2480	17,61	3	5554	678,1	2041
508	12,5	153	194,6	1,596	82	119511	4705	59755	2353	3070	17,52	2	6908	1090	2539
559	6,3	85,9	109,4	1,756	161	83552	2989	41776	1495	1925	19,54	4	2937	401,3	1427
559	8	109	138,5	1,756	127	105130	3761	52565	1881	2429	19,48	4	3885	527,5	1807
559	10	135	172,5	1,756	102	130002	4651	65001	2326	3014	19,41	3	6123	825,6	2250
559	12,5	168	214,6	1,756	82	160324	5736	80162	2868	3734	19,33	2	7619	1326	2800
610	8	119	151,3	1,916	127	137103	4495	68551	2248	2899	21,29	4	4181	621,1	1974
610	10	148	188,5	1,916	102	169693	5564	84847	2782	3600	21,22	4	5401	797,1	2460
610	12,5	184	234,6	1,916	82	209509	6869	104755	3435	4463	21,13	3	8330	1219	3062
610	14,2	209	265,8	1,916	72	236008	7738	118004	3869	5042	21,07	2	9436	1790	3468

d	t	M	A	A_u	A_m/V	I_t	W_t	I	W_{el}	W_{pl}	i	CL	$N_{c.Rd}$	$M_{c.Rd}$	$V_{pl.Rd}$
mm	mm	kg/m	mm^2	m²/m	1/m	mm^4	mm^3	mm^4	mm^3	mm^3	mm		kN	kNm	kN
			$x\ 10^2$			$x\ 10^4$	$x\ 10^3$	$x\ 10^4$	$x\ 10^3$	$x\ 10^3$	x 10				
660	8	129	163,9	2,073	127	174176	5278	87088	2639	3401	23,05	4	4463	718,8	2138
660	10	160	204,2	2,073	102	215741	6538	107870	3269	4225	22,98	4	5781	925,4	2664
660	12,5	200	254,3	2,073	82	266613	8079	133306	4040	5241	22,90	3	9027	1434	3318
660	14,2	226	288,1	2,073	72	300526	9107	150263	4553	5923	22,84	3	10227	1616	3759
711	8	139	176,7	2,234	126	218324	6141	109162	3071	3954	24,86	4	4742	824,2	2305
711	10	173	220,2	2,234	101	270603	7612	135301	3806	4914	24,79	4	6160	1065	2874
711	12,5	215	274,3	2,234	81	334686	9415	167343	4707	6099	24,70	3	9738	1671	3579
711	14,2	244	310,9	2,234	72	377470	10618	188735	5309	6895	24,64	3	11035	1885	4056
762	8	149	189,5	2,394	126	269366	7070	134683	3535	4548	26,66	4	5013	935,1	2473
762	10	185	236,3	2,394	101	334057	8768	167028	4384	5655	26,59	4	6529	1212	3083
762	12,5	231	294,3	2,394	81	413462	10852	206731	5426	7023	26,50	4	8434	1555	3840
762	14,2	262	333,6	2,394	72	466542	12245	233271	6123	7942	26,44	3	11843	2174	4353
813	8	159	202,3	2,554	126	327801	8064	163901	4032	5184	28,46	4	5275	1051	2640
813	10	198	252,3	2,554	101	406728	10006	203364	5003	6448	28,39	4	6890	1366	3292
813	12,5	247	314,4	2,554	81	503721	12392	251860	6196	8011	28,31	4	8920	1758	4102
813	14,2	280	356,4	2,554	72	568630	13988	284315	6994	9062	28,25	3	12650	2483	4650
813	16	314	400,6	2,554	64	636443	15657	318222	7828	10165	28,18	3	14222	2779	5227
914	10	223	284,0	2,871	101	580294	12698	290147	6349	8172	31,96	4	7579	1694	3706
914	12,5	278	354,0	2,871	81	719417	15742	359708	7871	10159	31,88	4	9855	2191	4619
914	14,2	315	401,4	2,871	72	812689	17783	406344	8892	11498	31,82	4	11409	2527	5238
914	16	354	451,4	2,871	64	910283	19919	455142	9959	12904	31,75	3	16024	3536	5890
1016	10	248	316,0	3,192	101	799699	15742	399850	7871	10121	35,57	4	8241	2052	4124
1016	12,5	309	394,1	3,192	81	992246	19532	496123	9766	12588	35,48	4	10763	2667	5142
1016	14,2	351	446,9	3,192	71	1121520	22077	560762	11039	14252	35,42	4	12487	3084	5831
1016	16	395	502,7	3,192	64	1256959	24743	628479	12372	16001	35,36	4	14316	3524	6559
1219	10	298	379,8	3,830	101	1388029	22773	694014	11387	14617	42,75	4	9461	2836	4956
1219	12,5	372	473,8	3,830	81	1724362	28291	862181	14146	18196	42,66	4	12469	3723	6182
1219	14,2	422	537,5	3,830	71	1950668	32004	975334	16002	20613	42,60	4	14526	4325	7013
1219	16	475	604,7	3,830	63	2188182	35901	1094091	17951	23157	42,54	4	16712	4961	7890

Table 11.1.5 Cross-sectional properties and resistance values for square hollow sections of steel grade S420MH (f_y = 420 N/mm^2)



M = weight
A = cross-section area
A_u = external area
A_m/V = cross-section factor in fire design
I_t = St. Venant torsional constant
W_t = torsional section modulus

I = second moment of area
W_{el} = elastic section modulus
W_{pl} = plastic section modulus (shall be used only for CL1...2)
i = radius of gyration

CL = cross-section Class at uniform compression
$N_{c.Rd}$ = design compression resistance (without buckling)
$M_{c.Rd}$ = design bending resistance (respective to cross-section Class)
$V_{pl.Rd}$ = design plastic shear resistance (without shear buckling)

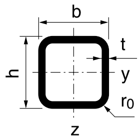
$r_0 = 2{,}0 \times t$ when $t \leq 6{,}0$ mm
$r_0 = 2{,}5 \times t$ when $6{,}0$ mm $< t \leq 10{,}0$ mm
$r_0 = 3{,}0 \times t$ when $t > 10{,}0$ mm

The calculated resistance values are design values (see Chapter 2) based on recommended partial safety factor values **γ_{M0} = 1,0** and **γ_{M1} = 1,0 (for Class 4 circular hollow sections γ_{M1} = 1,1)** as given in Eurocode 3 (EN 1993). Partial safety factor values may differ in each country. National values must be checked from the National Annex of the relevant country.

h mm	b mm	t mm	M kg/m	A mm^2 x 10^2	A_u m^2/m	A_m/V 1/m	I_t mm^4 x 10^4	W_t mm^3 x 10^3	I mm^4 x 10^4	W_{el} mm^3 x 10^3	W_{pl} mm^3 x 10^3	i mm x 10	CL	$N_{c.Rd}$ kN	$M_{c.Rd}$ kNm	$V_{pl.Rd}$ kN
25	25	2	1,36	1,74	0,093	536	2,53	1,80	1,48	1,19	1,47	0,92	1	72,95	0,62	21,06
25	25	2,5	1,64	2,09	0,091	438	2,97	2,07	1,69	1,35	1,71	0,90	1	87,74	0,72	25,33
25	25	3	1,89	2,41	0,090	372	3,33	2,27	1,84	1,47	1,91	0,87	1	101,2	0,80	29,20
30	30	2	1,68	2,14	0,113	529	4,54	2,75	2,72	1,82	2,21	1,13	1	89,75	0,93	25,91
30	30	2,5	2,03	2,59	0,111	430	5,40	3,20	3,16	2,10	2,61	1,10	1	108,7	1,10	31,39
30	30	3	2,36	3,01	0,110	365	6,15	3,58	3,50	2,34	2,96	1,08	1	126,4	1,24	36,47
40	40	2	2,31	2,94	0,153	521	11,28	5,23	6,94	3,47	4,13	1,54	1	123,4	1,74	35,61
40	40	2,5	2,82	3,59	0,151	422	13,61	6,21	8,22	4,11	4,97	1,51	1	150,7	2,09	43,51
40	40	3	3,30	4,21	0,150	356	15,75	7,07	9,32	4,66	5,72	1,49	1	176,8	2,40	51,02
40	40	4	4,20	5,35	0,146	273	19,44	8,48	11,07	5,54	7,01	1,44	1	224,6	2,95	64,84
50	50	2	2,93	3,74	0,193	517	22,63	8,51	14,15	5,66	6,66	1,95	1	157,0	2,80	45,31
50	50	2,5	3,60	4,59	0,191	417	27,53	10,22	16,94	6,78	8,07	1,92	1	192,7	3,39	55,64
50	50	3	4,25	5,41	0,190	351	32,13	11,76	19,47	7,79	9,39	1,90	1	227,2	3,94	65,57
50	50	4	5,45	6,95	0,186	268	40,42	14,43	23,74	9,49	11,73	1,85	1	291,8	4,93	84,24
50	50	5	6,56	8,36	0,183	219	47,46	16,56	27,04	10,82	13,70	1,80	1	351,0	5,75	101,3
60	60	2	3,56	4,54	0,233	514	39,79	12,59	25,14	8,38	9,79	2,35	2	190,6	4,11	55,01
60	60	2,5	4,39	5,59	0,231	414	48,66	15,22	30,34	10,11	11,93	2,33	1	234,7	5,01	67,76
60	60	3	5,19	6,61	0,230	348	57,09	17,65	35,13	11,71	13,95	2,31	1	277,6	5,86	80,12
60	60	4	6,71	8,55	0,226	265	72,64	21,97	43,55	14,52	17,64	2,26	1	359,0	7,41	103,6
60	60	5	8,13	10,36	0,223	215	86,42	25,61	50,49	16,83	20,88	2,21	1	435,0	8,77	125,6
70	70	2,5	5,17	6,59	0,271	412	78,49	21,22	49,41	14,12	16,54	2,74	2	276,7	6,95	79,89
70	70	3	6,13	7,81	0,270	345	92,42	24,74	57,53	16,44	19,42	2,71	1	328,0	8,15	94,67
70	70	4	7,97	10,15	0,266	262	118,5	31,11	72,12	20,61	24,76	2,67	1	426,2	10,40	123,0
70	70	5	9,70	12,36	0,263	213	142,2	36,65	84,63	24,18	29,56	2,62	1	519,0	12,41	149,8

h mm	b mm	t mm	M kg/m	A mm^2 x 10^2	A_u m^2/m	A_m/V 1/m	I_t mm^4 x 10^4	W_t mm^3 x 10^3	I mm^4 x 10^4	W_{el} mm^3 x 10^3	W_{pl} mm^3 x 10^3	i mm x 10	CL	$N_{c.Rd}$ kN	$M_{c.Rd}$ kNm	$V_{pl.Rd}$ kN
80	80	2,5	5,96	7,59	0,311	410	118,5	28,22	75,15	18,79	21,90	3,15	3	318,7	7,89	92,01
80	80	3	7,07	9,01	0,310	344	139,9	33,02	87,84	21,96	25,78	3,12	1	378,4	10,83	109,2
80	80	4	9,22	11,75	0,306	261	180,4	41,84	111,0	27,76	33,07	3,07	1	493,4	13,89	142,4
80	80	5	11,3	14,36	0,303	211	217,8	49,68	131,4	32,86	39,74	3,03	1	603,0	16,69	174,1
80	80	6	13,2	16,83	0,299	178	252,1	56,59	149,2	37,29	45,79	2,98	1	707,0	19,23	204,1
90	90	2,5	6,74	8,59	0,351	409	170,3	36,23	108,6	24,12	28,00	3,56	4	334,0	9,66	104,1
90	90	3	8,01	10,21	0,350	343	201,4	42,51	127,3	28,29	33,04	3,53	2	428,8	13,88	123,8
90	90	4	10,5	13,35	0,346	259	260,8	54,17	161,9	35,98	42,58	3,48	1	560,6	17,89	161,8
90	90	5	12,8	16,36	0,343	210	316,3	64,70	192,9	42,87	51,41	3,43	1	687,0	21,59	198,3
90	90	6	15,1	19,23	0,339	176	367,8	74,16	220,5	49,00	59,54	3,39	1	807,8	25,01	233,2
100	100	2,5	7,53	9,59	0,391	408	235,2	45,23	150,6	30,13	34,86	3,96	4	347,7	11,58	116,3
100	100	3	8,96	11,41	0,390	342	278,7	53,19	177,0	35,41	41,21	3,94	3	479,2	14,87	138,3
100	100	4	11,7	14,95	0,386	258	362,0	68,10	226,4	45,27	53,30	3,89	1	627,8	22,39	181,2
100	100	5	14,4	18,36	0,383	209	440,5	81,72	271,1	54,22	64,59	3,84	1	771,0	27,13	222,6
100	100	6	17,0	21,63	0,379	175	514,2	94,12	311,5	62,29	75,10	3,79	1	908,6	31,54	262,3
100	100	7,1	19,4	24,65	0,370	150	589,2	105,6	340,1	68,03	83,59	3,71	1	1035	35,11	298,9
100	100	8	21,4	27,24	0,366	134	644,5	114,2	365,9	73,19	91,05	3,67	1	1144	38,24	330,3
100	100	10	25,6	32,57	0,357	110	749,8	130,1	411,1	82,22	105,3	3,55	1	1368	44,20	394,9
110	110	2,5	8,31	10,59	0,431	407	314,9	55,23	202,4	36,80	42,47	4,37	4	358,7	13,59	128,4
110	110	3	9,90	12,61	0,430	341	373,5	65,07	238,3	43,33	50,27	4,35	4	484,6	17,24	152,9
110	110	4	13,0	16,55	0,426	258	486,5	83,63	305,9	55,62	65,21	4,30	1	695,0	27,39	200,6
110	110	5	16,0	20,36	0,423	208	593,6	100,7	367,9	66,90	79,27	4,25	1	855,0	33,29	246,8
110	110	6	18,9	24,03	0,419	175	694,9	116,5	424,6	77,19	92,46	4,20	1	1009	38,83	291,4
120	120	3	10,8	13,81	0,470	340	487,7	78,15	312,3	52,06	60,24	4,76	4	501,0	20,00	167,4
120	120	4	14,2	18,15	0,466	257	636,6	100,8	402,3	67,05	78,33	4,71	2	762,2	32,90	220,0
120	120	5	17,5	22,36	0,463	207	778,5	121,8	485,5	80,91	95,45	4,66	1	939,0	40,09	271,1
120	120	5,6	19,5	24,82	0,461	186	860,3	133,6	532,3	88,71	105,3	4,63	1	1042	44,21	300,9
120	120	6	20,7	26,43	0,459	174	913,5	141,2	562,2	93,69	111,6	4,61	1	1110	46,88	320,5
120	120	7,1	23,8	30,33	0,450	148	1056	160,1	623,5	103,9	125,7	4,53	1	1274	52,79	367,8
120	120	8	26,4	33,64	0,446	132	1163	174,6	676,9	112,8	137,8	4,49	1	1413	57,88	407,9
120	120	8,8	28,6	36,48	0,442	121	1252	186,5	719,9	120,0	147,9	4,44	1	1532	62,12	442,3
120	120	10	31,8	40,57	0,437	108	1376	202,5	776,8	129,5	161,8	4,38	1	1704	67,96	491,8
140	140	4	16,8	21,35	0,546	256	1023	139,8	651,6	93,09	108,2	5,52	4	844,9	37,69	258,8
140	140	5	20,7	26,36	0,543	206	1256	169,8	790,6	112,9	132,3	5,48	2	1107	55,57	319,6
140	140	5,6	23,0	29,30	0,541	185	1391	186,9	869,6	124,2	146,3	5,45	1	1231	61,43	355,2
140	140	6	24,5	31,23	0,539	173	1479	197,9	920,4	131,5	155,3	5,43	1	1312	65,24	378,7
140	140	7,1	28,3	36,01	0,530	147	1719	226,0	1032	147,4	176,3	5,35	1	1513	74,05	436,6
140	140	8	31,4	40,04	0,526	131	1901	247,7	1127	161,0	194,2	5,30	1	1682	81,55	485,5
140	140	8,8	34,2	43,52	0,522	120	2055	265,8	1205	172,1	209,2	5,26	1	1828	87,88	527,7
140	140	10	38,1	48,57	0,517	106	2274	290,9	1312	187,4	230,4	5,20	1	2040	96,76	588,8

Table 11.1.5 Cross-sectional properties and resistance values for square hollow sections of steel grade S420MH (f_y = 420 N/mm^2) *continued*



M = weight
A = cross-section area
A_u = external area
A_m/V = cross-section factor in fire design
I_t = St. Venant torsional constant
W_t = torsional section modulus

I = second moment of area
W_{el} = elastic section modulus
W_{pl} = plastic section modulus (shall be used only for CL1...2)
i = radius of gyration

CL = cross-section Class at uniform compression
$N_{c.Rd}$ = design compression resistance (without buckling)
$M_{c.Rd}$ = design bending resistance (respective to cross-section Class)
$V_{pl.Rd}$ = design plastic shear resistance (without shear buckling)

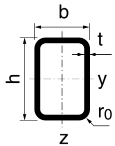
r_0 = 2,0 x t when t ≤ 6,0 mm
r_0 = 2,5 x t when 6,0 mm < t ≤ 10,0 mm
r_0 = 3,0 x t when t > 10,0 mm

The calculated resistance values are design values (see Chapter 2) based on recommended partial safety factor values **γ_{M0} = 1,0** and **γ_{M1} = 1,0 (for Class 4 circular hollow sections γ_{M1} = 1,1)** as given in Eurocode 3 (EN 1993). Partial safety factor values may differ in each country. National values must be checked from the National Annex of the relevant country.

h mm	b mm	t mm	M kg/m	A mm^2 x 10^2	A_u m^2/m	A_m/V 1/m	I_t mm^4 x 10^4	W_t mm^3 x 10^3	I mm^4 x 10^4	W_{el} mm^3 x 10^3	W_{pl} mm^3 x 10^3	i mm x 10	CL	$N_{c.Rd}$ kN	$M_{c.Rd}$ kNm	$V_{pl.Rd}$ kN
150	150	4	18,0	22,95	0,586	255	1265	161,7	807,8	107,7	124,9	5,93	4	869,1	42,47	278,2
150	150	5	22,3	28,36	0,583	206	1554	196,8	982,1	130,9	153,0	5,89	2	1191	64,25	343,8
150	150	6	26,4	33,63	0,579	172	1833	229,8	1146	152,8	179,9	5,84	1	1413	75,55	407,8
150	150	7,1	30,5	38,85	0,570	147	2134	263,2	1290	172,0	204,8	5,76	1	1632	86,03	471,1
150	150	8	33,9	43,24	0,566	131	2364	289,0	1412	188,2	226,0	5,71	1	1816	94,90	524,3
150	150	8,8	36,9	47,04	0,562	120	2560	310,7	1513	201,7	243,9	5,67	1	1976	102,4	570,4
150	150	10	41,3	52,57	0,557	106	2839	341,0	1653	220,3	269,2	5,61	1	2208	113,0	637,3
150	150	12,5	48,7	62,04	0,536	86	3321	389,3	1817	242,3	305,6	5,41	1	2606	128,3	752,2
160	160	4	19,3	24,55	0,626	255	1542	185,3	987,2	123,4	142,8	6,34	4	890,0	47,41	297,6
160	160	5	23,8	30,36	0,623	205	1896	225,8	1202	150,3	175,2	6,29	3	1275	63,12	368,1
160	160	6	28,3	36,03	0,619	172	2239	264,2	1405	175,7	206,2	6,25	1	1513	86,62	436,9
160	160	7,1	32,7	41,69	0,610	146	2611	303,2	1587	198,4	235,5	6,17	1	1751	98,89	505,5
160	160	8	36,5	46,44	0,606	130	2897	333,6	1741	217,7	260,1	6,12	1	1951	109,3	563,1
160	160	8,8	39,7	50,56	0,602	119	3141	359,2	1870	233,7	281,1	6,08	1	2124	118,1	613,1
160	160	10	44,4	56,57	0,597	106	3490	395,1	2048	256,0	311,0	6,02	1	2376	130,6	685,8
160	160	12,5	52,6	67,04	0,576	86	4114	454,6	2275	284,4	355,7	5,83	1	2816	149,4	812,9
180	180	5	27,0	34,36	0,703	205	2724	289,8	1737	193,0	224,0	7,11	4	1336	77,32	416,6
180	180	6	32,1	40,83	0,699	171	3223	340,1	2037	226,3	264,4	7,06	2	1715	111,0	495,1
180	180	7,1	37,2	47,37	0,690	146	3768	391,7	2313	257,0	303,1	6,99	1	1990	127,3	574,4
180	180	8	41,5	52,84	0,686	130	4189	432,2	2546	282,9	335,7	6,94	1	2219	141,0	640,7
180	180	8,8	45,2	57,60	0,682	118	4551	466,6	2742	304,6	363,6	6,90	1	2419	152,7	698,4
180	180	10	50,7	64,57	0,677	105	5074	515,3	3017	335,2	403,5	6,84	1	2712	169,5	782,8
180	180	12,5	60,5	77,04	0,656	85	6050	600,1	3406	378,5	467,1	6,65	1	3236	196,2	934,1

h mm	b mm	t mm	M kg/m	A mm^2 x 10^2	A_u m^2/m	A_m/V 1/m	I_t mm^4 x 10^4	W_t mm^3 x 10^3	I mm^4 x 10^4	W_{el} mm^3 x 10^3	W_{pl} mm^3 x 10^3	i mm x 10	CL	$N_{c.Rd}$ kN	$M_{c.Rd}$ kNm	$V_{pl.Rd}$ kN
200	200	5	30,1	38,36	0,783	204	3763	361,8	2410	241,0	278,9	7,93	4	1391	92,60	465,0
200	200	6	35,8	45,63	0,779	171	4459	425,5	2833	283,3	329,7	7,88	3	1917	119,0	553,3
200	200	7,1	41,6	53,05	0,770	145	5223	491,6	3232	323,2	379,3	7,81	2	2228	159,3	643,2
200	200	8	46,5	59,24	0,766	129	5815	543,6	3566	356,6	420,9	7,76	1	2488	176,8	718,3
200	200	8,8	50,7	64,64	0,762	118	6328	588,1	3850	385,0	456,6	7,72	1	2715	191,8	783,8
200	200	10	57,0	72,57	0,757	104	7072	651,5	4251	425,1	508,1	7,65	1	3048	213,4	879,8
200	200	12,5	68,3	87,04	0,736	85	8502	765,5	4859	485,9	593,5	7,47	1	3656	249,3	1055
220	220	6	39,6	50,43	0,859	170	5976	520,6	3813	346,7	402,2	8,70	4	1938	137,9	611,5
220	220	7,1	46,1	58,73	0,850	145	7010	602,9	4367	397,0	464,0	8,62	2	2467	194,9	712,1
220	220	8	51,5	65,64	0,846	129	7815	667,9	4828	438,9	515,6	8,58	1	2757	216,6	795,9
220	220	8,8	56,3	71,68	0,842	117	8514	723,6	5221	474,7	560,2	8,53	1	3011	235,3	869,1
220	220	10	63,2	80,57	0,837	104	9533	803,6	5782	525,7	624,7	8,47	1	3384	262,4	976,8
220	220	12,5	76,2	97,04	0,816	84	11530	950,8	6674	606,7	734,9	8,29	1	4076	308,7	1177
250	250	6	45,2	57,63	0,979	170	8843	681,2	5672	453,8	524,5	9,92	4	2030	171,5	698,8
250	250	7,1	52,8	67,25	0,970	144	10387	791,0	6523	521,8	607,1	9,85	4	2651	210,8	815,4
250	250	8	59,1	75,24	0,966	128	11598	878,2	7229	578,3	675,8	9,80	2	3160	283,8	912,3
250	250	8,8	64,6	82,24	0,962	117	12653	953,3	7835	626,8	735,3	9,76	2	3454	308,8	997,2
250	250	10	72,7	92,57	0,957	103	14197	1062	8707	696,5	822,0	9,70	1	3888	345,2	1122
250	250	12,5	88,0	112,0	0,936	84	17283	1266	10161	812,9	975,2	9,52	1	4706	409,6	1358
260	260	6	47,1	60,03	1,019	170	9970	739,5	6405	492,7	568,8	10,33	4	2056	183,1	727,9
260	260	7,1	55,0	70,09	1,010	144	11716	859,4	7374	567,2	659,0	10,26	4	2694	225,6	849,8
260	260	8	61,6	78,44	1,006	128	13087	954,7	8178	629,1	734,0	10,21	3	3295	264,2	951,1
260	260	8,8	67,3	85,76	1,002	117	14283	1037	8869	682,2	799,0	10,17	2	3602	335,6	1040
260	260	10	75,8	96,57	0,997	103	16035	1156	9865	758,8	893,8	10,11	1	4056	375,4	1171
260	260	12,5	91,9	117,0	0,976	83	19553	1381	11548	888,3	1063	9,93	1	4916	446,4	1419
300	300	6	54,7	69,63	1,179	169	15434	996,8	9964	664,2	764,2	11,96	4	2141	232,4	844,3
300	300	7,1	63,9	81,45	1,170	144	18160	1161	11516	767,7	888,0	11,89	4	2838	288,2	987,6
300	300	8	71,6	91,24	1,166	128	20312	1293	12801	853,4	990,7	11,84	4	3453	336,3	1106
300	300	8,8	78,4	99,84	1,162	116	22195	1406	13911	927,4	1080	11,80	3	4193	389,5	1211
300	300	10	88,4	112,6	1,157	103	24966	1572	15519	1035	1211	11,74	2	4728	508,6	1365
300	300	12,5	108,0	137,0	1,136	83	30601	1892	18348	1223	1451	11,57	1	5756	609,3	1662

Table 11.1.6 Cross-sectional properties and resistance values for rectangular hollow sections of steel grade S420MH (f_y = 420 N/mm^2)



M = weight
A = cross-section area
A_u = external area
A_m/V = cross-section factor in fire design
I_t = St. Venant torsional constant
W_t = torsional section modulus

I = second moment of area
W_{el} = elastic section modulus
W_{pl} = plastic section modulus (shall be used only for CL1...2)
i = radius of gyration

CL = cross-section Class at uniform compression
$N_{c.Rd}$ = design compression resistance (without buckling)
$M_{c.Rd}$ = design bending resistance (respective to cross-section Class)
$V_{pl.Rd}$ = design plastic shear resistance (without shear buckling)

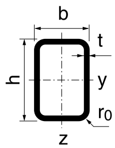
$r_0 = 2{,}0 \times t$ when $t \leq 6{,}0$ mm
$r_0 = 2{,}5 \times t$ when $6{,}0$ mm $< t \leq 10{,}0$ mm
$r_0 = 3{,}0 \times t$ when $t > 10{,}0$ mm

The calculated resistance values are design values (see Chapter 2) based on recommended partial safety factor values **γ_{M0} = 1,0** and **γ_{M1} = 1,0 (for Class 4 circular hollow sections γ_{M1} = 1,1)** as given in Eurocode 3 (EN 1993). Partial safety factor values may differ in each country. National values must be checked from the National Annex of the relevant country.

h mm	b mm	t mm	M kg/m	A mm^2 x 10^2	A_u m^2/m	A_m/V 1/m	I_t mm^4 x 10^4	W_t mm^3 x 10^3	I_y mm^4 x 10^4	$W_{el.y}$ mm^3 x 10^3	$W_{pl.y}$ mm^3 x 10^3	i_y mm x 10	I_z mm^4 x 10^4	$W_{el.z}$ mm^3 x 10^3	$W_{pl.z}$ mm^3 x 10^3	i_z mm x 10	CL h	CL b	$N_{c.Rd}$ kN	$M_{c.y.Rd}$ kNm	$M_{c.z.Rd}$ kNm	$V_{pl.y.Rd}$ kN	$V_{pl.z.Rd}$ kN
40	20	2	1,68	2,14	0,113	529	3,45	2,36	4,05	2,02	2,61	1,38	1,34	1,34	1,60	0,79	1	1	89,75	1,10	0,67	17,27	34,55
40	20	2,5	2,03	2,59	0,111	430	4,06	2,72	4,69	2,35	3,09	1,35	1,54	1,54	1,88	0,77	1	1	108,7	1,30	0,79	20,93	41,85
40	20	3	2,36	3,01	0,110	365	4,57	3,00	5,21	2,60	3,50	1,32	1,68	1,68	2,12	0,75	1	1	126,4	1,47	0,89	24,32	48,63
40	30	2	1,99	2,54	0,133	525	7,07	3,79	5,49	2,75	3,37	1,47	3,51	2,34	2,77	1,18	1	1	106,6	1,42	1,16	26,37	35,15
40	30	2,5	2,42	3,09	0,131	425	8,47	4,46	6,45	3,23	4,03	1,45	4,10	2,74	3,30	1,15	1	1	129,7	1,69	1,38	32,10	42,80
40	30	3	2,83	3,61	0,130	359	9,72	5,03	7,27	3,63	4,61	1,42	4,60	3,07	3,77	1,13	1	1	151,6	1,94	1,58	37,50	50,00
50	30	2	2,31	2,94	0,153	521	9,77	4,84	9,54	3,81	4,74	1,80	4,29	2,86	3,33	1,21	1	1	123,4	1,99	1,40	26,71	44,51
50	30	2,5	2,82	3,59	0,151	422	11,74	5,72	11,30	4,52	5,70	1,77	5,05	3,37	3,98	1,19	1	1	150,7	2,39	1,67	32,64	54,39
50	30	3	3,30	4,21	0,150	356	13,53	6,49	12,83	5,13	6,57	1,75	5,70	3,80	4,58	1,16	1	1	176,8	2,76	1,92	38,27	63,78
50	30	4	4,20	5,35	0,146	273	16,53	7,71	15,25	6,10	8,05	1,69	6,69	4,46	5,58	1,12	1	1	224,6	3,38	2,34	48,63	81,05
60	40	2	2,93	3,74	0,193	517	20,70	8,12	18,41	6,14	7,47	2,22	9,83	4,92	5,65	1,62	2	1	157,0	3,14	2,37	36,25	54,37
60	40	2,5	3,60	4,59	0,191	417	25,14	9,72	22,07	7,36	9,06	2,19	11,74	5,87	6,84	1,60	1	1	192,7	3,80	2,87	44,51	66,77
60	40	3	4,25	5,41	0,190	351	29,28	11,17	25,38	8,46	10,53	2,17	13,44	6,72	7,94	1,58	1	1	227,2	4,42	3,34	52,46	78,69
60	40	4	5,45	6,95	0,186	268	36,67	13,65	30,99	10,33	13,16	2,11	16,28	8,14	9,89	1,53	1	1	291,8	5,53	4,16	67,39	101,1
60	40	5	6,56	8,36	0,183	219	42,85	15,60	35,33	11,78	15,38	2,06	18,43	9,21	11,52	1,48	1	1	351,0	6,46	4,84	81,05	121,6
70	50	2	3,56	4,54	0,233	514	37,45	12,20	31,48	8,99	10,80	2,63	18,76	7,50	8,58	2,03	4	1	184,1	4,54	3,03	45,84	64,18
70	50	2,5	4,39	5,59	0,231	414	45,75	14,72	38,01	10,86	13,16	2,61	22,59	9,04	10,45	2,01	2	1	234,7	5,53	4,39	56,47	79,06
70	50	3	5,19	6,61	0,230	348	53,62	17,06	44,05	12,59	15,40	2,58	26,10	10,44	12,21	1,99	1	1	277,6	6,47	5,13	66,77	93,47
70	50	4	6,71	8,55	0,226	265	68,07	21,19	54,67	15,62	19,48	2,53	32,22	12,89	15,41	1,94	1	1	359,0	8,18	6,47	86,37	120,9
70	50	5	8,13	10,36	0,223	215	80,77	24,64	63,46	18,13	23,06	2,48	37,20	14,88	18,20	1,90	1	1	435,0	9,68	7,64	104,63	146,5
80	40	2	3,56	4,54	0,233	514	30,88	11,00	37,36	9,34	11,61	2,87	12,72	6,36	7,17	1,67	4	1	172,9	4,88	2,39	36,67	73,34
80	40	2,5	4,39	5,59	0,231	414	37,58	13,24	45,11	11,28	14,15	2,84	15,26	7,63	8,72	1,65	3	1	234,7	5,94	3,20	45,18	90,35
80	40	3	5,19	6,61	0,230	348	43,88	15,28	52,25	13,06	16,54	2,81	17,56	8,78	10,16	1,63	1	1	277,6	6,95	4,27	53,41	106,8
80	40	4	6,71	8,55	0,226	265	55,24	18,84	64,79	16,20	20,91	2,75	21,49	10,74	12,77	1,59	1	1	359,0	8,78	5,37	69,09	138,2
80	40	5	8,13	10,36	0,223	215	64,97	21,74	75,11	18,78	24,74	2,69	24,59	12,30	15,02	1,54	1	1	435,0	10,39	6,31	83,71	167,4

h mm	b mm	t mm	M kg/m	A mm^2 $x\ 10^2$	A_u m^2/m	A_m/V 1/m	I_t mm^4 $x\ 10^4$	W_t mm^3 $x\ 10^3$	I_y mm^4 $x\ 10^4$	$W_{el.y}$ mm^3 $x\ 10^3$	$W_{pl.y}$ mm^3 $x\ 10^3$	i_y mm x 10	I_z mm^4 $x\ 10^4$	$W_{el.z}$ mm^3 $x\ 10^3$	$W_{pl.z}$ mm^3 $x\ 10^3$	i_z mm x 10	CL h	CL b	$N_{c.Rd}$ kN	$M_{c.y.Rd}$ kNm	$M_{c.z.Rd}$ kNm	$V_{pl.y.Rd}$ kN	$V_{pl.z.Rd}$ kN
80	60	2,5	5,17	6,59	0,271	412	75,07	20,73	60,13	15,03	18,02	3,02	38,61	12,87	14,81	2,42	3	1	276,7	7,57	5,41	68,48	91,30
80	60	3	6,13	7,81	0,270	345	88,35	24,14	70,05	17,51	21,16	3,00	44,89	14,96	17,37	2,40	1	1	328,0	8,89	7,30	81,15	108,2
80	60	4	7,97	10,15	0,266	262	113,1	30,32	87,92	21,98	26,99	2,94	56,12	18,71	22,12	2,35	1	1	426,2	11,34	9,29	105,5	140,6
80	60	5	9,70	12,36	0,263	213	135,5	35,67	103,3	25,82	32,24	2,89	65,66	21,89	26,38	2,31	1	1	519,0	13,54	11,08	128,4	171,2
90	50	2,5	5,17	6,59	0,271	412	65,30	19,24	70,26	15,61	19,25	3,27	28,24	11,29	12,82	2,07	4	1	263,4	8,09	4,48	57,06	102,7
90	50	3	6,13	7,81	0,270	345	76,67	22,36	81,85	18,19	22,60	3,24	32,74	13,10	15,03	2,05	2	1	328,0	9,49	6,31	67,62	121,7
90	50	4	7,97	10,15	0,266	262	97,70	27,96	102,7	22,82	28,82	3,18	40,71	16,28	19,09	2,00	1	1	426,2	12,11	8,02	87,88	158,2
90	50	5	9,70	12,36	0,263	213	116,5	32,75	120,6	26,80	34,41	3,12	47,37	18,95	22,70	1,96	1	1	519,0	14,45	9,53	107,0	192,6
100	40	2,5	5,17	6,59	0,271	412	50,52	16,76	79,32	15,86	20,23	3,47	18,78	9,39	10,59	1,69	4	1	249,2	8,50	3,52	45,65	114,1
100	40	3	6,13	7,81	0,270	345	59,05	19,39	92,34	18,47	23,75	3,44	21,67	10,84	12,38	1,67	3	1	328,0	9,97	4,55	54,10	135,2
100	40	4	7,97	10,15	0,266	262	74,53	24,04	115,7	23,14	30,26	3,38	26,69	13,35	15,65	1,62	1	1	426,2	12,71	6,58	70,31	175,8
100	40	5	9,70	12,36	0,263	213	87,92	27,90	135,6	27,12	36,09	3,31	30,76	15,38	18,52	1,58	1	1	519,0	15,16	7,78	85,61	214,0
100	50	2,5	5,56	7,09	0,291	411	75,39	21,49	91,20	18,24	22,67	3,59	31,06	12,42	14,01	2,09	4	1	270,2	9,52	4,68	57,30	114,6
100	50	3	6,60	8,41	0,290	345	88,56	25,01	106,5	21,29	26,66	3,56	36,06	14,42	16,44	2,07	3	1	353,2	11,20	6,06	67,96	135,9
100	50	4	8,59	10,95	0,286	261	113,0	31,35	134,1	26,83	34,10	3,50	44,95	17,98	20,93	2,03	1	1	459,8	14,32	8,79	88,49	177,0
100	50	5	10,5	13,36	0,283	212	134,9	36,80	158,2	31,64	40,84	3,44	52,45	20,98	24,95	1,98	1	1	561,0	17,15	10,48	108,0	215,9
100	50	6	12,3	15,63	0,279	179	154,2	41,43	178,8	35,75	46,90	3,38	58,67	23,47	28,52	1,94	1	1	656,6	19,70	11,98	126,4	252,7
100	60	2,5	5,96	7,59	0,311	410	103,3	26,23	103,1	20,62	25,11	3,69	46,88	15,63	17,68	2,49	4	1	291,2	10,55	5,91	69,01	115,0
100	60	3	7,07	9,01	0,310	344	121,7	30,64	120,6	24,11	29,57	3,66	54,65	18,22	20,79	2,46	3	1	378,4	12,42	7,65	81,91	136,5
100	60	4	9,22	11,75	0,306	261	156,3	38,68	152,6	30,52	37,94	3,60	68,68	22,89	26,60	2,42	1	1	493,4	15,93	11,17	106,8	178,1
100	60	5	11,3	14,36	0,303	211	187,9	45,75	180,8	36,15	45,59	3,55	80,83	26,94	31,88	2,37	1	1	603,0	19,15	13,39	130,5	217,6
100	60	6	13,2	16,83	0,299	178	216,4	51,92	205,3	41,06	52,54	3,49	91,20	30,40	36,64	2,33	1	1	707,0	22,07	15,39	153,1	255,1
100	80	2,5	6,74	8,59	0,351	409	165,8	35,73	126,9	25,37	29,98	3,84	90,17	22,54	25,77	3,24	4	3	332,1	10,66	8,60	92,57	115,7
100	80	3	8,01	10,21	0,350	343	196,1	41,91	148,8	29,76	35,39	3,82	105,6	26,41	30,40	3,22	3	1	428,8	14,86	11,09	110,0	137,5
100	80	4	10,5	13,35	0,346	259	253,8	53,38	189,5	37,89	45,62	3,77	134,2	33,54	39,15	3,17	1	1	560,6	19,16	16,44	143,9	179,8
100	80	5	12,8	16,36	0,343	210	307,6	63,72	225,9	45,19	55,09	3,72	159,6	39,90	47,24	3,12	1	1	687,0	23,14	19,84	176,3	220,3
100	80	6	15,1	19,23	0,339	176	357,4	72,98	258,4	51,68	63,82	3,67	182,1	45,53	54,67	3,08	1	1	807,8	26,80	22,96	207,3	259,1
120	40	2,5	5,96	7,59	0,311	410	63,77	20,27	126,7	21,12	27,32	4,09	22,30	11,15	12,47	1,71	4	1	259,2	11,48	3,74	46,01	138,0
120	40	3	7,07	9,01	0,310	344	74,56	23,51	148,0	24,67	32,16	4,05	25,79	12,89	14,60	1,69	4	1	338,7	13,51	4,81	54,61	163,8
120	40	4	9,22	11,75	0,306	261	94,23	29,24	186,9	31,15	41,21	3,99	31,90	15,95	18,53	1,65	2	1	493,4	17,31	7,78	71,22	213,7
120	40	5	11,3	14,36	0,303	211	111,4	34,05	220,8	36,80	49,45	3,92	36,93	18,46	22,02	1,60	1	1	603,0	20,77	9,25	87,03	261,1
120	40	6	13,2	16,83	0,299	178	126,0	38,02	250,0	41,66	56,89	3,85	40,97	20,49	25,08	1,56	1	1	707,0	23,90	10,53	102,0	306,1
120	50	2,5	6,35	8,09	0,331	410	96,03	26,01	144,0	23,99	30,26	4,22	36,70	14,68	16,39	2,13	4	1	280,2	12,71	4,98	57,69	138,5
120	50	3	7,54	9,61	0,330	343	112,9	30,32	168,6	28,10	35,67	4,19	42,69	17,08	19,26	2,11	4	1	363,9	14,98	6,40	68,53	164,5
120	50	4	9,85	12,55	0,326	260	144,2	38,13	213,8	35,64	45,85	4,13	53,43	21,37	24,61	2,06	2	1	527,0	19,26	10,34	89,49	214,8
120	50	5	12,1	15,36	0,323	210	172,4	44,92	253,9	42,32	55,20	4,07	62,62	25,05	29,45	2,02	1	1	645,0	23,18	12,37	109,5	262,9
120	50	6	14,2	18,03	0,319	177	197,6	50,76	289,0	48,16	63,73	4,00	70,36	28,14	33,80	1,98	1	1	757,4	26,77	14,20	128,6	308,7
120	60	2,5	6,74	8,59	0,351	409	132,6	31,75	161,2	26,87	33,20	4,33	55,15	18,38	20,56	2,53	4	1	301,2	13,94	6,29	69,42	138,9
120	60	3	8,01	10,21	0,350	343	156,3	37,14	189,1	31,52	39,18	4,30	64,40	21,47	24,21	2,51	4	1	389,1	16,45	8,08	82,51	165,0
120	60	4	10,5	13,35	0,346	259	201,1	47,05	240,7	40,12	50,49	4,25	81,25	27,08	31,08	2,47	2	1	560,6	21,20	13,05	107,9	215,8
120	60	5	12,8	16,36	0,343	210	242,2	55,85	287,0	47,83	60,95	4,19	95,99	32,00	37,38	2,42	1	1	687,0	25,60	15,70	132,2	264,4
120	60	6	15,1	19,23	0,339	176	279,7	63,60	328,0	54,67	70,57	4,13	108,8	36,26	43,12	2,38	1	1	807,8	29,64	18,11	155,5	310,9

Table 11.1.6 Cross-sectional properties and resistance values for rectangular hollow sections of steel grade S420MH (f_y = 420 N/mm^2) *continued*



M = weight
A = cross-section area
A_u = external area
A_m/V = cross-section factor in fire design
I_t = St. Venant torsional constant
W_t = torsional section modulus

I = second moment of area
W_{el} = elastic section modulus
W_{pl} = plastic section modulus (shall be used only for CL1...2)
i = radius of gyration

CL = cross-section Class at uniform compression
$N_{c.Rd}$ = design compression resistance (without buckling)
$M_{c.Rd}$ = design bending resistance (respective to cross-section Class)
$V_{pl.Rd}$ = design plastic shear resistance (without shear buckling)

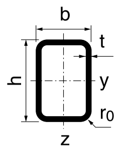
r_0 = 2,0 x t when t ≤ 6,0 mm
r_0 = 2,5 x t when 6,0 mm < t ≤ 10,0 mm
r_0 = 3,0 x t when t > 10,0 mm

The calculated resistance values are design values (see Chapter 2) based on recommended partial safety factor values **γ_{M0} = 1,0** and **γ_{M1} = 1,0 (for Class 4 circular hollow sections γ_{M1} = 1,1)** as given in Eurocode 3 (EN 1993). Partial safety factor values may differ in each country. National values must be checked from the National Annex of the relevant country.

h mm	b mm	t mm	M kg/m	A mm^2 x 10^2	A_u m^2/m	A_m/V 1/m	I_t mm^4 x 10^4	W_t mm^3 x 10^3	I_y mm^4 x 10^4	$W_{el.y}$ mm^3 x 10^3	$W_{pl.y}$ mm^3 x 10^3	i_y mm x 10	I_z mm^4 x 10^4	$W_{el.z}$ mm^3 x 10^3	$W_{pl.z}$ mm^3 x 10^3	i_z mm x 10	CL h	CL b	$N_{c.Rd}$ kN	$M_{c.y.Rd}$ kNm	$M_{c.z.Rd}$ kNm	$V_{pl.y.Rd}$ kN	$V_{pl.z.Rd}$ kN
120	80	2,5	7,53	9,59	0,391	408	215,8	43,23	195,8	32,63	39,07	4,52	105,2	26,30	29,65	3,31	4	3	342,1	13,70	9,13	93,01	139,5
120	80	3	8,96	11,41	0,390	342	255,5	50,80	230,2	38,37	46,20	4,49	123,4	30,86	35,02	3,29	4	1	439,5	19,40	11,71	110,7	166,0
120	80	4	11,7	14,95	0,386	258	331,2	64,93	294,6	49,10	59,77	4,44	157,3	39,32	45,23	3,24	2	1	627,8	25,10	19,00	145,0	217,5
120	80	5	14,4	18,36	0,383	209	402,3	77,77	353,1	58,86	72,45	4,39	187,8	46,94	54,74	3,20	1	1	771,0	30,43	22,99	178,1	267,1
120	80	6	17,0	21,63	0,379	175	468,5	89,40	406,1	67,68	84,25	4,33	215,0	53,76	63,55	3,15	1	1	908,6	35,39	26,69	209,8	314,7
120	80	7,1	19,4	24,65	0,369	150	535,1	99,99	442,1	73,68	93,59	4,24	234,4	58,60	70,68	3,08	1	1	1035	39,31	29,69	239,1	358,6
120	80	8	21,4	27,24	0,366	134	584,0	108,0	475,8	79,31	102,0	4,18	251,7	62,92	76,93	3,04	1	1	1144	42,83	32,31	264,2	396,4
120	80	8,8	23,1	29,44	0,362	123	623,5	114,3	501,8	83,63	108,8	4,13	264,8	66,21	81,97	3,00	1	1	1237	45,68	34,43	285,6	428,4
120	80	10	25,6	32,57	0,357	110	675,6	122,4	534,1	89,02	117,8	4,05	281,1	70,29	88,68	2,94	1	1	1368	49,48	37,25	315,9	473,8
120	100	2,5	8,31	10,59	0,431	407	309,4	54,73	230,3	38,38	44,95	4,66	174,4	34,88	39,73	4,06	4	4	357,7	14,84	12,27	116,7	140,1
120	100	3	9,90	12,61	0,430	341	367,0	64,47	271,3	45,21	53,22	4,64	205,3	41,06	47,03	4,04	4	3	482,8	18,99	15,68	139,0	166,8
120	100	4	13,0	16,55	0,426	258	477,8	82,83	348,4	58,07	69,05	4,59	263,2	52,65	60,98	3,99	2	1	695,0	29,00	25,61	182,4	218,9
120	100	5	16,0	20,36	0,423	208	582,9	99,75	419,3	69,88	83,95	4,54	316,3	63,25	74,09	3,94	1	1	855,0	35,26	31,12	224,4	269,2
120	100	6	18,9	24,03	0,419	175	682,0	115,3	484,1	80,68	97,93	4,49	364,6	72,91	86,38	3,89	1	1	1009	41,13	36,28	264,9	317,9
140	60	2,5	7,53	9,59	0,391	408	162,7	37,26	236,6	33,79	42,29	4,97	63,43	21,14	23,43	2,57	4	1	308,2	17,76	6,57	69,76	162,8
140	60	3	8,96	11,41	0,390	342	191,9	43,64	278,1	39,73	49,98	4,94	74,16	24,72	27,63	2,55	4	1	401,5	20,99	8,52	82,99	193,6
140	60	4	11,7	14,95	0,386	258	247,1	55,42	355,6	50,80	64,63	4,88	93,81	31,27	35,56	2,51	4	1	602,0	27,15	12,54	108,7	253,7
140	60	5	14,4	18,36	0,383	209	298,0	65,94	425,9	60,84	78,30	4,82	111,2	37,05	42,88	2,46	2	1	771,0	32,89	18,01	133,5	311,6
140	60	6	17,0	21,63	0,379	175	344,5	75,29	489,2	69,88	91,01	4,76	126,3	42,11	49,60	2,42	1	1	908,6	38,22	20,83	157,4	367,2
140	70	2,5	7,92	10,09	0,411	408	213,1	44,00	260,2	37,17	45,72	5,08	89,30	25,51	28,35	2,98	4	2	329,2	19,20	8,01	81,55	163,1
140	70	3	9,43	12,01	0,410	341	252,0	51,66	306,2	43,75	54,09	5,05	104,7	29,91	33,49	2,95	4	1	426,7	22,72	10,38	97,06	194,1
140	70	4	12,4	15,75	0,406	258	326,0	65,94	392,6	56,09	70,07	4,99	133,2	38,05	43,24	2,91	4	1	635,6	29,43	15,28	127,3	254,6
140	70	5	15,2	19,36	0,403	208	395,1	78,88	471,5	67,35	85,05	4,94	158,7	45,35	52,31	2,86	2	1	813,0	35,72	21,97	156,5	312,9
140	70	6	17,9	22,83	0,399	175	459,1	90,54	543,1	77,59	99,05	4,88	181,4	51,84	60,71	2,82	1	1	959,0	41,60	25,50	184,6	369,1
140	80	3	9,90	12,61	0,430	341	317,1	59,69	334,4	47,77	58,20	5,15	141,2	35,31	39,64	3,35	4	1	451,9	24,45	12,34	111,2	194,6
140	80	4	13,0	16,55	0,426	258	411,6	76,48	429,6	61,37	75,51	5,10	180,4	45,10	51,31	3,30	4	1	669,2	31,72	18,14	145,9	255,4
140	80	5	16,0	20,36	0,423	208	500,5	91,83	517,1	73,87	91,80	5,04	215,9	53,99	62,24	3,26	2	1	855,0	38,56	26,14	179,5	314,1
140	80	6	18,9	24,03	0,419	175	583,8	105,8	597,0	85,29	107,1	4,98	248,0	61,99	72,43	3,21	1	1	1009	44,98	30,42	211,9	370,9

h mm	b mm	t mm	M kg/m	A mm^2 $x 10^2$	A_u m^2/m	A_m/V 1/m	I_t mm^4 $x 10^4$	W_t mm^3 $x 10^3$	I_y mm^4 $x 10^4$	$W_{el.y}$ mm^3 $x 10^3$	$W_{pl.y}$ mm^3 $x 10^3$	i_y mm x 10	I_z mm^4 $x 10^4$	$W_{el.z}$ mm^3 $x 10^3$	$W_{pl.z}$ mm^3 $x 10^3$	i_z mm x 10	CL h	CL b	$N_{c.Rd}$ kN	$M_{c.y.Rd}$ kNm	$M_{c.z.Rd}$ kNm	$V_{pl.y.Rd}$ kN	$V_{pl.z.Rd}$ kN
150	50	2,5	7,53	9,59	0,391	408	127,7	32,78	254,1	33,88	43,52	5,15	45,17	18,07	19,95	2,17	4	2	290,0	18,28	5,29	58,13	174,4
150	50	3	8,96	11,41	0,390	342	150,2	38,28	298,6	39,81	51,43	5,12	52,65	21,06	23,49	2,15	4	1	381,2	21,60	6,90	69,16	207,5
150	50	4	11,7	14,95	0,386	258	192,1	48,30	381,4	50,85	66,47	5,05	66,16	26,47	30,13	2,10	4	1	580,5	27,92	10,22	90,62	271,9
150	50	5	14,4	18,36	0,383	209	230,1	57,11	456,3	60,84	80,48	4,99	77,87	31,15	36,20	2,06	2	1	771,0	33,80	15,20	111,3	333,8
150	50	6	17,0	21,63	0,379	175	264,0	64,77	523,5	69,80	93,48	4,92	87,89	35,16	41,72	2,02	1	1	908,6	39,26	17,52	131,1	393,4
150	100	3	11,3	14,41	0,490	340	507,2	81,40	460,6	61,42	73,48	5,65	247,6	49,53	55,76	4,15	4	3	500,1	25,80	16,85	139,8	209,6
150	100	4	14,9	18,95	0,486	257	661,6	104,9	594,6	79,28	95,67	5,60	318,6	63,71	72,50	4,10	4	1	748,5	40,18	24,90	183,8	275,7
150	100	5	18,3	23,36	0,483	207	808,7	126,8	719,2	95,89	116,7	5,55	384,0	76,80	88,34	4,05	2	1	981,0	49,03	37,10	226,5	339,8
150	100	6	21,7	27,63	0,479	173	948,3	147,1	834,7	111,3	136,7	5,50	444,2	88,84	103,3	4,01	1	1	1161	57,41	43,39	268,0	402,0
150	100	7,1	24,9	31,75	0,470	148	1096	166,8	926,9	123,6	154,1	5,40	493,6	98,72	116,6	3,94	1	1	1334	64,72	48,96	308,0	462,0
150	100	8	27,7	35,24	0,466	132	1206	181,9	1008	134,4	169,2	5,35	535,7	107,1	127,9	3,90	1	1	1480	71,05	53,70	341,8	512,8
150	100	8,8	30,0	38,24	0,462	121	1298	194,2	1074	143,2	181,7	5,30	569,5	113,9	137,3	3,86	1	1	1606	76,33	57,64	370,9	556,4
150	100	10	33,4	42,57	0,457	107	1426	211,0	1162	154,9	199,2	5,22	614,4	122,9	150,3	3,80	1	1	1788	83,65	63,10	412,9	619,3
160	70	4	13,6	17,35	0,446	257	389,7	75,90	549,0	68,63	86,62	5,63	150,6	43,04	48,52	2,95	4	1	658,1	36,38	16,15	128,0	292,6
160	70	5	16,8	21,36	0,443	207	472,5	90,96	661,6	82,70	105,4	5,57	179,9	51,39	58,81	2,90	3	1	897,0	44,27	21,59	157,6	360,3
160	70	6	19,8	25,23	0,439	174	549,6	104,6	764,8	95,60	123,1	5,51	206,1	58,88	68,39	2,86	1	1	1060	51,69	28,72	186,2	425,6
160	70	8	25,2	32,04	0,426	133	683,7	126,7	908,7	113,6	150,7	5,33	243,3	69,50	83,55	2,76	1	1	1346	63,29	35,09	236,5	540,5
160	80	3	10,8	13,81	0,470	340	380,3	68,59	463,8	57,98	71,41	5,80	159,0	39,76	44,26	3,39	4	1	461,0	29,99	12,83	111,6	223,2
160	80	4	14,3	18,15	0,466	257	494,1	88,03	597,7	74,71	92,86	5,74	203,5	50,89	57,39	3,35	4	1	691,7	39,00	19,15	146,7	293,4
160	80	5	17,6	22,36	0,463	207	601,3	105,9	721,7	90,21	113,2	5,68	244,1	61,03	69,74	3,30	3	1	939,0	47,53	25,63	180,7	361,4
160	80	6	20,8	26,43	0,459	174	702,1	122,3	836,0	104,5	132,3	5,62	280,9	70,22	81,31	3,26	1	1	1110	55,57	34,15	213,7	427,3
160	80	7,1	23,8	30,33	0,449	148	805,5	137,8	922,6	115,3	148,6	5,52	310,1	77,53	91,39	3,20	1	1	1274	62,40	38,38	245,1	490,3
160	80	8	26,4	33,64	0,446	132	882,3	149,5	1001	125,2	162,9	5,46	335,0	83,74	99,97	3,16	1	1	1413	68,40	41,99	271,9	543,9
160	80	10	31,8	40,57	0,437	108	1031	171,6	1146	143,3	191,0	5,32	379,8	94,95	116,7	3,06	1	1	1704	80,20	49,01	327,9	655,8
160	90	3	11,3	14,41	0,490	340	465,4	77,82	500,8	62,60	76,12	5,90	206,8	45,95	51,31	3,79	4	2	486,2	31,97	14,95	125,8	223,6
160	90	4	14,9	18,95	0,486	257	606,2	100,2	646,4	80,80	99,10	5,84	265,5	59,01	66,66	3,74	4	1	725,3	41,62	22,28	165,4	294,1
160	90	5	18,3	23,36	0,483	207	739,7	120,9	781,8	97,72	120,9	5,79	319,5	71,00	81,16	3,70	3	1	981,0	50,78	29,82	203,9	362,5
160	90	6	21,7	27,63	0,479	173	866,0	140,0	907,2	113,4	141,6	5,73	368,9	81,98	94,82	3,65	1	1	1161	59,46	39,83	241,2	428,8
160	90	7,1	24,9	31,75	0,470	148	997,9	158,4	1006	125,8	159,5	5,63	409,3	90,96	106,9	3,59	1	1	1334	66,98	44,91	277,2	492,8
160	90	8	27,7	35,24	0,466	132	1097	172,4	1094	136,7	175,0	5,57	443,5	98,55	117,2	3,55	1	1	1480	73,51	49,22	307,7	546,9
160	90	8,8	30,0	38,24	0,462	121	1178	183,9	1165	145,6	188,0	5,52	470,8	104,6	125,7	3,51	1	1	1606	78,95	52,80	333,9	593,5
180	100	4	16,8	21,35	0,546	256	853,9	127,1	926,0	102,9	125,9	6,59	373,9	74,78	84,02	4,18	4	1	776,1	52,87	26,59	184,9	332,8
180	100	5	20,7	26,36	0,543	206	1045	153,9	1124	124,9	154,0	6,53	451,8	90,35	102,6	4,14	4	1	1053	64,69	35,87	228,3	410,9
180	100	5,6	23,0	29,30	0,541	185	1155	169,1	1237	137,4	170,3	6,50	495,7	99,14	113,3	4,11	3	1	1231	71,51	41,64	253,7	456,7
180	100	6	24,5	31,23	0,539	173	1227	178,9	1310	145,5	180,8	6,48	523,8	104,8	120,2	4,10	2	1	1312	75,95	50,49	270,5	486,9
180	100	7,1	28,3	36,01	0,530	147	1420	203,5	1464	162,7	204,9	6,38	585,7	117,1	136,4	4,03	1	1	1513	86,07	57,27	311,9	561,4
180	100	8	31,4	40,04	0,526	131	1565	222,5	1598	177,6	225,6	6,32	637,5	127,5	149,9	3,99	1	1	1682	94,76	62,97	346,8	624,2
180	100	8,8	34,2	43,52	0,522	120	1688	238,2	1709	189,9	243,1	6,27	679,7	135,9	161,3	3,95	1	1	1828	102,1	67,76	376,9	678,5
180	100	10	38,1	48,57	0,517	106	1859	259,6	1859	206,6	267,5	6,19	736,4	147,3	177,3	3,89	1	1	2040	112,4	74,44	420,6	757,1

Table 11.1.6 Cross-sectional properties and resistance values for rectangular hollow sections of steel grade S420MH (f_y = 420 N/mm²) *continued*



M = weight
A = cross-section area
A_u = external area
A_m/V = cross-section factor in fire design
I_t = St. Venant torsional constant
W_t = torsional section modulus

I = second moment of area
W_{el} = elastic section modulus
W_{pl} = plastic section modulus (shall be used only for CL1...2)
i = radius of gyration

CL = cross-section Class at uniform compression
$N_{c.Rd}$ = design compression resistance (without buckling)
$M_{c.Rd}$ = design bending resistance (respective to cross-section Class)
$V_{pl.Rd}$ = design plastic shear resistance (without shear buckling)

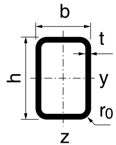
r_0 = 2,0 x t when t ≤ 6,0 mm
r_0 = 2,5 x t when 6,0 mm < t ≤ 10,0 mm
r_0 = 3,0 x t when t > 10,0 mm

The calculated resistance values are design values (see Chapter 2) based on recommended partial safety factor values **γ_{M0} = 1,0** and **γ_{M1} = 1,0 (for Class 4 circular hollow sections γ_{M1} = 1,1)** as given in Eurocode 3 (EN 1993). Partial safety factor values may differ in each country. National values must be checked from the National Annex of the relevant country.

h mm	b mm	t mm	M kg/m	A mm² x 10²	A_u m²/m	A_m/V 1/m	I_t mm⁴ x 10⁴	W_t mm³ x 10³	I_y mm⁴ x 10⁴	$W_{el.y}$ mm³ x 10³	$W_{pl.y}$ mm³ x 10³	i_y mm x 10	I_z mm⁴ x 10⁴	$W_{el.z}$ mm³ x 10³	$W_{pl.z}$ mm³ x 10³	i_z mm x 10	CL h	CL b	$N_{c.Rd}$ kN	$M_{c.y.Rd}$ kNm	$M_{c.z.Rd}$ kNm	$V_{pl.y.Rd}$ kN	$V_{pl.z.Rd}$ kN
180	120	4	18,0	22,95	0,586	255	1160	154,6	1050	116,7	140,0	6,76	563,8	93,97	106,2	4,96	4	2	843,3	58,79	33,69	222,6	333,9
180	120	5	22,3	28,36	0,583	206	1424	187,8	1277	141,9	171,5	6,71	684,0	114,0	130,0	4,91	4	1	1137	72,04	45,37	275,0	412,6
180	120	6	26,4	33,63	0,579	172	1677	219,1	1491	165,7	201,7	6,66	796,3	132,7	152,7	4,87	2	1	1413	84,72	64,11	326,2	489,3
180	120	7,1	30,5	38,85	0,570	147	1949	250,5	1676	186,3	229,5	6,57	895,4	149,2	173,8	4,80	1	1	1632	96,38	72,99	376,9	565,3
180	120	8	34,0	43,24	0,566	131	2156	274,8	1835	203,9	253,1	6,51	978,4	163,1	191,6	4,76	1	1	1816	106,3	80,46	419,4	629,1
180	120	8,8	36,9	47,04	0,562	120	2332	295,1	1967	218,6	273,2	6,47	1047	174,5	206,6	4,72	1	1	1976	114,7	86,78	456,3	684,4
180	120	10	41,3	52,57	0,557	106	2582	323,3	2149	238,8	301,5	6,39	1141	190,1	227,8	4,66	1	1	2208	126,6	95,68	509,9	764,8
180	120	12,5	48,7	62,04	0,536	86	3001	367,5	2352	261,4	341,5	6,16	1252	208,7	258,5	4,49	1	1	2606	143,4	108,6	601,8	902,7
200	80	4	16,8	21,35	0,546	256	663,6	111,1	1046	104,6	132,4	7,00	249,8	62,45	69,55	3,42	4	1	722,4	55,59	20,63	147,9	369,8
200	80	5	20,7	26,36	0,543	206	808,4	134,1	1269	126,9	161,9	6,94	300,4	75,11	84,74	3,38	4	1	996,8	67,99	28,12	182,6	456,5
200	80	6	24,5	31,23	0,539	173	944,8	155,2	1477	147,7	190,0	6,88	346,7	86,69	99,07	3,33	3	1	1312	79,79	36,41	216,4	541,0
200	80	7,1	28,3	36,01	0,529	147	1086	175,6	1646	164,6	214,9	6,76	385,8	96,45	112,1	3,27	2	1	1512	90,26	47,08	249,5	623,7
200	80	8	31,4	40,04	0,526	131	1192	191,1	1796	179,6	236,5	6,70	418,2	104,6	123,0	3,23	1	1	1682	99,35	51,66	277,4	693,6
200	80	8,8	34,2	43,52	0,522	120	1279	203,8	1918	191,8	254,7	6,64	444,2	111,1	132,1	3,19	1	1	1828	107,0	55,48	301,5	753,9
200	80	10	38,1	48,57	0,517	106	1399	220,8	2083	208,3	280,1	6,55	478,5	119,6	144,7	3,14	1	1	2040	117,6	60,77	336,5	841,2
200	100	4	18,0	22,95	0,586	255	985,4	141,8	1200	120,0	148,0	7,23	410,8	82,16	91,70	4,23	4	1	789,6	62,18	27,47	185,5	371,0
200	100	5	22,3	28,36	0,583	206	1206	171,9	1459	145,9	181,4	7,17	496,9	99,39	112,1	4,19	4	1	1081	76,18	37,41	229,2	458,4
200	100	6	26,4	33,63	0,579	172	1417	200,1	1703	170,3	213,3	7,12	576,9	115,4	131,5	4,14	3	1	1413	89,57	48,46	271,9	543,7
200	100	7,1	30,5	38,85	0,570	147	1641	228,1	1911	191,1	242,4	7,01	647,1	129,4	149,6	4,08	2	1	1632	101,8	62,81	314,0	628,1
200	100	8	34,0	43,24	0,566	131	1811	249,6	2091	209,1	267,3	6,95	705,4	141,1	164,7	4,04	1	1	1816	112,3	69,15	349,5	699,1
200	100	8,8	36,9	47,04	0,562	120	1954	267,5	2240	224,0	288,3	6,90	753,1	150,6	177,4	4,00	1	1	1976	121,1	74,50	380,3	760,5
200	100	10	41,3	52,57	0,557	106	2154	292,1	2444	244,4	318,1	6,82	817,7	163,6	195,3	3,94	1	1	2208	133,6	82,00	424,9	849,8
200	100	12,5	48,7	62,04	0,536	86	2474	328,8	2659	265,9	359,1	6,55	892,2	178,4	220,8	3,79	1	1	2606	150,8	92,73	501,5	1003

h mm	b mm	t mm	M kg/m	A mm^2 x 10^2	A_u m^2/m	A_m/V 1/m	I_t mm^4 x 10^4	W_t mm^3 x 10^3	I_y mm^4 x 10^4	$W_{el.y}$ mm^3 x 10^3	$W_{pl.y}$ mm^3 x 10^3	i_y mm x 10	I_z mm^4 x 10^4	$W_{el.z}$ mm^3 x 10^3	$W_{pl.z}$ mm^3 x 10^3	i_z mm x 10	CL h	CL b	$N_{c.Rd}$ kN	$M_{c.y.Rd}$ kNm	$M_{c.z.Rd}$ kNm	$V_{pl.y.Rd}$ kN	$V_{pl.z.Rd}$ kN
200	120	5	23,8	30,36	0,623	205	1652	209,9	1649	164,9	200,9	7,37	750,1	125,0	141,5	4,97	4	1	1165	84,37	47,29	276,0	460,1
200	120	6	28,3	36,03	0,619	172	1947	245,1	1929	192,9	236,6	7,32	874,4	145,7	166,3	4,93	3	1	1513	99,35	61,20	327,7	546,1
200	120	7,1	32,7	41,69	0,610	146	2265	280,7	2175	217,5	269,7	7,22	986,0	164,3	189,8	4,86	2	1	1751	113,3	79,72	379,1	631,9
200	120	8	36,5	46,44	0,606	130	2507	308,3	2386	238,6	298,0	7,17	1079	179,8	209,5	4,82	1	1	1951	125,2	87,99	422,3	703,9
200	120	8,8	39,7	50,56	0,602	119	2714	331,4	2562	256,2	322,0	7,12	1156	192,7	226,2	4,78	1	1	2124	135,2	95,00	459,8	766,3
200	120	10	44,4	56,57	0,597	106	3007	363,7	2806	280,6	356,1	7,04	1262	210,4	249,8	4,72	1	1	2376	149,6	104,9	514,4	857,3
200	120	12,5	52,6	67,04	0,576	86	3514	415,7	3099	309,9	406,0	6,80	1397	232,9	285,3	4,57	1	1	2816	170,5	119,8	609,6	1016
220	120	5	25,4	32,36	0,663	205	1885	231,9	2082	189,3	232,2	8,02	816,3	136,1	153,0	5,02	4	1	1187	97,54	48,90	276,9	507,7
220	120	6	30,2	38,43	0,659	172	2222	271,1	2439	221,7	273,8	7,97	952,4	158,7	180,0	4,98	4	1	1524	115,0	62,46	328,9	603,0
220	120	7,1	35,0	44,53	0,650	146	2586	310,8	2757	250,7	312,9	7,87	1077	179,4	205,9	4,92	2	1	1870	131,4	86,46	381,1	698,7
220	120	8	39,0	49,64	0,646	130	2864	341,7	3029	275,4	346,0	7,81	1179	196,6	227,4	4,87	1	1	2085	145,3	95,51	424,9	778,9
220	120	8,8	42,5	54,08	0,642	119	3102	367,7	3258	296,1	374,3	7,76	1265	210,9	245,8	4,84	1	1	2272	157,2	103,2	462,9	848,6
220	120	10	47,5	60,57	0,637	105	3440	404,1	3576	325,1	414,7	7,68	1383	230,6	271,8	4,78	1	1	2544	174,2	114,2	518,4	950,3
250	100	5	26,2	33,36	0,683	205	1620	217,1	2554	204,3	258,5	8,75	609,9	122,0	135,8	4,28	4	1	1129	108,6	40,30	231,1	577,7
250	100	6	31,1	39,63	0,679	171	1905	253,2	2992	239,4	304,9	8,69	709,6	141,9	159,7	4,23	4	1	1470	128,0	51,99	274,6	686,5
250	100	8	40,2	51,24	0,666	130	2439	317,4	3714	297,1	385,4	8,51	875,1	175,0	201,5	4,13	2	1	2152	161,9	84,61	355,0	887,5
250	100	10	49,1	62,57	0,657	105	2910	373,3	4384	350,7	462,0	8,37	1021	204,2	240,3	4,04	1	1	2628	194,0	100,9	433,5	1084
250	150	5	30,1	38,36	0,783	204	3285	336,9	3304	264,3	319,8	9,28	1508	201,1	225,5	6,27	4	2	1339	134,3	67,96	348,8	581,3
250	150	6	35,8	45,63	0,779	171	3886	395,7	3886	310,8	378,1	9,23	1768	235,8	266,3	6,23	4	1	1722	158,8	87,42	415,0	691,6
250	150	7,1	41,7	53,05	0,770	145	4543	456,3	4428	354,2	434,6	9,14	2015	268,7	306,3	6,16	4	1	2141	182,5	107,8	482,4	804,0
250	150	8	46,5	59,24	0,766	129	5050	504,0	4886	390,9	482,2	9,08	2219	295,9	339,6	6,12	2	1	2488	202,5	142,6	538,7	897,8
250	150	8,8	50,8	64,64	0,762	118	5488	544,5	5274	422,0	523,1	9,03	2392	318,9	368,1	6,08	2	1	2715	219,7	154,6	587,8	979,7
250	150	10	57,0	72,57	0,757	104	6121	602,1	5825	466,0	582,0	8,96	2634	351,2	409,2	6,02	1	1	3048	244,4	171,9	659,9	1100
250	150	12,5	68,3	87,04	0,736	85	7315	704,1	6633	530,6	678,3	8,73	3002	400,3	477,5	5,87	1	1	3656	284,9	200,5	791,5	1319
260	140	6	35,8	45,63	0,779	171	3646	382,5	4082	314,0	385,9	9,46	1567	223,9	251,8	5,86	4	1	1684	162,1	81,08	387,3	719,3
260	140	7,1	41,7	53,05	0,770	145	4258	440,8	4648	357,6	443,5	9,36	1785	255,0	289,6	5,80	4	1	2104	186,3	100,3	450,3	836,2
260	140	8	46,5	59,24	0,766	129	4731	486,5	5129	394,5	492,0	9,30	1964	280,6	320,9	5,76	3	1	2488	206,7	117,9	502,8	933,8
260	140	8,8	50,8	64,64	0,762	118	5138	525,3	5536	425,9	533,7	9,25	2115	302,2	347,8	5,72	2	1	2715	224,2	146,1	548,6	1019
260	140	10	57,0	72,57	0,757	104	5724	580,4	6113	470,2	593,8	9,18	2328	332,5	386,4	5,66	1	1	3048	249,4	162,3	615,9	1144
260	140	12,5	68,3	87,04	0,736	85	6821	677,2	6950	534,6	691,5	8,94	2648	378,3	450,5	5,52	1	1	3656	290,4	189,2	738,7	1372
260	180	6	39,6	50,43	0,859	170	5566	501,4	4856	373,5	446,9	9,81	2763	307,1	347,9	7,40	4	2	1886	187,7	112,3	500,3	722,6
260	180	7,1	46,1	58,73	0,850	145	6522	580,3	5557	427,5	515,3	9,73	3163	351,4	401,3	7,34	4	1	2342	216,4	138,8	582,6	841,6
260	180	8	51,5	65,64	0,846	129	7267	642,4	6145	472,7	572,7	9,68	3493	388,1	445,8	7,29	3	1	2757	240,5	163,0	651,2	940,6
260	180	8,8	56,3	71,68	0,842	117	7912	695,6	6647	511,3	622,2	9,63	3774	419,4	484,1	7,26	2	1	3011	261,3	203,3	711,1	1027
260	180	10	63,2	80,57	0,837	104	8850	771,9	7363	566,4	693,8	9,56	4174	463,8	539,5	7,20	1	1	3384	291,4	226,6	799,2	1154

Table 11.1.6 Cross-sectional properties and resistance values for rectangular hollow sections of steel grade S420MH (f_y = 420 N/mm^2) *continued*



M = weight
A = cross-section area
A_u = external area
A_m/V = cross-section factor in fire design
I_t = St. Venant torsional constant
W_t = torsional section modulus

I = second moment of area
W_{el} = elastic section modulus
W_{pl} = plastic section modulus (shall be used only for CL1...2)
i = radius of gyration

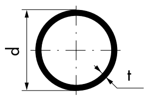
CL = cross-section Class at uniform compression
$N_{c.Rd}$ = design compression resistance (without buckling)
$M_{c.Rd}$ = design bending resistance (respective to cross-section Class)
$V_{pl.Rd}$ = design plastic shear resistance (without shear buckling)

$r_0 = 2,0 \times t$ when $t \leq 6,0$ mm
$r_0 = 2,5 \times t$ when $6,0$ mm $< t \leq 10,0$ mm
$r_0 = 3,0 \times t$ when $t > 10,0$ mm

The calculated resistance values are design values (see Chapter 2) based on recommended partial safety factor values **γ_{M0} = 1,0** and **γ_{M1} = 1,0 (for Class 4 circular hollow sections γ_{M1} = 1,1)** as given in Eurocode 3 (EN 1993). Partial safety factor values may differ in each country. National values must be checked from the National Annex of the relevant country.

h mm	b mm	t mm	M kg/m	A mm^2 x 10^2	A_u m^2/m	A_m/V 1/m	I_t mm^4 x 10^4	W_t mm^3 x 10^3	I_y mm^4 x 10^4	$W_{el.y}$ mm^3 x 10^3	$W_{pl.y}$ mm^3 x 10^3	i_y mm x 10	I_z mm^4 x 10^4	$W_{el.z}$ mm^3 x 10^3	$W_{pl.z}$ mm^3 x 10^3	i_z mm x 10	CL h	CL b	$N_{c.Rd}$ kN	$M_{c.y.Rd}$ kNm	$M_{c.z.Rd}$ kNm	$V_{pl.y.Rd}$ kN	$V_{pl.z.Rd}$ kN
300	100	5	30,1	38,36	0,783	204	2044	262,2	4065	271,0	348,2	10,29	722,8	144,6	159,6	4,34	4	2	1160	146,2	42,32	232,5	697,6
300	100	6	35,8	45,63	0,779	171	2403	306,2	4777	318,5	411,4	10,23	842,4	168,5	187,9	4,30	4	1	1525	172,8	55,19	276,6	829,9
300	100	7,1	41,7	53,05	0,770	145	2787	350,8	5424	361,6	472,1	10,11	954,1	190,8	215,5	4,24	4	1	1937	198,3	68,92	321,6	964,8
300	100	8	46,5	59,24	0,766	129	3080	385,2	5978	398,5	523,5	10,05	1045	209,0	238,3	4,20	4	1	2299	219,9	80,65	359,1	1077
300	100	8,8	50,8	64,64	0,762	118	3329	414,2	6446	429,7	567,6	9,99	1120	224,0	257,6	4,16	3	1	2715	238,4	94,10	391,9	1176
300	100	10	57,0	72,57	0,757	104	3681	454,5	7106	473,7	630,9	9,90	1224	244,9	285,3	4,11	2	1	3048	265,0	119,8	439,9	1320
300	100	12,5	68,3	87,04	0,736	85	4292	521,2	8010	534,0	731,9	9,59	1374	274,8	330,2	3,97	1	1	3656	307,4	138,7	527,7	1583
300	150	6	40,5	51,63	0,879	170	4988	478,6	6074	404,9	499,6	10,85	2080	277,3	309,5	6,35	4	1	1777	209,8	92,71	417,3	834,7
300	150	7,1	47,2	60,15	0,870	145	5834	553,0	6947	463,1	576,1	10,75	2378	317,1	357,0	6,29	4	1	2235	242,0	115,9	486,2	972,4
300	150	8	52,8	67,24	0,866	129	6491	611,5	7684	512,2	640,3	10,69	2623	349,7	396,4	6,25	4	1	2635	268,9	135,8	543,5	1087
300	150	10	64,8	82,57	0,857	104	7879	732,8	9209	614,0	775,9	10,56	3125	416,7	479,2	6,15	2	1	3468	325,9	201,3	667,4	1335
300	150	12,5	78,1	99,54	0,836	84	9452	861,8	10590	706,3	911,5	10,32	3595	479,3	563,4	6,01	1	1	4181	382,8	236,6	804,6	1609
300	200	6	45,2	57,63	0,979	170	8115	651,2	7370	491,4	587,8	11,31	3962	396,2	446,1	8,29	4	3	2000	206,4	134,8	559,0	838,5
300	200	7,1	52,8	67,25	0,970	144	9524	755,7	8470	564,7	680,1	11,22	4554	455,4	516,3	8,23	4	2	2533	285,6	168,1	652,3	978,5
300	200	8	59,1	75,24	0,966	128	10630	838,4	9389	626,0	757,1	11,17	5042	504,2	574,5	8,19	4	1	2971	318,0	196,9	729,8	1095
300	200	8,8	64,6	82,24	0,962	117	11590	909,5	10178	678,6	823,8	11,12	5459	545,9	624,9	8,15	3	1	3454	346,0	229,3	797,7	1197
300	200	10	72,7	92,57	0,957	103	12990	1012	11313	754,2	920,9	11,05	6058	605,8	698,1	8,09	2	1	3888	386,8	293,2	897,8	1347
300	200	12,5	88,0	112,0	0,936	84	15770	1204	13179	878,6	1091	10,85	7060	706,0	827,9	7,94	1	1	4706	458,3	347,7	1087	1630
400	200	6	54,7	69,63	1,179	169	12069	877,1	14789	739,5	906,0	14,57	5092	509,2	562,5	8,55	4	3	2067	310,6	145,1	562,8	1126
400	200	7,1	63,9	81,45	1,170	144	14169	1020	17070	853,5	1052	14,48	5875	587,5	653,2	8,49	4	2	2646	441,8	183,5	658,4	1317
400	200	8	71,6	91,24	1,166	128	15820	1133	18974	948,7	1173	14,42	6517	651,7	728,1	8,45	4	1	3135	492,8	217,5	737,5	1475
400	200	8,8	78,4	99,84	1,162	116	17260	1231	20619	1031	1279	14,37	7069	706,9	793,1	8,41	4	1	3587	537,2	248,5	807,0	1614
400	200	10	88,4	112,6	1,157	103	19368	1373	23003	1150	1434	14,30	7864	786,4	888,1	8,36	4	1	4287	602,2	295,7	909,9	1820
400	200	12,5	108	137,0	1,136	83	23594	1644	27101	1355	1714	14,06	9260	926,1	1062	8,22	3	1	5756	719,9	388,9	1108	2215

Table 11.1.7 Cross-sectional properties and resistance values for circular longitudinally welded hollow sections of steel grade S420MH (f_y = 420 N/mm^2)



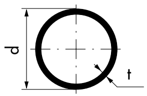
M = weight
A = cross-section area
A_u = external area
A_m/V = cross-section factor in fire design
I_t = St. Venant torsional constant
W_t = torsional section modulus
I = second moment of area
W_{el} = elastic section modulus
W_{pl} = plastic section modulus (shall be used only for CL1...2)
i = radius of gyration
CL = cross-section Class at uniform compression
$N_{c.Rd}$ = design compression resistance (without buckling)
$M_{c.Rd}$ = design bending resistance (respective to cross-section Class)
$V_{pl.Rd}$ = design plastic shear resistance (without shear buckling)

The calculated resistance values are design values (see Chapter 2) based on recommended partial safety factor values **γ_{M0} = 1,0** and **γ_{M1} = 1,0 (for Class 4 circular hollow sections γ_{M1} = 1,1)** as given in Eurocode 3 (EN 1993). Partial safety factor values may differ in each country. National values must be checked from the National Annex of the relevant country.

d mm	t mm	M kg/m	A mm^2 x 10^2	A_u m²/m	A_m/V 1/m	I_t mm^4 x 10^4	W_t mm^3 x 10^3	I mm^4 x 10^4	W_{el} mm^3 x 10^3	W_{pl} mm^3 x 10^3	i mm x 10	CL	$N_{c.Rd}$ kN	$M_{c.Rd}$ kNm	$V_{pl.Rd}$ kN
26,9	2	1,23	1,56	0,085	540	2,44	1,81	1,22	0,91	1,24	0,88	1	65,71	0,52	24,15
26,9	2,5	1,50	1,92	0,085	441	2,88	2,14	1,44	1,07	1,49	0,87	1	80,49	0,63	29,58
26,9	2,6	1,56	1,98	0,085	426	2,96	2,20	1,48	1,10	1,54	0,86	1	83,36	0,65	30,64
33,7	2	1,56	1,99	0,106	532	5,02	2,98	2,51	1,49	2,01	1,12	1	83,65	0,85	30,75
33,7	2,5	1,92	2,45	0,106	432	6,00	3,56	3,00	1,78	2,44	1,11	1	102,9	1,02	37,83
33,7	2,6	1,99	2,54	0,106	417	6,19	3,67	3,09	1,84	2,52	1,10	1	106,7	1,06	39,22
33,7	3	2,27	2,89	0,106	366	6,88	4,08	3,44	2,04	2,84	1,09	1	121,5	1,19	44,67
33,7	3,2	2,41	3,07	0,106	345	7,21	4,28	3,60	2,14	2,99	1,08	1	128,8	1,25	47,33
42,4	2	1,99	2,54	0,133	525	10,38	4,90	5,19	2,45	3,27	1,43	1	106,6	1,37	39,19
42,4	2,5	2,46	3,13	0,133	425	12,52	5,91	6,26	2,95	3,99	1,41	1	131,6	1,67	48,38
42,4	2,6	2,55	3,25	0,133	410	12,93	6,10	6,46	3,05	4,12	1,41	1	136,5	1,73	50,19
42,4	2,9	2,82	3,60	0,133	370	14,11	6,66	7,06	3,33	4,53	1,40	1	151,2	1,90	55,55
42,4	3	2,91	3,71	0,133	359	14,49	6,84	7,25	3,42	4,67	1,40	1	156,0	1,96	57,32
42,4	3,2	3,09	3,94	0,133	338	15,24	7,19	7,62	3,59	4,93	1,39	1	165,5	2,07	60,84
42,4	4	3,79	4,83	0,133	276	17,98	8,48	8,99	4,24	5,92	1,36	1	202,7	2,49	74,49
48,3	2	2,28	2,91	0,152	522	15,62	6,47	7,81	3,23	4,29	1,64	1	122,2	1,80	44,91
48,3	2,5	2,82	3,60	0,152	422	18,92	7,83	9,46	3,92	5,25	1,62	1	151,1	2,20	55,53
48,3	2,6	2,93	3,73	0,152	406	19,55	8,10	9,78	4,05	5,44	1,62	1	156,8	2,28	57,62
48,3	3	3,35	4,27	0,152	355	22,00	9,11	11,00	4,55	6,17	1,61	1	179,3	2,59	65,91
48,3	3,2	3,56	4,53	0,152	335	23,17	9,59	11,59	4,80	6,52	1,60	1	190,4	2,74	69,99
48,3	4	4,37	5,57	0,152	273	27,54	11,40	13,77	5,70	7,87	1,57	1	233,8	3,31	85,94

d mm	t mm	M kg/m	A mm^2 x 10^2	A_u m²/m	A_m/V 1/m	I_t mm^4 x 10^4	W_t mm^3 x 10^3	I mm^4 x 10^4	W_{el} mm^3 x 10^3	W_{pl} mm^3 x 10^3	i mm x 10	CL	$N_{c.Rd}$ kN	$M_{c.Rd}$ kNm	$V_{pl.Rd}$ kN
60,3	2	2,88	3,66	0,189	517	31,16	10,34	15,58	5,17	6,80	2,06	2	153,9	2,86	56,55
60,3	2,5	3,56	4,54	0,189	417	37,99	12,60	18,99	6,30	8,36	2,05	1	190,7	3,51	70,08
60,3	2,9	4,11	5,23	0,189	362	43,18	14,32	21,59	7,16	9,56	2,03	1	219,6	4,02	80,73
60,3	3	4,24	5,40	0,189	351	44,45	14,74	22,22	7,37	9,86	2,03	1	226,8	4,14	83,37
60,3	3,2	4,51	5,74	0,189	330	46,94	15,57	23,47	7,78	10,44	2,02	1	241,1	4,39	88,61
60,3	4	5,55	7,07	0,189	268	56,35	18,69	28,17	9,34	12,70	2,00	1	297,1	5,33	109,2
60,3	5	6,82	8,69	0,189	218	66,95	22,21	33,48	11,10	15,33	1,96	1	364,8	6,44	134,1
76,1	2	3,65	4,66	0,239	513	63,96	16,81	31,98	8,40	10,98	2,62	2	195,6	4,61	71,87
76,1	2,5	4,54	5,78	0,239	414	78,37	20,60	39,19	10,30	13,55	2,60	2	242,8	5,69	89,24
76,1	2,9	5,24	6,67	0,239	358	89,48	23,52	44,74	11,76	15,55	2,59	1	280,1	6,53	103,0
76,1	3	5,41	6,89	0,239	347	92,19	24,23	46,10	12,11	16,04	2,59	1	289,4	6,74	106,4
76,1	4	7,11	9,06	0,239	264	118,1	31,04	59,06	15,52	20,81	2,55	1	380,5	8,74	139,9
76,1	5	8,77	11,17	0,239	214	141,8	37,28	70,92	18,64	25,32	2,52	1	469,1	10,63	172,4
76,1	6	10,4	13,21	0,239	181	163,5	42,97	81,76	21,49	29,56	2,49	1	555,0	12,41	204,0
76,1	6,3	10,8	13,81	0,239	173	169,6	44,58	84,82	22,29	30,78	2,48	1	580,2	12,93	213,3
88,9	2	4,29	5,46	0,279	512	103,1	23,20	51,57	11,60	15,11	3,07	3	229,3	4,87	84,29
88,9	2,5	5,33	6,79	0,279	412	126,8	28,51	63,37	14,26	18,67	3,06	2	285,0	7,84	104,8
88,9	3	6,36	8,10	0,279	345	149,5	33,64	74,76	16,82	22,15	3,04	2	340,0	9,30	125,0
88,9	3,2	6,76	8,62	0,279	324	158,4	35,64	79,21	17,82	23,51	3,03	1	361,9	9,88	133,0
88,9	4	8,38	10,67	0,279	262	192,7	43,35	96,34	21,67	28,85	3,00	1	448,1	12,12	164,7
88,9	5	10,4	13,18	0,279	212	232,8	52,36	116,4	26,18	35,24	2,97	1	553,5	14,80	203,5
88,9	6	12,3	15,63	0,279	179	269,9	60,72	134,9	30,36	41,31	2,94	1	656,3	17,35	241,2
88,9	6,3	12,8	16,35	0,279	171	280,5	63,10	140,2	31,55	43,07	2,93	1	686,6	18,09	252,4
101,6	2	4,91	6,26	0,319	510	155,3	30,56	77,63	15,28	19,84	3,52	4	213,0	5,20	96,61
101,6	2,5	6,11	7,78	0,319	410	191,2	37,64	95,61	18,82	24,56	3,50	3	326,9	7,90	120,2
101,6	3	7,29	9,29	0,319	343	226,1	44,50	113,0	22,25	29,17	3,49	2	390,3	12,25	143,5
101,6	3,6	8,70	11,08	0,319	288	266,5	52,46	133,2	26,23	34,59	3,47	2	465,5	14,53	171,1
101,6	4	9,63	12,26	0,319	260	292,6	57,59	146,3	28,80	38,12	3,45	1	515,1	16,01	189,3
101,6	5	11,9	15,17	0,319	210	354,9	69,87	177,5	34,93	46,70	3,42	1	637,3	19,61	234,2
101,6	6	14,2	18,02	0,319	177	413,4	81,37	206,7	40,68	54,91	3,39	1	756,9	23,06	278,2
101,6	6,3	14,8	18,86	0,319	169	430,1	84,67	215,1	42,34	57,30	3,38	1	792,2	24,07	291,2
108	2	5,23	6,66	0,339	509	187,2	34,66	93,58	17,33	22,47	3,75	4	224,7	5,85	102,8
108	2,5	6,50	8,29	0,339	409	230,7	42,72	115,4	21,36	27,83	3,73	3	348,0	8,97	127,9
108	3	7,77	9,90	0,339	343	273,0	50,55	136,5	25,28	33,08	3,71	2	415,6	13,90	152,8
108	3,6	9,27	11,81	0,339	287	322,1	59,65	161,1	29,83	39,25	3,69	2	495,9	16,49	182,3
108	4	10,3	13,07	0,339	260	353,9	65,54	177,0	32,77	43,29	3,68	1	548,9	18,18	201,8
108	5	12,7	16,18	0,339	210	430,1	79,65	215,1	39,83	53,09	3,65	1	679,5	22,30	249,8
108	6	15,1	19,23	0,339	176	501,8	92,93	250,9	46,46	62,50	3,61	1	807,5	26,25	296,8
108	6,3	15,8	20,13	0,339	169	522,5	96,75	261,2	48,38	65,24	3,60	1	845,4	27,40	310,7

Table 11.1.7 Cross-sectional properties and resistance values for circular longitudinally welded hollow sections of steel grade S420MH (f_y = 420 N/mm^2) *cont.*



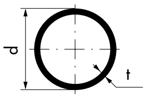
M = weight
A = cross-section area
A_u = external area
A_m/V = cross-section factor in fire design
I_t = St. Venant torsional constant
W_t = torsional section modulus
I = second moment of area
W_{el} = elastic section modulus
W_{pl} = plastic section modulus (shall be used only for CL1...2)
i = radius of gyration
CL = cross-section Class at uniform compression
$N_{c.Rd}$ = design compression resistance (without buckling)
$M_{c.Rd}$ = design bending resistance (respective to cross-section Class)
$V_{pl.Rd}$ = design plastic shear resistance (without shear buckling)

The calculated resistance values are design values (see Chapter 2) based on recommended partial safety factor values **γ_{M0} = 1,0** and **γ_{M1} = 1,0 (for Class 4 circular hollow sections γ_{M1} = 1,1)** as given in Eurocode 3 (EN 1993). Partial safety factor values may differ in each country. National values must be checked from the National Annex of the relevant country.

d mm	t mm	M kg/m	A mm² x 10²	A_u m²/m	A_m/V 1/m	I_t mm⁴ x 10⁴	W_t mm³ x 10³	I mm⁴ x 10⁴	W_{el} mm³ x 10³	W_{pl} mm³ x 10³	i mm x 10	CL	$N_{c.Rd}$ kN	$M_{c.Rd}$ kNm	$V_{pl.Rd}$ kN
114,3	2	5,54	7,06	0,359	509	222,5	38,94	111,3	19,47	25,23	3,97	4	236,0	6,51	108,9
114,3	2,5	6,89	8,78	0,359	409	274,5	48,03	137,3	24,02	31,25	3,95	3	368,8	10,09	135,6
114,3	3	8,23	10,49	0,359	342	325,1	56,88	162,6	28,44	37,17	3,94	2	440,6	15,61	161,9
114,3	3,6	9,83	12,52	0,359	287	384,0	67,19	192,0	33,59	44,13	3,92	2	525,8	18,54	193,3
114,3	4	10,9	13,86	0,359	259	422,1	73,86	211,1	36,93	48,69	3,90	2	582,2	20,45	214,0
114,3	5	13,5	17,17	0,359	209	513,8	89,91	256,9	44,96	59,77	3,87	1	721,0	25,11	265,0
114,3	6	16,0	20,41	0,359	176	600,4	105,1	300,2	52,53	70,45	3,83	1	857,4	29,59	315,1
114,3	6,3	16,8	21,38	0,359	168	625,4	109,4	312,7	54,72	73,57	3,82	1	897,8	30,90	330,0
127	2	6,17	7,85	0,399	508	306,9	48,33	153,4	24,16	31,25	4,42	4	258,2	7,94	121,2
127	2,5	7,68	9,78	0,399	408	379,1	59,70	189,5	29,85	38,76	4,40	4	332,9	10,16	151,0
127	3	9,17	11,69	0,399	341	449,5	70,79	224,8	35,39	46,14	4,39	3	490,8	14,87	180,4
127	4	12,1	15,46	0,399	258	585,2	92,16	292,6	46,08	60,54	4,35	2	649,2	25,43	238,6
127	5	15,0	19,16	0,399	208	714,3	112,5	357,1	56,24	74,46	4,32	1	804,9	31,27	295,8
127	6	17,9	22,81	0,399	175	836,9	131,8	418,4	65,90	87,92	4,28	1	957,9	36,93	352,1
127	6,3	18,8	23,89	0,399	167	872,4	137,4	436,2	68,70	91,86	4,27	1	1003	38,58	368,8
133	2	6,46	8,23	0,418	508	353,2	53,11	176,6	26,56	34,32	4,63	4	268,4	8,66	127,1
133	2,5	8,05	10,25	0,418	408	436,5	65,64	218,3	32,82	42,58	4,61	4	346,5	11,10	158,2
133	3	9,62	12,25	0,418	341	517,9	77,88	259,0	38,94	50,71	4,60	3	514,6	16,36	189,1
133	4	12,7	16,21	0,418	258	675,1	101,5	337,5	50,76	66,59	4,56	2	680,9	27,97	250,3
133	5	15,8	20,11	0,418	208	824,8	124,0	412,4	62,02	81,96	4,53	1	844,5	34,42	310,4
133	6	18,8	23,94	0,418	175	967,4	145,5	483,7	72,74	96,85	4,50	1	1005	40,68	369,6
133	6,3	19,7	25,08	0,418	167	1009	151,7	504,4	75,85	101,2	4,49	1	1053	42,51	387,1

d mm	t mm	M kg/m	A mm^2 x 10^2	A_u m²/m	A_m/V 1/m	I_t mm^4 x 10^4	W_t mm^3 x 10^3	I mm^4 x 10^4	W_{el} mm^3 x 10^3	W_{pl} mm^3 x 10^3	i mm x 10	CL	$N_{c.Rd}$ kN	$M_{c.Rd}$ kNm	$V_{pl.Rd}$ kN
139,7	3	10,1	12,88	0,439	341	602,2	86,21	301,1	43,11	56,07	4,83	3	541,1	18,10	198,9
139,7	4	13,4	17,05	0,439	257	785,7	112,5	392,9	56,24	73,68	4,80	2	716,2	30,95	263,2
139,7	5	16,6	21,16	0,439	207	961,1	137,6	480,5	68,80	90,76	4,77	1	888,7	38,12	326,6
139,7	6	19,8	25,20	0,439	174	1129	161,6	564,3	80,78	107,3	4,73	1	1058	45,08	389,1
139,7	6,3	20,7	26,40	0,439	166	1177	168,5	588,6	84,27	112,2	4,72	1	1109	47,12	407,6
139,7	8	26,0	33,10	0,439	133	1441	206,2	720,3	103,1	138,9	4,66	1	1390	58,35	511,0
139,7	10	32,0	40,75	0,439	108	1724	246,8	861,9	123,4	168,6	4,60	1	1711	70,79	629,0
152,4	3	11,1	14,08	0,479	340	786,0	103,2	393,0	51,58	66,97	5,28	4	479,3	17,56	217,4
152,4	4	14,6	18,65	0,479	257	1027	134,8	513,7	67,42	88,11	5,25	2	783,2	37,01	287,9
152,4	5	18,2	23,15	0,479	207	1259	165,2	629,5	82,62	108,7	5,21	2	972,5	45,64	357,4
152,4	6	21,7	27,60	0,479	173	1481	194,4	740,6	97,19	128,7	5,18	1	1159	54,04	426,0
152,4	6,3	22,7	28,92	0,479	166	1546	202,9	773,0	101,4	134,6	5,17	1	1214	56,51	446,4
159	3	11,5	14,70	0,500	340	894,8	112,6	447,4	56,28	73,02	5,52	4	497,4	19,04	227,0
159	4	15,3	19,48	0,500	256	1171	147,3	585,3	73,63	96,12	5,48	3	818,1	30,92	300,7
159	5	19,0	24,19	0,500	206	1436	180,6	717,9	90,30	118,6	5,45	2	1016	49,82	373,4
159	6	22,6	28,84	0,500	173	1690	212,6	845,2	106,3	140,5	5,41	1	1211	59,02	445,2
159	6,3	23,7	30,22	0,500	165	1765	222,0	882,4	111,0	147,0	5,40	1	1269	61,73	466,6
168,3	3	12,2	15,58	0,529	339	1065	126,5	532,3	63,25	81,98	5,85	4	522,5	21,22	240,5
168,3	3,2	13,0	16,60	0,529	319	1131	134,5	565,7	67,23	87,24	5,84	4	562,1	22,77	256,2
168,3	4	16,2	20,65	0,529	256	1394	165,7	697,1	82,84	108,0	5,81	3	867,2	34,79	318,7
168,3	4,5	18,2	23,16	0,529	228	1554	184,7	777,2	92,36	120,8	5,79	2	972,6	50,72	357,5
168,3	5	20,1	25,65	0,529	206	1712	203,4	855,9	101,7	133,4	5,78	2	1077	56,02	396,0
168,3	6	24,0	30,59	0,529	173	2017	239,7	1009	119,9	158,1	5,74	2	1285	66,41	472,3
168,3	6,3	25,2	32,06	0,529	165	2107	250,4	1053	125,2	165,4	5,73	1	1347	69,48	495,0
168,3	8	31,6	40,29	0,529	131	2595	308,3	1297	154,2	205,7	5,67	1	1692	86,41	621,9
168,3	10	39,0	49,73	0,529	106	3128	371,7	1564	185,9	250,9	5,61	1	2089	105,4	767,7
193,7	4	18,7	23,84	0,609	255	2146	221,5	1073	110,8	144,0	6,71	3	1001	46,52	368,0
193,7	5	23,3	29,64	0,609	205	2640	272,6	1320	136,3	178,1	6,67	2	1245	74,79	457,6
193,7	6	27,8	35,38	0,609	172	3119	322,1	1560	161,1	211,5	6,64	2	1486	88,81	546,2
193,7	6,3	29,1	37,09	0,609	164	3260	336,6	1630	168,3	221,3	6,63	2	1558	92,96	572,6
193,7	8	36,6	46,67	0,609	130	4031	416,2	2016	208,1	276,1	6,57	1	1960	115,9	720,5
193,7	10	45,3	57,71	0,609	105	4883	504,2	2442	252,1	337,8	6,50	1	2424	141,9	891,0
193,7	12,5	55,9	71,16	0,609	86	5869	606,0	2934	303,0	411,1	6,42	1	2989	172,7	1098

Table 11.1.7 Cross-sectional properties and resistance values for circular longitudinally welded hollow sections of steel grade S420MH (f_y = 420 N/mm^2) *cont.*

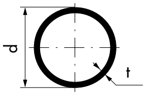


M = weight
A = cross-section area
A_u = external area
A_m/V = cross-section factor in fire design
I_t = St. Venant torsional constant
W_t = torsional section modulus
I = second moment of area
W_{el} = elastic section modulus
W_{pl} = plastic section modulus (shall be used only for CL1...2)
i = radius of gyration
CL = cross-section Class at uniform compression
$N_{c.Rd}$ = design compression resistance (without buckling)
$M_{c.Rd}$ = design bending resistance (respective to cross-section Class)
$V_{pl.Rd}$ = design plastic shear resistance (without shear buckling)

The calculated resistance values are design values (see Chapter 2) based on recommended partial safety factor values **γ_{M0} = 1,0** and **γ_{M1} = 1,0 (for Class 4 circular hollow sections γ_{M1} = 1,1)** as given in Eurocode 3 (EN 1993). Partial safety factor values may differ in each country. National values must be checked from the National Annex of the relevant country.

d mm	t mm	M kg/m	A mm^2 x 10^2	A_u m²/m	A_m/V 1/m	I_t mm^4 x 10^4	W_t mm^3 x 10^3	I mm^4 x 10^4	W_{el} mm^3 x 10^3	W_{pl} mm^3 x 10^3	i mm x 10	CL	$N_{c.Rd}$ kN	$M_{c.Rd}$ kNm	$V_{pl.Rd}$ kN
219,1	4	21,2	27,03	0,688	255	3128	285,5	1564	142,8	185,1	7,61	4	909,9	48,05	417,3
219,1	4,5	23,8	30,34	0,688	227	3494	319,0	1747	159,5	207,3	7,59	3	1274	66,99	468,3
219,1	5	26,4	33,63	0,688	205	3856	352,0	1928	176,0	229,2	7,57	3	1412	73,92	519,2
219,1	6	31,5	40,17	0,688	171	4564	416,6	2282	208,3	272,5	7,54	2	1687	114,5	620,1
219,1	6,3	33,1	42,12	0,688	163	4772	435,6	2386	217,8	285,4	7,53	2	1769	120,0	650,2
219,1	8	41,7	53,06	0,688	130	5919	540,3	2960	270,2	356,7	7,47	1	2228	149,8	819,0
219,1	10	51,6	65,69	0,688	105	7197	657,0	3598	328,5	437,6	7,40	1	2759	183,8	1014
219,1	12,5	63,7	81,13	0,688	85	8689	793,2	4345	396,6	534,2	7,32	1	3408	224,4	1252
244,5	6	35,3	44,96	0,768	171	6397	523,3	3199	261,6	341,4	8,43	3	1888	110,0	694,0
244,5	8	46,7	59,44	0,768	129	8321	680,7	4160	340,3	447,6	8,37	2	2496	188,0	917,6
244,5	10	57,8	73,67	0,768	104	10150	830,0	5073	415,0	550,2	8,30	1	3094	231,1	1137
244,5	12,5	71,5	91,11	0,768	84	12290	1006	6147	502,9	673,5	8,21	1	3826	282,9	1406
273	4	26,5	33,80	0,858	254	6117	448,1	3058	224,1	289,5	9,51	4	1097	72,71	521,8
273	5	33,1	42,10	0,858	204	7562	554,0	3781	277,0	359,2	9,48	4	1418	93,29	649,9
273	6	39,5	50,33	0,858	170	8974	657,5	4487	328,7	427,8	9,44	3	2114	138,1	776,9
273	6,3	41,4	52,79	0,858	162	9392	688,0	4696	344,0	448,2	9,43	3	2217	144,5	814,9
273	8	52,3	66,60	0,858	129	11703	857,4	5852	428,7	562,0	9,37	2	2797	236,0	1028
273	10	64,9	82,62	0,858	104	14308	1048	7154	524,1	692,0	9,31	1	3470	290,7	1275
273	12,5	80,3	102,3	0,858	84	17395	1274	8697	637,2	848,9	9,22	1	4297	356,5	1579
323,9	4	31,6	40,20	1,018	253	10286	635,2	5143	317,6	409,4	11,31	4	1262	99,66	620,6
323,9	5	39,3	50,09	1,018	203	12739	786,6	6369	393,3	508,5	11,28	4	1641	128,8	773,3
323,9	6	47,0	59,92	1,018	170	15145	935,2	7572	467,6	606,4	11,24	4	2022	157,8	925,0
323,9	6,3	49,3	62,86	1,018	162	15858	979,2	7929	489,6	635,6	11,23	4	2136	166,4	970,4
323,9	8	62,3	79,39	1,018	128	19820	1224	9910	611,9	798,5	11,17	3	3335	257,0	1226
323,9	10	77,4	98,61	1,018	103	24317	1501	12158	750,8	985,7	11,10	2	4142	414,0	1522
323,9	12,5	96,0	122,3	1,018	83	29693	1833	14847	916,7	1213	11,02	1	5136	509,4	1888

Table 11.1.8 Cross-sectional properties and resistance values for circular spirally welded hollow sections of steel grade S420MH (f_y = 420 N/mm^2)
(Technical delivery conditions to be agreed when ordering)



M = weight
A = cross-section area
A_u = external area
A_m/V = cross-section factor in fire design
I_t = St. Venant torsional constant
W_t = torsional section modulus
I = second moment of area
W_{el} = elastic section modulus
W_{pl} = plastic section modulus (shall be used only for CL1...2)
i = radius of gyration
CL = cross-section Class at uniform compression
$N_{c.Rd}$ = design compression resistance (without buckling)
$M_{c.Rd}$ = design bending resistance (respective to cross-section Class)
$V_{pl.Rd}$ = design plastic shear resistance (without shear buckling)

The calculated resistance values are design values (see Chapter 2) based on recommended partial safety factor values **γ_{M0} = 1,0** and **γ_{M1} = 1,0 (for Class 4 circular hollow sections γ_{M1} = 1,1)** as given in Eurocode 3 (EN 1993). Partial safety factor values may differ in each country. National values must be checked from the National Annex of the relevant country.

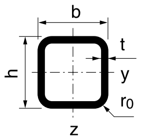
d mm	t mm	M kg/m	A mm^2 x 10^2	A_u m²/m	A_m/V 1/m	I_t mm^4 x 10^4	W_t mm^3 x 10^3	I mm^4 x 10^4	W_{el} mm^3 x 10^3	W_{pl} mm^3 x 10^3	i mm x 10	CL	$N_{c.Rd}$ kN	$M_{c.Rd}$ kNm	$V_{pl.Rd}$ kN
406,4	6,3	62,2	79,19	1,277	161	31699	1560	15849	780,0	1009	14,15	4	2596	255,7	1222
406,4	8	78,6	100,1	1,277	128	39748	1956	19874	978,1	1270	14,09	4	3408	332,9	1546
406,4	10	97,8	124,5	1,277	103	48952	2409	24476	1205	1572	14,02	3	5230	505,9	1922
406,4	12,5	121	154,7	1,277	83	60061	2956	30031	1478	1940	13,93	2	6497	814,9	2388
457	6,3	70,0	89,20	1,436	161	45308	1983	22654	991,4	1280	15,94	4	2862	318,1	1377
457	8	88,6	112,9	1,436	127	56893	2490	28446	1245	1613	15,88	4	3774	416,4	1742
457	10	110	140,4	1,436	102	70183	3071	35091	1536	1998	15,81	3	5898	645,0	2168
457	12	132	167,8	1,436	86	83113	3637	41556	1819	2377	15,74	2	7046	998,3	2590
457	12,5	137	174,6	1,436	82	86290	3776	43145	1888	2470	15,72	2	7331	1038	2695
508	6,3	78,0	99,30	1,596	161	62493	2460	31246	1230	1586	17,74	4	3119	386,4	1533
508	8	98,7	125,7	1,596	127	78560	3093	39280	1546	2000	17,68	4	4130	508,3	1940
508	10	123	156,5	1,596	102	97040	3820	48520	1910	2480	17,61	4	5326	650,2	2415
508	12,5	153	194,6	1,596	82	119511	4705	59755	2353	3070	17,52	3	8172	988,1	3004
559	6,3	85,9	109,4	1,756	161	83552	2989	41776	1495	1925	19,54	4	3364	459,7	1689
559	8	109	138,5	1,756	127	105130	3761	52565	1881	2429	19,48	4	4475	607,7	2138
559	10	135	172,5	1,756	102	130002	4651	65001	2326	3014	19,41	4	5788	780,4	2663
559	12,5	168	214,6	1,756	82	160324	5736	80162	2868	3734	19,33	3	9014	1205	3313
610	8	119	151,3	1,916	127	137103	4495	68551	2248	2899	21,29	4	4807	714,1	2336
610	10	148	188,5	1,916	102	169693	5564	84847	2782	3600	21,22	4	6238	920,6	2910
610	12,5	184	234,6	1,916	82	209509	6869	104755	3435	4463	21,13	3	9855	1443	3622
610	14,2	209	265,8	1,916	72	236008	7738	118004	3869	5042	21,07	3	11163	1625	4103

d mm	t mm	M kg/m	A mm^2 $\times 10^2$	A_u m²/m	A_m/V 1/m	I_t mm^4 $\times 10^4$	W_t mm^3 $\times 10^3$	I mm^4 $\times 10^4$	W_{el} mm^3 $\times 10^3$	W_{pl} mm^3 $\times 10^3$	i mm $\times 10$	CL	$N_{c.Rd}$ kN	$M_{c.Rd}$ kNm	$V_{pl.Rd}$ kN
660	8	129	163,9	2,073	127	174176	5278	87088	2639	3401	23,05	4	5122	824,9	2530
660	10	160	204,2	2,073	102	215741	6538	107870	3269	4225	22,98	4	6667	1067	3152
660	12,5	200	254,3	2,073	82	266613	8079	133306	4040	5241	22,90	4	8607	1367	3925
660	14,2	226	288,1	2,073	72	300526	9107	150263	4553	5923	22,84	3	12100	1912	4447
711	8	139	176,7	2,234	126	218324	6141	109162	3071	3954	24,86	4	5432	944,0	2727
711	10	173	220,2	2,234	101	270603	7612	135301	3806	4914	24,79	4	7093	1226	3400
711	12,5	215	274,3	2,234	81	334686	9415	167343	4707	6099	24,70	4	9180	1575	4234
711	14,2	244	310,9	2,234	72	377470	10618	188735	5309	6895	24,64	3	13056	2230	4799
762	8	149	189,5	2,394	126	269366	7070	134683	3535	4548	26,66	4	5731	1069	2925
762	10	185	236,3	2,394	101	334057	8768	167028	4384	5655	26,59	4	7507	1393	3647
762	12,5	231	294,3	2,394	81	413462	10852	206731	5426	7023	26,50	4	9741	1796	4544
762	14,2	262	333,6	2,394	72	466542	12245	233271	6123	7942	26,44	4	11265	2067	5150
813	8	159	202,3	2,554	126	327801	8064	163901	4032	5184	28,46	4	6019	1200	3123
813	10	198	252,3	2,554	101	406728	10006	203364	5003	6448	28,39	4	7910	1569	3894
813	12,5	247	314,4	2,554	81	503721	12392	251860	6196	8011	28,31	4	10290	2028	4853
813	14,2	280	356,4	2,554	72	568630	13988	284315	6994	9062	28,25	4	11914	2338	5501
813	16	314	400,6	2,554	64	636443	15657	318222	7828	10165	28,18	4	13636	2665	6184
914	10	223	284,0	2,871	101	580294	12698	290147	6349	8172	31,96	4	8674	1939	4384
914	12,5	278	354,0	2,871	81	719417	15742	359708	7871	10159	31,88	4	11341	2521	5465
914	14,2	315	401,4	2,871	72	812689	17783	406344	8892	11498	31,82	4	13163	2916	6197
914	16	354	451,4	2,871	64	910283	19919	455142	9959	12904	31,75	4	15096	3331	6968
1016	10	248	316,0	3,192	101	799699	15742	399850	7871	10121	35,57	4	9403	2342	4879
1016	12,5	309	394,1	3,192	81	992246	19532	496123	9766	12588	35,48	4	12357	3062	6083
1016	14,2	351	446,9	3,192	71	1121520	22077	560762	11039	14252	35,42	4	14376	3551	6899
1016	16	395	502,7	3,192	64	1256959	24743	628479	12372	16001	35,36	4	16521	4066	7760
1219	10	298	379,8	3,830	101	1388029	22773	694014	11387	14617	42,75	4	10728	3216	5863
1219	12,5	372	473,8	3,830	81	1724362	28291	862181	14146	18196	42,66	4	14245	4253	7314
1219	14,2	422	537,5	3,830	71	1950668	32004	975334	16002	20613	42,60	4	16654	4958	8297
1219	16	475	604,7	3,830	63	2188182	35901	1094091	17951	23157	42,54	4	19215	5704	9335

Annex 11.2 Buckling tables for steel grades S355J2H and S420MH

Table	Steel grade		Shape
11.2.1	S355J2H		Square
11.2.2	S355J2H		Rectangular
11.2.3	S355J2H		Circular longitudinally welded
11.2.4	S355J2H		Circular sprirally welded
11.2.5		S420MH	Square
11.2.6		S420MH	Rectangular
11.2.7		S420MH	Circular longitudinally welded
11.2.8		S420MH	Circular sprirally welded

Table 11.2.1 Buckling resistance values for square hollow sections of steel grade S355J2H (f_y = 355 N/mm^2)



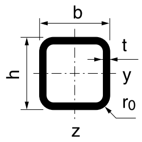
h = depth
b = width
t = wall thickness
L_{cr} = buckling length
$N_{b.Rd}$ = buckling resistance

The calculated resistance values are design values (see Chapter 2) based on recommended partial safety factor values **γ_{M0} = 1,0** and **γ_{M1} = 1,0 (for Class 4 circular hollow sections γ_{M1} = 1,1)** as given in Eurocode 3 (EN 1993). Partial safety factor values may differ in each country. National values must be checked from the National Annex of the relevant country.

h mm	b mm	t mm	$N_{b.Rd}$ (kN) L_{cr} (m) 0	0,5	1	1,5	2	2,5	3	3,5	4	4,5	5	6	7	8	9	10	h mm	b mm	t mm
25	25	2	61,66	44,38	21,17	10,90	6,52	4,31											25	25	2
25	25	2,5	74,16	52,47	24,44	12,50	7,46	4,93											25	25	2,5
25	25	3	85,49	59,38	26,98	13,71	8.161	5,39											25	25	3
30	30	2	75,86	60,49	34,40	18,83	11,49	7,68	5,48										30	30	2
30	30	2,5	91,91	72,58	40,51	22,00	13,39	8,94	6,38										30	30	2,5
30	30	3	106,8	83,47	45,69	24,61	14,94	9,96	7,10										30	30	3
40	40	2	104,3	92,13	65,68	41,56	26,80	18,37	13,29	10,04	7,84								40	40	2
40	40	2,5	127,4	112,1	79,18	49,67	31,91	21,83	15,78	11,91	9,30								40	40	2,5
40	40	3	149,4	130,9	91,52	56,91	36,42	24,87	17,97	13,56	10,58								40	40	3
40	40	4	189,9	164,8	112,8	68,88	43,75	29,78	21,47	16,19	12,63								40	40	4
50	50	2	132,7	123,4	98,38	70,94	49,12	34,80	25,63	19,57	15,39	12,41							50	50	2
50	50	2,5	162,9	151,2	120,0	85,93	59,23	41,88	30,80	23,50	18,48	14,89							50	50	2,5
50	50	3	192,0	177,8	140,3	99,85	68,52	48,33	35,51	27,07	21,28	17,15							50	50	3
50	50	4	246,7	227,3	177,5	124,5	84,68	59,47	43,60	33,19	26,06	20,99							50	50	4
50	50	5	296,6	271,8	209,9	145,0	97,75	68,36	50,00	38,02	29,83	24,00							50	50	5
60	60	2	161,1	154,7	130,7	103,2	76,98	56,86	42,82	33,12	26,27	21,31	17,61						60	60	2
60	60	2,5	198,4	190,3	160,3	126,1	93,64	68,98	51,88	40,09	31,79	25,77	21,29						60	60	2,5
60	60	3	234,6	224,6	188,7	147,8	109,3	80,30	60,30	46,56	36,90	29,90	24,70						60	60	3
60	60	4	303,5	289,6	241,9	187,7	137,6	100,6	75,32	58,06	45,96	37,22	30,73						60	60	4
60	60	5	367,6	349,6	290,3	223,0	162,1	117,8	87,99	67,72	53,55	43,33	35,76						60	60	5
70	70	2,5	233,9	229,3	200,1	167,1	132,4	102,0	78,82	61,89	49,57	40,46	33,59	24,16					70	70	2,5
70	70	3	277,2	271,4	236,4	196,9	155,5	119,5	92,17	72,30	57,87	47,21	39,19	28,18					70	70	3
70	70	4	360,3	351,9	305,5	252,9	198,3	151,6	116,5	91,24	72,93	59,45	49,32	35,43					70	70	4
70	70	5	438,6	427,5	369,8	304,2	236,8	180,1	137,9	107,8	86,03	70,07	58,09	41,70					70	70	5
80	80	2,5	269,4	268,3	239,5	207,8	173,0	139,3	110,9	88,73	71,92	59,18	49,41	35,81	27,07				80	80	2,5
80	80	3	319,8	318,2	283,7	245,7	204,1	164,0	130,3	104,1	84,31	69,34	57,88	41,92	31,68				80	80	3
80	80	4	417,1	414,3	368,5	317,9	262,7	210,0	166,3	132,6	107,2	88,08	73,46	53,16	40,15				80	80	4
80	80	5	509,6	505,4	448,5	385,3	316,7	251,9	198,7	158,0	127,6	104,8	87,31	63,12	47,65				80	80	5
80	80	6	597,6	591,5	523,6	448,0	366,1	289,7	227,6	180,7	145,7	119,4	99,49	71,86	54,22				80	80	6

h	b	t	$N_{b.Rd}$ (kN)																h	b	t
			L_{cr} (m)																		
mm	mm	mm	0	0,5	1	1,5	2	2,5	3	3,5	4	4,5	5	6	7	8	9	10	mm	mm	mm
90	90	2,5	304,9	304,9	278,7	248,0	214,1	179,1	146,9	120,1	98,67	81,96	68,88	50,34	38,25	30,00			90	90	2,5
90	90	3	362,4	362,4	330,8	294,0	253,3	211,5	173,2	141,3	116,1	96,34	80,94	59,12	44,90	35,21			90	90	3
90	90	4	473,9	473,9	431,2	382,3	328,2	272,9	222,7	181,2	148,6	123,2	103,4	75,45	57,28	44,89			90	90	4
90	90	5	580,6	580,6	526,8	465,7	398,3	329,7	268,1	217,7	178,1	147,5	123,8	90,19	68,43	53,61			90	90	5
90	90	6	682,8	682,8	617,5	544,4	463,6	382,1	309,6	250,7	204,9	169,5	142,0	103,4	78,40	61,40			90	90	6
100	100	2,5	309,5	309,5	291,4	265,6	237,6	207,4	177,2	149,5	125,8	106,2	90,26	66,92	51,29	40,45	32,67		100	100	2,5
100	100	3	405,0	405,0	377,7	341,8	302,4	260,4	219,5	183,0	152,6	128,0	108,4	79,91	61,05	48,04	38,75		100	100	3
100	100	4	530,7	530,7	493,8	446,0	393,5	337,8	283,8	236,1	196,5	164,6	139,2	102,5	78,26	61,56	49,64		100	100	4
100	100	5	651,6	651,6	605,0	545,4	479,8	410,5	343,7	285,2	236,9	198,3	167,5	123,2	93,99	73,90	59,57		100	100	5
100	100	6	768,0	768,0	711,2	639,9	561,4	478,6	399,4	330,5	274,1	229,0	193,3	142,0	108,3	85,11	68,58		100	100	6
100	100	7,1	875,2	875,2	807,1	723,7	631,7	535,3	444,2	366,0	302,6	252,4	212,7	156,0	118,8	93,29	75,13		100	100	7,1
100	100	8	967,1	967,1	889,5	795,8	692,4	584,4	483,3	397,2	327,8	273,0	229,9	168,4	128,1	100,6	81,00		100	100	8
100	100	10	1156	1156	1057	940,1	811,4	678,7	556,7	454,8	373,8	310,4	260,9	190,6	144,9	113,6	91,40		100	100	10
110	110	2,5	320,5	320,5	308,0	285,1	260,5	234,0	206,3	179,2	154,2	132,5	114,1	85,98	66,54	52,80	42,83	35,40	110	110	2,5
110	110	3	447,6	447,6	424,6	389,3	351,0	309,7	267,6	228,1	193,3	164,1	140,1	104,4	80,24	63,40	51,28	42,30	110	110	3
110	110	4	587,5	587,5	556,3	509,3	458,3	403,4	347,6	295,5	250,0	211,9	180,7	134,5	103,3	81,58	65,96	54,39	110	110	4
110	110	5	722,6	722,6	683,0	624,5	560,8	492,3	423,1	358,7	302,9	256,4	218,4	162,3	124,6	98,35	79,49	65,54	110	110	5
110	110	6	853,2	853,2	804,8	734,8	658,5	576,5	494,0	417,8	352,0	297,6	253,2	188,0	144,2	113,8	91,91	75,75	110	110	6
120	120	3	445,7	445,7	431,5	401,4	369,5	335,0	298,7	262,3	228,1	197,5	171,1	130,0	101,0	80,43	65,37	54,11	120	120	3
120	120	4	644,3	644,3	618,7	572,3	522,6	468,9	412,9	358,1	307,9	264,3	227,4	171,3	132,5	105,1	85,23	70,44	120	120	4
120	120	5	793,6	793,6	761,0	703,2	641,1	574,1	504,3	436,4	374,6	321,0	275,9	207,5	160,4	127,1	103,1	85,16	120	120	5
120	120	5,6	881,0	881,0	844,0	779,4	709,9	634,9	557,0	481,3	412,6	353,3	303,5	228,0	176,1	139,6	113,1	93,47	120	120	5,6
120	120	6	938,4	938,4	898,4	829,2	754,9	674,5	591,1	510,3	437,2	374,1	321,2	241,2	186,2	147,6	119,6	98,79	120	120	6
120	120	7,1	1077	1077	1028	947,3	860,1	765,9	668,6	575,0	491,0	419,2	359,3	269,2	207,6	164,3	133,1	109,9	120	120	7,1
120	120	8	1194	1194	1139	1048	949,6	843,7	734,6	630,3	537,2	457,9	392,0	293,3	226,0	178,8	144,8	119,5	120	120	8
120	120	8,8	1295	1295	1233	1133	1025	909,1	789,7	676,1	575,2	489,7	418,9	313,0	241,0	190,6	154,3	127,3	120	120	8,8
120	120	10	1440	1440	1368	1255	1133	1001	866,3	739,2	627,2	532,9	455,1	339,5	261,1	206,4	166,9	137,7	120	120	10
140	140	4	757,9	757,9	743,6	697,8	649,9	598,7	544,2	488,0	432,7	380,9	334,4	258,8	203,4	163,0	133,1	110,5	140	140	4
140	140	5	935,6	935,6	917,1	859,9	800,2	736,3	668,2	598,2	529,5	465,5	408,2	315,3	247,6	198,3	161,8	134,4	140	140	5
140	140	5,6	1040	1040	1019	954,9	888,1	816,5	740,4	662,1	585,5	514,2	450,6	347,7	272,8	218,4	178,3	148,0	140	140	5,6
140	140	6	1109	1109	1086	1017	945,7	869,0	787,5	703,7	621,9	545,9	478,1	368,7	289,2	231,5	188,9	156,7	140	140	6
140	140	7,1	1278	1278	1250	1169	1086	995,5	899,8	801,9	706,7	618,9	541,0	416,1	325,8	260,5	212,4	176,2	140	140	7,1
140	140	8	1422	1422	1388	1298	1204	1102	994,6	884,8	778,4	680,7	594,3	456,3	357,0	285,3	232,5	192,8	140	140	8
140	140	8,8	1545	1545	1507	1408	1305	1193	1075	954,9	838,8	732,5	638,9	489,8	382,9	305,8	249,1	206,6	140	140	8,8
140	140	10	1724	1724	1679	1567	1450	1324	1190	1054	923,6	804,9	700,9	536,2	418,6	334,0	272,0	225,4	140	140	10

Table 11.2.1 Buckling resistance values for square hollow sections of steel grade S355J2H (f_y = 355 N/mm^2) *continued*



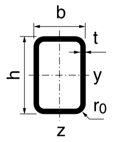
h = depth
b = width
t = wall thickness
L_{cr} = buckling length
$N_{b.Rd}$ = buckling resistance

The calculated resistance values are design values (see Chapter 2) based on recommended partial safety factor values **γ_{M0} = 1,0** and **γ_{M1} = 1,0 (for Class 4 circular hollow sections γ_{M1} = 1,1)** as given in Eurocode 3 (EN 1993). Partial safety factor values may differ in each country. National values must be checked from the National Annex of the relevant country.

h mm	b mm	t mm	$N_{b.Rd}$ (kN), L_{cr} (m): 0	0,5	1	1,5	2	2,5	3	3,5	4	4,5	5	6	7	8	9	10	h mm	b mm	t mm
150	150	4	771,4	771,4	765,6	723,5	680,1	634,3	585,5	534,4	482,5	431,9	384,7	303,9	242,0	195,6	160,6	133,9	150	150	4
150	150	5	1007	1007	995,2	938,1	879,2	816,6	749,9	680,3	610,3	543,1	481,1	377,1	298,8	240,7	197,2	164,2	150	150	5
150	150	6	1194	1194	1179	1111	1040	965,4	885,4	802,0	718,5	638,5	564,9	442,1	349,9	281,6	230,7	192,0	150	150	6
150	150	7,1	1379	1379	1360	1280	1197	1109	1015	917,6	820,0	727,1	642,1	501,1	395,9	318,3	260,5	216,7	150	150	7,1
150	150	8	1535	1535	1512	1423	1330	1230	1125	1015	905,6	801,8	707,3	551,0	434,9	349,4	285,9	237,7	150	150	8
150	150	8,8	1670	1670	1644	1546	1443	1334	1218	1098	978,2	865,0	762,2	592,9	467,5	375,4	307,0	255,2	150	150	8,8
150	150	10	1866	1866	1834	1723	1607	1484	1352	1216	1081	954,1	839,3	651,3	512,9	411,5	336,3	279,4	150	150	10
150	150	12,5	2203	2203	2156	2019	1877	1724	1561	1394	1232	1081	945,9	729,0	571,7	457,4	373,2	309,7	150	150	12,5
160	160	4	792,3	792,3	792,3	753,9	713,6	671,4	626,8	579,7	531,0	482,3	435,2	351,0	283,4	231,1	190,9	159,8	160	160	4
160	160	5	1078	1078	1073	1016	957,9	896,4	831,0	762,3	692,1	622,9	557,4	443,6	354,9	287,7	236,7	197,7	160	160	5
160	160	6	1279	1279	1273	1205	1135	1061	982,7	900,4	816,4	733,8	655,9	521,0	416,4	337,3	277,4	231,5	160	160	6
160	160	7,1	1480	1480	1471	1391	1309	1222	1130	1034	935,2	838,9	748,4	592,9	473,0	382,6	314,4	262,3	160	160	7,1
160	160	8	1649	1649	1637	1547	1455	1358	1254	1145	1035	927,2	826,3	653,4	520,6	420,9	345,7	288,3	160	160	8
160	160	8,8	1795	1795	1781	1683	1582	1475	1361	1241	1120	1002	892,3	704,5	560,8	453,0	372,0	310,1	160	160	8,8
160	160	10	2008	2008	1990	1879	1764	1643	1514	1378	1242	1109	985,7	776,4	617,1	498,1	408,7	340,5	160	160	10
160	160	12,5	2380	2380	2350	2214	2073	1923	1763	1596	1430	1270	1123	878,6	695,2	559,5	458,2	381,3	160	160	12,5
180	180	5	1220	1220	1220	1172	1115	1055	992,0	925,6	856,3	785,9	716,4	587,8	480,2	394,7	327,8	275,5	180	180	5
180	180	6	1450	1450	1450	1392	1323	1251	1176	1096	1013	929,0	845,9	692,8	565,3	464,2	385,4	323,8	180	180	6
180	180	7,1	1682	1682	1682	1613	1532	1447	1358	1265	1167	1068	971,4	793,4	646,1	529,9	439,5	369,0	180	180	7,1
180	180	8	1876	1876	1876	1797	1706	1611	1511	1406	1296	1185	1076	877,7	713,9	585,0	484,9	406,9	180	180	8
180	180	8,8	2045	2045	2045	1957	1857	1753	1643	1527	1407	1285	1166	949,5	771,4	631,7	523,3	439,1	180	180	8,8
180	180	10	2292	2292	2292	2191	2077	1959	1835	1703	1567	1429	1295	1052	853,3	697,9	577,8	484,5	180	180	10
180	180	12,5	2735	2735	2735	2603	2463	2318	2163	2001	1833	1665	1502	1212	978,0	797,4	658,7	551,5	180	180	12,5
200	200	5	1238	1238	1238	1215	1166	1115	1063	1008	950,3	890,7	829,7	709,0	598,2	503,1	424,7	361,0	200	200	5
200	200	6	1620	1620	1620	1579	1511	1441	1367	1290	1209	1126	1042	877,8	732,1	610,5	512,2	433,5	200	200	6
200	200	7,1	1883	1883	1883	1834	1753	1671	1584	1494	1399	1301	1202	1010	841,0	700,3	586,9	496,4	200	200	7,1
200	200	8	2103	2103	2103	2046	1956	1863	1766	1664	1557	1447	1335	1121	931,8	775,2	649,3	548,9	200	200	8
200	200	8,8	2295	2295	2295	2231	2132	2030	1923	1811	1694	1573	1451	1216	1010	839,3	702,6	593,7	200	200	8,8
200	200	10	2576	2576	2576	2502	2390	2274	2153	2026	1892	1755	1617	1353	1121	930,9	778,6	657,6	200	200	10
200	200	12,5	3090	3090	3090	2992	2853	2710	2561	2403	2238	2069	1900	1579	1302	1078	899,2	758,2	200	200	12,5

h mm	b mm	t mm	$N_{b.Rd}$ (kN) L_{cr} (m) 0	0,5	1	1,5	2	2,5	3	3,5	4	4,5	5	6	7	8	9	10	h mm	b mm	t mm
220	220	6	1790	1790	1790	1767	1698	1629	1557	1482	1404	1323	1239	1071	912,2	773,1	656,3	560,2	220	220	6
220	220	7,1	2085	2085	2085	2056	1975	1893	1809	1721	1629	1533	1434	1237	1052	890,4	755,0	643,9	220	220	7,1
220	220	8	2330	2330	2330	2296	2206	2114	2019	1920	1816	1708	1597	1375	1168	987,9	837,1	713,6	220	220	8
220	220	8,8	2545	2545	2545	2506	2407	2306	2201	2092	1978	1860	1738	1495	1269	1072	907,7	773,4	220	220	8,8
220	220	10	2860	2860	2860	2814	2702	2587	2469	2345	2216	2081	1943	1669	1414	1193	1009	859,3	220	220	10
220	220	12,5	3445	3445	3445	3381	3242	3101	2955	2802	2641	2475	2305	1968	1660	1395	1177	1001	220	220	12,5
250	250	6	1811	1811	1811	1811	1766	1709	1651	1591	1530	1466	1400	1262	1121	985,2	860,6	750,8	250	250	6
250	250	7,1	2387	2387	2387	2387	2308	2227	2144	2059	1970	1878	1782	1585	1388	1204	1040	899,7	250	250	7,1
250	250	8	2671	2671	2671	2671	2580	2489	2396	2300	2200	2097	1989	1767	1546	1339	1157	999,8	250	250	8
250	250	8,8	2920	2920	2920	2918	2818	2718	2616	2511	2401	2287	2169	1925	1683	1457	1257	1086	250	250	8,8
250	250	10	3286	3286	3286	3282	3169	3056	2940	2821	2696	2567	2433	2156	1882	1627	1403	1211	250	250	10
250	250	12,5	3978	3978	3978	3965	3826	3686	3543	3395	3241	3080	2914	2572	2237	1927	1657	1428	250	250	12,5
260	260	6	1836	1836	1836	1836	1803	1748	1693	1636	1578	1518	1455	1324	1189	1056	930,5	817,7	260	260	6
260	260	7,1	2488	2488	2488	2488	2418	2338	2255	2171	2083	1992	1898	1701	1503	1314	1142	993,1	260	260	7,1
260	260	8	2785	2785	2785	2785	2705	2614	2521	2426	2328	2225	2119	1898	1675	1463	1271	1104	260	260	8
260	260	8,8	3045	3045	3045	3045	2956	2856	2754	2650	2541	2429	2312	2069	1825	1592	1383	1201	260	260	8,8
260	260	10	3428	3428	3428	3428	3325	3212	3097	2978	2855	2727	2595	2320	2043	1781	1545	1340	260	260	10
260	260	12,5	4155	4155	4155	4155	4021	3881	3739	3592	3439	3281	3116	2776	2436	2116	1831	1585	260	260	12,5
300	300	6	1921	1921	1921	1921	1921	1881	1834	1786	1738	1689	1638	1533	1421	1306	1190	1077	300	300	6
300	300	7,1	2533	2533	2533	2533	2525	2459	2392	2325	2257	2186	2114	1962	1802	1639	1477	1324	300	300	7,1
300	300	8	3066	3066	3066	3066	3043	2959	2875	2789	2702	2612	2520	2325	2122	1915	1714	1526	300	300	8
300	300	8,8	3544	3544	3544	3544	3505	3405	3305	3203	3098	2990	2878	2644	2400	2154	1918	1700	300	300	8,8
300	300	10	3996	3996	3996	3996	3949	3836	3722	3607	3488	3365	3238	2973	2696	2417	2150	1904	300	300	10
300	300	12,5	4865	4865	4865	4865	4800	4660	4520	4376	4229	4077	3919	3590	3247	2904	2576	2277	300	300	12,5

Table 11.2.2 Buckling resistance values for rectangular hollow sections of steel grade S355J2H (f_y = 355 N/mm^2)



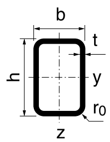
h = depth
b = width
t = wall thickness
L_{cr} = buckling length
$N_{b.Rd}$ = buckling resistance

The calculated resistance values are design values (see Chapter 2) based on recommended partial safety factor values **γ_{M0} = 1,0** and **γ_{M1} = 1,0 (for Class 4 circular hollow sections γ_{M1} = 1,1)** as given in Eurocode 3 (EN 1993). Partial safety factor values may differ in each country. National values must be checked from the National Annex of the relevant country.

h mm	b mm	t mm	axis	$N_{b.Rd}$ (kN) L_{cr} (m) 0	0,5	1	1,5	2	2,5	3	3,5	4	4,5	5	6	7	8	9	10	h mm	b mm	t mm
40	20	2	y-y	75,86	64,99	43,17	25,77	16,22	11,00	7,91	5,96									40	20	2
			z-z	75,86	49,02	20,50	10,22	6,04	3,98	2,82	2,10											
40	20	2,5	y-y	91,91	78,20	51,14	30,20	18,93	12,81	9,21	6,93									40	20	2,5
			z-z	91,91	58,02	23,72	11,77	6,94	4,57	3,23	2,40											
40	20	3	y-y	106,8	90,18	57,99	33,87	21,14	14,28	10,26	7,71									40	20	3
			z-z	106,8	65,75	26,27	12,97	7,64	5,02	3,55	2,64											
40	30	2	y-y	90,06	78,67	54,62	33,76	21,55	14,70	10,61	8,00	6,25								40	30	2
			z-z	90,06	73,03	42,99	23,90	14,66	9,83	7,03	5,27	4,09										
40	30	2,5	y-y	109,7	95,30	65,42	40,05	25,46	17,34	12,51	9,43	7,35								40	30	2,5
			z-z	109,7	88,21	51,07	28,18	17,24	11,54	8,24	6,18	4,80										
40	30	3	y-y	128,1	110,70	75,09	45,53	28,83	19,60	14,12	10,64	8,30								40	30	3
			z-z	128,1	102,20	58,14	31,83	19,42	12,99	9,27	6,94	5,39										
50	30	2	y-y	104,3	95,57	73,86	51,08	34,45	24,10	17,63	13,40	10,52	8,46							50	30	2
			z-z	104,3	85,46	51,47	28,94	17,83	11,97	8,56	6,42	4,99	3,99									
50	30	2,5	y-y	127,4	116,4	89,35	61,26	41,11	28,69	20,96	15,93	12,49	10,05							50	30	2,5
			z-z	127,4	103,7	61,49	34,31	21,08	14,13	10,11	7,58	5,89	4,71									
50	30	3	y-y	149,4	136,0	103,7	70,44	47,04	32,75	23,90	18,15	14,22	11,44							50	30	3
			z-z	149,4	120,6	70,42	39,00	23,89	16,00	11,44	8,57	6,66	5,32									
50	30	4	y-y	189,9	171,6	128,7	85,82	56,74	39,33	28,63	21,70	17,00	13,66							50	30	4
			z-z	189,9	150,8	85,13	46,44	28,31	18,92	13,50	10,11	7,85	6,27									
60	40	2	y-y	132,7	126,3	105,0	80,85	58,88	42,86	32,03	24,66	19,50	15,79	13,03						60	40	2
			z-z	132,7	118,8	87,27	56,89	37,19	25,65	18,62	14,09	11,02	8,85	7,26								
60	40	2,5	y-y	162,9	154,8	128,3	98,21	71,17	51,67	38,55	29,65	23,44	18,97	15,65						60	40	2,5
			z-z	162,9	145,4	106,0	68,56	44,65	30,73	22,29	16,86	13,19	10,59	8,69								
60	40	3	y-y	192,0	182,0	150,3	114,4	82,52	59,75	44,51	34,21	27,03	21,86	18,03						60	40	3
			z-z	192,0	170,7	123,5	79,24	51,41	35,33	25,60	19,36	15,13	12,15	9,96								
60	40	4	y-y	246,7	232,9	190,8	143,5	102,5	73,77	54,79	42,04	33,17	26,81	22,10						60	40	4
			z-z	246,7	217,7	154,8	97,75	62,96	43,13	31,20	23,56	18,41	14,77	12,11								
60	40	5	y-y	296,6	278,8	226,5	168,1	118,8	85,05	62,99	48,24	38,03	30,71	25,30						60	40	5
			z-z	296,6	259,7	181,4	112,6	72,03	49,18	35,52	26,80	20,92	16,77	13,75								

h mm	b mm	t mm	axis	$N_{b.Rd}$ (kN) L_{cr} (m) 0	0,5	1	1,5	2	2,5	3	3,5	4	4,5	5	6	7	8	9	10	h mm	b mm	t mm
70	50	2	y-y	161,1	157,1	136,1	112,2	87,55	66,70	51,15	39,99	31,94	26,02	21,58	15,49					70	50	2
			z-z	161,1	151,1	122,3	90,23	63,50	45,36	33,56	25,68	20,24	16,34	13,46	9,58							
70	50	2,5	y-y	198,4	193,3	167,1	137,3	106,7	81,06	62,06	48,46	38,68	31,50	26,11	18,74					70	50	2,5
			z-z	198,4	185,7	149,8	109,9	76,98	54,88	40,55	31,01	24,42	19,71	16,23	11,55							
70	50	3	y-y	234,6	228,2	196,9	161,2	124,8	94,53	72,24	56,35	44,95	36,59	30,32	21,75					70	50	3
			z-z	234,6	219,1	176,0	128,3	89,55	63,70	47,01	35,93	28,29	22,82	18,79	13,36							
70	50	4	y-y	303,5	294,4	252,8	205,5	157,8	118,8	90,47	70,43	56,11	45,63	37,79	27,09					70	50	4
			z-z	303,5	282,3	224,8	161,9	112,0	79,33	58,41	44,58	35,07	28,27	23,27	16,54							
70	50	5	y-y	367,6	355,6	304,0	245,0	186,5	139,6	106,0	82,32	65,49	53,22	44,04	31,54					70	50	5
			z-z	367,6	340,4	268,6	191,0	131,0	92,40	67,89	51,75	40,67	32,77	26,96	19,15							
80	40	2	y-y	151,2	149,6	132,3	113,0	92,19	72,83	57,17	45,34	36,54	29,96	24,95	18,01	13,59				80	40	2
			z-z	151,2	137,3	104,2	70,4	46,85	32,57	23,75	18,03	14,13	11,36	9,33	6,61	4,93						
80	40	2,5	y-y	198,4	195,3	171,6	144,9	116,4	90,76	70,62	55,70	44,74	36,59	30,42	21,92	16,51				80	40	2,5
			z-z	198,4	178,5	132,4	87,2	57,29	39,59	28,78	21,80	17,06	13,71	11,25	7,96	5,93						
80	40	3	y-y	234,6	230,7	202,2	170,3	136,3	105,9	82,25	64,79	51,99	42,50	35,32	25,44	19,16				80	40	3
			z-z	234,6	210,3	154,9	101,3	66,29	45,73	33,21	25,14	19,67	15,80	12,96	9,18	6,83						
80	40	4	y-y	303,5	297,6	260,0	217,5	172,7	133,3	103,1	81,00	64,90	52,99	44,00	31,66	23,83				80	40	4
			z-z	303,5	270,2	196,1	126,2	82,01	56,40	40,89	30,92	24,17	19,40	15,92	11,26	8,38						
80	40	5	y-y	367,6	359,6	312,8	259,8	204,6	156,9	120,8	94,67	75,73	61,76	51,25	36,84	27,71				80	40	5
			z-z	367,6	325,1	232,1	147,1	94,88	65,04	47,07	35,56	27,78	22,29	18,27	12,92	9,62						
80	60	2,5	y-y	233,9	231,9	205,7	176,7	145,2	115,4	90,98	72,35	58,41	47,94	39,95	28,88	21,80	17,03			80	60	2,5
			z-z	233,9	225,6	191,8	153,3	115,6	86,02	65,05	50,44	40,07	32,54	26,91	19,26	14,45	11,23					
80	60	3	y-y	277,2	274,6	243,3	208,5	170,7	135,4	106,5	84,60	68,26	55,99	46,65	33,70	25,44	19,86			80	60	3
			z-z	277,2	266,9	226,5	180,3	135,5	100,5	75,92	58,81	46,70	37,90	31,34	22,42	16,81	13,07					
80	60	4	y-y	360,3	356,2	314,7	268,4	218,5	172,2	135,0	107,0	86,19	70,62	58,80	42,44	32,01	24,98			80	60	4
			z-z	360,3	345,9	292,1	230,7	172,0	127,0	95,61	73,94	58,66	47,57	39,31	28,10	21,07	16,37					
80	60	5	y-y	438,6	432,8	381,2	323,6	261,7	205,1	160,2	126,6	101,9	83,37	69,36	50,02	37,71	29,42			80	60	5
			z-z	438,6	419,9	352,8	276,3	204,3	150,1	112,7	87,02	68,96	55,88	46,16	32,98	24,71	19,19					
90	50	2,5	y-y	233,9	233,9	209,8	183,5	154,7	126,0	101,2	81,45	66,28	54,68	45,74	33,22	25,15	19,68	15,80		90	50	2,5
			z-z	233,9	220,1	179,2	133,4	94,54	67,79	50,24	38,49	30,35	24,52	20,20	14,38	10,75	8,34	6,66				
90	50	3	y-y	277,2	276,9	248,1	216,7	182,1	148,0	118,6	95,30	77,47	63,88	53,41	38,77	29,34	22,95	18,43		90	50	3
			z-z	277,2	260,3	211,2	156,4	110,4	78,95	58,44	44,75	35,27	28,48	23,46	16,70	12,48	9,68	7,72				
90	50	4	y-y	360,3	359,2	321,1	279,3	233,4	188,6	150,5	120,6	97,89	80,61	67,34	48,83	36,93	28,87	23,18		90	50	4
			z-z	360,3	337,0	271,4	198,7	139,0	99,04	73,14	55,93	44,04	35,54	29,26	20,82	15,55	12,06	9,62				
90	50	5	y-y	438,6	436,5	389,2	337,1	280,0	225,0	178,8	142,9	115,7	95,19	79,45	57,55	43,49	33,99	27,28		90	50	5
			z-z	438,6	408,7	326,4	236,2	163,9	116,3	85,67	65,42	51,48	41,51	34,17	24,29	18,14	14,06	11,22				

Table 11.2.2 Buckling resistance values for rectangular hollow sections of steel grade S355J2H (f_y = 355 N/mm^2) *continued*



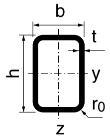
h = depth
b = width
t = wall thickness
L_{cr} = buckling length
$N_{b.Rd}$ = buckling resistance

The calculated resistance values are design values (see Chapter 2) based on recommended partial safety factor values **γ_{M0} = 1,0** and **γ_{M1} = 1,0 (for Class 4 circular hollow sections γ_{M1} = 1,1)** as given in Eurocode 3 (EN 1993). Partial safety factor values may differ in each country. National values must be checked from the National Annex of the relevant country.

h mm	b mm	t mm	axis	$N_{b.Rd}$ (kN) L_{cr} (m) 0	0,5	1	1,5	2	2,5	3	3,5	4	4,5	5	6	7	8	9	10	h mm	b mm	t mm
100	40	2,5	y-y	218,4	218,4	200,1	178,4	154,4	129,5	106,5	87,20	71,76	59,66	50,17	36,69	27,89	21,88	17,61	14,47	100	40	2,5
			z-z	218,4	198,9	151,6	103,1	68,82	47,92	34,97	26,55	20,81	16,74	13,75	9,75	7,27	5,63	4,48	3,66			
100	40	3	y-y	277,2	277,2	251,6	222,5	190,3	157,6	128,2	104,1	85,21	70,58	59,21	43,16	32,74	25,65	20,63	16,94	100	40	3
			z-z	277,2	249,8	186,1	123,1	81,09	56,10	40,80	30,92	24,20	19,45	15,96	11,30	8,42	6,52	5,19	4,23			
100	40	4	y-y	360,3	360,3	325,6	286,9	244,2	201,1	162,8	131,8	107,6	89,01	74,59	54,30	41,16	32,23	25,91	21,27	100	40	4
			z-z	360,3	322,6	237,0	154,5	101,0	69,63	50,55	38,26	29,93	24,04	19,72	13,96	10,39	8,04	6,40	5,22			
100	40	5	y-y	438,6	438,6	394,8	346,4	293,2	240,0	193,4	156,0	127,2	105,0	87,91	63,91	48,41	37,89	30,44	24,99	100	40	5
			z-z	438,6	390,2	282,3	181,3	117,6	80,84	58,59	44,30	34,63	27,80	22,80	16,13	12,01	9,28	7,39	6,02			
100	50	2,5	y-y	236,2	236,2	217,7	195,1	170,1	144,0	119,4	98,35	81,28	67,76	57,10	41,86	31,87	25,03	20,16	16,57	100	50	2,5
			z-z	236,2	223,8	184,7	140,4	101,2	73,20	54,52	41,89	33,09	26,76	22,07	15,73	11,77	9,14	7,30	5,96			
100	50	3	y-y	298,5	298,5	272,9	242,9	209,7	175,5	144,0	117,7	96,72	80,35	67,53	49,35	37,50	29,41	23,67	19,44	100	50	3
			z-z	298,5	280,9	228,7	170,3	120,7	86,55	64,15	49,15	38,76	31,30	25,79	18,36	13,73	10,65	8,50	6,94			
100	50	4	y-y	388,7	388,7	354,1	314,2	270,1	224,9	183,8	149,7	122,8	101,9	85,54	62,43	47,40	37,16	29,89	24,55	100	50	4
			z-z	388,7	364,3	294,5	217,0	152,5	108,9	80,49	61,59	48,53	39,17	32,26	22,95	17,15	13,30	10,61	8,67			
100	50	5	y-y	474,1	474,1	430,4	380,6	325,7	269,8	219,5	178,3	145,9	120,9	101,4	73,92	56,08	43,94	35,33	29,01	100	50	5
			z-z	474,1	442,7	355,2	258,6	180,2	128,1	94,54	72,24	56,87	45,88	37,77	26,86	20,06	15,55	12,41	10,13			
100	50	6	y-y	555,0	555,0	501,8	442,2	376,5	310,2	251,3	203,4	166,2	137,5	115,2	83,86	63,58	49,79	40,02	32,86	100	50	6
			z-z	555,0	516,0	410,6	295,4	204,2	144,6	106,4	81,22	63,88	51,51	42,38	30,12	22,49	17,43	13,90	11,35			
100	60	2,5	y-y	253,9	253,9	235,2	211,6	185,7	158,3	132,1	109,4	90,69	75,79	63,98	47,01	35,84	28,17	22,70	18,67	100	60	2,5
			z-z	253,9	246,8	212,5	173,4	133,8	101,1	77,17	60,15	47,95	39,02	32,32	23,18	17,41	13,55	10,84	8,87			
100	60	3	y-y	319,8	319,8	294,0	263,0	228,7	192,9	159,5	131,0	108,1	90,01	75,79	55,51	42,24	33,16	26,70	21,94	100	60	3
			z-z	319,8	309,1	263,9	212,3	161,3	120,6	91,43	71,00	56,47	45,88	37,96	27,18	20,40	15,86	12,69	10,38			
100	60	4	y-y	417,1	417,1	382,3	341,0	295,4	248,1	204,3	167,4	137,8	114,6	96,39	70,51	53,61	42,07	33,86	27,82	100	60	4
			z-z	417,1	402,1	341,8	273,0	205,9	153,1	115,8	89,75	71,30	57,89	47,87	34,26	25,70	19,98	15,97	13,06			
100	60	5	y-y	509,6	509,6	465,7	414,2	357,4	298,8	245,1	200,1	164,5	136,6	114,8	83,86	63,71	49,97	40,20	33,03	100	60	5
			z-z	509,6	490,0	414,7	328,7	245,9	182,0	137,2	106,2	84,30	68,39	56,53	40,43	30,31	23,56	18,83	15,39			
100	60	6	y-y	597,6	597,6	544,1	482,6	414,6	345,0	281,8	229,4	188,2	156,0	131,0	95,60	72,58	56,89	45,76	37,59	100	60	6
			z-z	597,6	572,9	482,6	379,4	281,7	207,4	156,0	120,5	95,56	77,47	64,01	45,74	34,28	26,63	21,28	17,40			

h mm	b mm	t mm	axis	$N_{b.Rd}$ (kN) L_{cr} (m) 0	0,5	1	1,5	2	2,5	3	3,5	4	4,5	5	6	7	8	9	10	h mm	b mm	t mm
100	80	2,5	y-y	289,4	289,4	270,0	244,4	216,2	186,3	157,1	131,0	109,3	91,70	77,62	57,24	43,73	34,42	27,76	22,85	100	80	2,5
			z-z	289,4	289,4	260,7	229,0	194,0	159,0	128,3	103,5	84,42	69,74	58,39	42,46	32,17	25,18	20,23	16,61			
100	80	3	y-y	362,4	362,4	336,0	302,6	265,9	227,0	189,8	157,2	130,5	109,1	92,16	67,75	51,67	40,62	32,73	26,93	100	80	3
			z-z	362,4	361,8	323,9	282,4	236,8	192,1	153,7	123,4	100,2	82,61	69,05	50,10	37,91	29,65	23,81	19,53			
100	80	4	y-y	473,9	473,9	438,2	393,8	344,9	293,5	244,5	202,0	167,3	139,7	117,9	86,57	65,97	51,83	41,76	34,35	100	80	4
			z-z	473,9	472,3	422,0	366,7	306,2	247,2	197,1	157,8	128,0	105,4	88,05	63,83	48,27	37,74	30,29	24,85			
100	80	5	y-y	580,6	580,6	535,6	480,2	419,3	355,3	294,9	243,0	200,9	167,6	141,3	103,6	78,89	61,96	49,90	41,03	100	80	5
			z-z	580,6	577,8	515,2	446,2	370,7	297,8	236,6	189,1	153,2	126,0	105,2	76,16	57,56	44,99	36,10	29,61			
100	80	6	y-y	682,8	682,8	628,0	561,8	488,8	412,6	341,2	280,4	231,4	192,8	162,3	118,9	90,48	71,03	57,19	47,01	100	80	6
			z-z	682,8	678,4	603,5	520,7	430,4	344,2	272,5	217,3	175,7	144,4	120,4	87,16	65,84	51,43	41,27	33,84			
120	40	2,5	y-y	228,5	228,5	217,4	199,8	180,8	160,3	139,2	119,1	101,3	86,25	73,76	55,11	42,43	33,56	27,16	22,41	120	40	2,5
			z-z	228,5	210,8	165,1	116,3	79,26	55,73	40,88	31,13	24,45	19,70	16,20	11,50	8,58	6,65	5,30	4,32			
120	40	3	y-y	297,5	297,5	280,7	256,2	229,6	201,0	172,3	145,7	122,8	103,8	88,30	65,55	50,27	39,66	32,05	26,42	120	40	3
			z-z	297,5	271,2	207,1	141,1	94,34	65,73	47,98	36,44	28,56	22,98	18,87	13,38	9,98	7,72	6,15	5,02			
120	40	4	y-y	417,1	417,1	389,8	353,4	313,4	270,8	229,0	191,5	160,0	134,4	113,9	84,06	64,26	50,60	40,82	33,61	120	40	4
			z-z	417,1	374,9	277,7	182,6	119,9	82,84	60,20	45,59	35,68	28,66	23,52	16,66	12,41	9,60	7,64	6,23			
120	40	5	y-y	509,6	509,6	474,9	429,5	379,6	326,5	274,8	229,0	190,8	160,0	135,4	99,79	76,21	59,97	48,36	39,81	120	40	5
			z-z	509,6	455,2	332,3	215,3	140,3	96,62	70,10	53,03	41,47	33,30	27,32	19,33	14,39	11,13	8,86	7,22			
120	40	6	y-y	597,6	597,6	555,0	500,6	440,7	377,3	316,2	262,5	218,1	182,6	154,3	113,5	86,61	68,11	54,90	45,17	120	40	6
			z-z	597,6	530,0	381,1	243,1	157,4	108,0	78,22	59,12	46,20	37,08	30,40	21,50	16,01	12,37	9,85	8,03			
120	50	2,5	y-y	246,2	246,2	235,3	216,9	197,1	175,7	153,5	132,2	113,0	96,50	82,75	62,04	47,86	37,90	30,70	25,35	120	50	2,5
			z-z	246,2	235,6	197,9	154,8	114,4	83,98	63,05	48,67	38,57	31,25	25,81	18,44	13,82	10,73	8,58	7,01			
120	50	3	y-y	318,8	318,8	302,3	277,0	249,6	220,1	190,0	161,8	137,0	116,3	99,22	73,91	56,81	44,88	36,30	29,94	120	50	3
			z-z	318,8	302,6	250,4	191,2	138,2	100,2	74,72	57,45	45,40	36,73	30,29	21,60	16,17	12,55	10,02	8,19			
120	50	4	y-y	445,5	445,5	418,9	381,6	340,8	297,0	253,4	213,4	179,3	151,2	128,5	95,19	72,92	57,50	46,43	38,26	120	50	4
			z-z	445,5	418,9	340,7	253,3	179,3	128,5	95,16	72,90	57,48	46,42	38,24	27,23	20,35	15,79	12,60	10,29			
120	50	5	y-y	545,1	545,1	511,3	464,8	413,9	359,4	305,4	256,4	214,9	180,9	153,5	113,6	86,90	68,48	55,28	45,53	120	50	5
			z-z	545,1	510,7	412,4	303,2	212,8	151,9	112,3	85,87	67,65	54,60	44,96	31,99	23,91	18,54	14,79	12,07			
120	50	6	y-y	640,2	640,2	598,8	543,1	482,1	416,9	352,8	295,3	246,8	207,5	175,8	129,8	99,28	78,18	63,08	51,94	120	50	6
			z-z	640,2	597,4	478,7	348,0	242,2	172,1	126,9	96,98	76,33	61,57	50,68	36,04	26,92	20,87	16,65	13,59			
120	60	2,5	y-y	264,0	264,0	253,1	233,8	213,2	190,9	167,6	145,0	124,5	106,7	91,67	68,93	53,27	42,23	34,24	28,29	120	60	2,5
			z-z	264,0	258,6	225,4	187,9	148,6	114,3	88,22	69,23	55,42	45,23	37,54	27,00	20,31	15,83	12,67	10,38			
120	60	3	y-y	340,1	340,1	323,8	297,6	269,3	238,8	207,4	177,6	151,1	128,6	110,0	82,22	63,30	50,07	40,53	33,45	120	60	3
			z-z	340,1	331,1	285,8	234,4	181,8	137,8	105,4	82,27	65,64	53,44	44,29	31,78	23,88	18,59	14,88	12,17			
120	60	4	y-y	473,9	473,9	447,8	409,3	367,5	322,6	277,1	234,9	198,3	167,8	142,9	106,2	81,53	64,36	52,02	42,89	120	60	4
			z-z	473,9	458,1	391,3	315,0	239,5	179,1	135,8	105,5	83,92	68,18	56,42	40,40	30,32	23,58	18,86	15,42			
120	60	5	y-y	580,6	580,6	547,4	499,6	447,4	391,4	335,1	283,2	238,5	201,5	171,4	127,2	97,53	76,94	62,16	51,23	120	60	5
			z-z	580,6	560,0	476,3	380,7	287,3	213,8	161,7	125,4	99,61	80,87	66,89	47,87	35,91	27,92	22,32	18,25			
120	60	6	y-y	682,8	682,8	642,1	584,9	522,4	455,4	388,5	327,3	275,0	232,0	197,1	146,0	111,9	88,20	71,23	58,68	120	60	6
			z-z	682,8	656,7	556,1	441,2	330,4	244,7	184,5	142,8	113,4	91,99	76,04	54,38	40,78	31,69	25,33	20,71			

Table 11.2.2 Buckling resistance values for rectangular hollow sections of steel grade S355J2H (f_y = 355 N/mm^2) *continued*



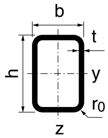
h = depth
b = width
t = wall thickness
L_{cr} = buckling length
$N_{b.Rd}$ = buckling resistance

The calculated resistance values are design values (see Chapter 2) based on recommended partial safety factor values **γ_{M0} = 1,0** and **γ_{M1} = 1,0 (for Class 4 circular hollow sections γ_{M1} = 1,1)** as given in Eurocode 3 (EN 1993). Partial safety factor values may differ in each country. National values must be checked from the National Annex of the relevant country.

h mm	b mm	t mm	axis	$N_{b.Rd}$ (kN) L_{cr} (m) 0	0,5	1	1,5	2	2,5	3	3,5	4	4,5	5	6	7	8	9	10	h mm	b mm	t mm
120	80	2,5	y-y	299,5	299,5	288,6	267,5	245,1	220,9	195,5	170,4	147,1	126,7	109,3	82,61	64,02	50,85	41,28	34,14	120	80	2,5
			z-z	299,5	299,5	273,3	242,9	209,3	174,8	143,1	116,8	95,91	79,62	66,89	48,85	37,11	29,10	23,41	19,23			
120	80	3	y-y	382,7	382,7	366,5	338,4	308,1	275,4	241,5	208,6	178,7	153,0	131,4	98,70	76,22	60,41	48,96	40,44	120	80	3
			z-z	382,7	382,7	346,1	305,1	259,8	214,1	173,4	140,4	114,7	94,90	79,53	57,90	43,90	34,38	27,63	22,69			
120	80	4	y-y	530,7	530,7	505,1	464,2	420,0	372,3	323,4	276,8	235,5	200,5	171,5	128,1	98,63	78,01	63,14	52,11	120	80	4
			z-z	530,7	530,2	475,2	415,1	349,1	283,9	227,6	183,0	148,8	122,70	102,6	74,47	56,37	44,09	35,41	29,05			
120	80	5	y-y	651,6	651,6	619,0	568,1	513,0	453,6	392,8	335,4	284,7	241,9	206,7	154,2	118,6	93,77	75,86	62,59	120	80	5
			z-z	651,6	650,1	581,6	506,4	424,0	343,2	274,2	220,0	178,6	147,10	122,9	89,17	67,45	52,75	42,35	34,74			
120	80	6	y-y	768,0	768,0	728,1	667,2	601,3	530,1	457,7	389,7	330,1	280,1	239,0	178,0	136,8	108,1	87,43	72,11	120	80	6
			z-z	768,0	765,0	683,0	592,8	494,0	398,1	317,0	253,7	205,7	169,2	141,3	102,4	77,43	60,53	48,58	39,85			
120	80	7,1	y-y	875,0	875,0	826,4	755,2	677,8	594,4	510,4	432,3	364,8	308,6	262,8	195,2	149,8	118,3	95,57	78,78	120	80	7,1
			z-z	875,0	869,5	773,8	668,0	552,6	442,2	350,3	279,4	226,0	185,8	155,0	112,1	84,72	66,19	53,11	43,54			
120	80	8	y-y	967,1	967,1	911,4	831,5	744,4	650,8	556,9	470,4	396,0	334,5	284,5	211,0	161,8	127,6	103,1	84,98	120	80	8
			z-z	967,1	959,6	852,0	732,9	603,3	480,6	379,4	302,0	244,0	200,3	167,0	120,7	91,17	71,20	57,12	46,82			
120	80	8,8	y-y	1045	1045	983,0	895,3	799,6	697,0	594,6	500,9	420,8	354,9	301,5	223,4	171,1	134,9	109,0	89,78	120	80	8,8
			z-z	1045	1036	917,6	786,6	644,6	511,2	402,4	319,7	258,0	211,6	176,3	127,4	96,15	75,07	60,20	49,34			
120	80	10	y-y	1156	1156	1084	984,4	875,9	759,8	644,9	541,0	453,1	381,3	323,5	239,2	183,0	144,2	116,4	95,84	120	80	10
			z-z	1156	1143	1009	860,5	700,0	551,6	432,2	342,4	275,8	226,0	188,1	135,8	102,4	79,91	64,06	52,49			
120	100	2,5	y-y	319,5	319,5	310,0	288,7	266,2	242,0	216,3	190,6	166,1	144,1	125,1	95,26	74,16	59,07	48,04	39,79	120	100	2,5
			z-z	319,5	319,5	303,7	278,8	251,9	222,9	193,2	165,1	140,2	119,2	101,9	76,06	58,52	46,27	37,44	30,89			
120	100	3	y-y	425,3	425,3	409,1	378,8	346,5	311,5	274,9	239,0	205,9	177,0	152,5	115,0	89,05	70,69	57,35	47,41	120	100	3
			z-z	425,3	425,3	400,2	364,6	325,8	284,2	242,6	204,5	171,9	145,0	123,2	91,32	69,97	55,17	44,56	36,71			
120	100	4	y-y	587,5	587,5	562,0	518,4	471,6	421,0	368,5	317,8	272,0	232,6	199,6	149,8	115,6	91,58	74,21	61,29	120	100	4
			z-z	587,5	587,5	549,1	497,8	441,5	381,5	322,5	269,7	225,3	189,3	160,4	118,4	90,51	71,26	57,50	47,34			
120	100	5	y-y	722,6	722,6	690,2	635,9	577,5	514,3	449,1	386,3	330,0	281,7	241,5	180,9	139,5	110,5	89,49	73,89	120	100	5
			z-z	722,6	722,6	674,0	610,0	539,7	464,9	391,8	326,8	272,6	228,7	193,6	142,7	109,0	85,81	69,22	56,97			
120	100	6	y-y	853,2	853,2	813,4	748,5	678,5	602,9	525,0	450,5	384,0	327,4	280,3	209,7	161,6	127,9	103,5	85,47	120	100	6
			z-z	853,2	853,2	794,0	717,3	633,0	543,5	456,7	379,9	316,3	265,0	224,1	165,1	126,0	99,14	79,94	65,78			

h mm	b mm	t mm	axis	$N_{b.Rd}$ (kN) L_{cr} (m) 0	0,5	1	1,5	2	2,5	3	3,5	4	4,5	5	6	7	8	9	10	h mm	b mm	t mm
140	60	2,5	y-y	271,0	271,0	266,1	249,9	232,9	214,8	195,5	175,5	155,9	137,4	120,8	93,57	73,62	59,03	48,23	40,06	140	60	2,5
			z-z	271,0	267,2	235,3	199,5	161,2	126,3	98,54	77,86	62,61	51,24	42,63	30,74	23,17	18,07	14,48	11,87			
140	60	3	y-y	352,5	352,5	344,0	321,7	298,2	273,0	246,3	218,9	192,5	168,3	146,9	112,7	88,18	70,45	57,41	47,61	140	60	3
			z-z	352,5	345,4	301,3	251,4	199,1	153,4	118,4	92,97	74,44	60,76	50,44	36,28	27,30	21,27	17,04	13,95			
140	60	4	y-y	530,7	530,7	512,2	475,5	436,3	394,1	349,7	305,6	264,6	228,3	197,2	149,3	115,9	92,09	74,79	61,87	140	60	4
			z-z	530,7	514,1	440,6	356,8	273,0	205,0	155,9	121,2	96,51	78,46	64,95	46,54	34,94	27,18	21,74	17,78			
140	60	5	y-y	651,6	651,6	627,9	582,1	533,3	480,6	425,3	370,7	320,2	275,7	237,9	179,7	139,3	110,6	89,81	74,27	140	60	5
			z-z	651,6	629,8	537,6	432,4	328,4	245,4	186,0	144,5	114,9	93,34	77,23	55,30	41,50	32,27	25,81	21,11			
140	60	6	y-y	768,0	768,0	738,6	683,9	625,4	562,2	496,2	431,2	371,5	319,3	275,1	207,5	160,6	127,5	103,4	85,51	140	60	6
			z-z	768,0	740,3	629,3	502,6	378,8	281,7	213,0	165,1	131,2	106,5	88,07	63,02	47,27	36,75	29,38	24,02			
140	70	2,5	y-y	288,7	288,7	284,1	267,0	249,3	230,4	210,3	189,4	168,7	149,1	131,3	102,1	80,47	64,61	52,82	43,91	140	70	2,5
			z-z	288,7	288,7	259,8	228,0	193,0	158,0	127,3	102,7	83,73	69,15	57,89	42,08	31,88	24,95	20,05	16,45			
140	70	3	y-y	373,8	373,8	365,7	342,5	318,2	292,1	264,4	236,0	208,3	182,6	159,8	123,1	96,45	77,16	62,94	52,23	140	70	3
			z-z	373,8	372,4	332,4	288,6	240,5	193,8	154,3	123,5	100,1	82,40	68,80	49,86	37,70	29,47	23,65	19,40			
140	70	4	y-y	559,1	559,1	541,4	503,7	463,6	420,5	375,0	329,4	286,4	248,0	214,9	163,3	127,0	101,1	82,15	68,00	140	70	4
			z-z	559,1	551,9	486,7	413,7	335,3	263,4	205,9	162,9	131,1	107,3	89,32	64,44	48,58	37,90	30,38	24,89			
140	70	5	y-y	687,1	687,1	664,4	617,4	567,5	513,6	456,9	400,4	347,4	300,3	259,8	197,1	153,0	121,7	98,91	81,85	140	70	5
			z-z	687,1	677,2	595,6	504,1	406,2	317,5	247,5	195,4	157,0	128,5	106,8	77,01	58,03	45,26	36,27	29,71			
140	70	6	y-y	810,6	810,6	782,4	726,3	666,5	602,0	534,2	466,8	404,1	348,7	301,3	228,1	177,0	140,7	114,2	94,49	140	70	6
			z-z	810,6	797,3	699,3	589,2	472,1	367,2	285,2	224,8	180,4	147,5	122,6	88,28	66,50	51,85	41,54	34,02			
140	80	3	y-y	395,1	395,1	387,3	363,2	338,0	311,0	282,3	252,8	223,8	196,8	172,6	133,3	104,7	83,85	68,45	56,83	140	80	3
			z-z	395,1	395,1	361,3	321,6	277,8	232,5	190,9	156,0	128,3	106,5	89,56	65,45	49,74	39,01	31,39	25,79			
140	80	4	y-y	587,5	587,5	570,5	531,7	490,7	446,5	399,8	352,7	307,9	267,5	232,3	177,2	138,0	110,0	89,49	74,12	140	80	4
			z-z	587,5	587,5	528,3	463,3	391,7	320,3	257,9	208,0	169,4	139,9	117,1	85,09	64,45	50,44	40,52	33,26			
140	80	5	y-y	722,6	722,6	700,7	652,4	601,3	546,2	488,0	429,5	374,2	324,5	281,5	214,2	166,7	132,8	108,0	89,40	140	80	5
			z-z	722,6	722,3	647,7	566,4	476,8	388,3	311,6	250,7	203,9	168,2	140,7	102,2	77,33	60,50	48,59	39,87			
140	80	6	y-y	853,2	853,2	826,0	768,3	707,1	641,1	571,5	501,8	436,2	377,6	327,1	248,5	193,2	153,8	125,0	103,4	140	80	6
			z-z	853,2	851,5	762,2	664,4	557,0	451,5	361,1	289,8	235,4	194,0	162,1	117,6	89,01	69,61	55,89	45,85			
150	50	2,5	y-y	256,0	256,0	253,3	238,9	224,1	208,3	191,6	174,0	156,4	139,4	123,6	97,09	77,02	62,09	50,90	42,39	150	50	2,5
			z-z	256,0	248,0	212,5	172,0	131,5	98,75	75,08	58,39	46,48	37,79	31,28	22,42	16,83	13,09	10,47	8,56			
150	50	3	y-y	336,1	336,1	330,4	310,4	289,6	267,4	243,7	219,2	195,0	172,1	151,5	117,6	92,60	74,30	60,73	50,46	150	50	3
			z-z	336,1	322,8	272,8	215,7	161,0	118,9	89,60	69,32	54,99	44,60	36,86	26,36	19,76	15,35	12,27	10,03			
150	50	4	y-y	509,0	509,0	514,7	479,3	441,9	401,6	358,9	316,1	275,5	239,0	207,3	157,9	122,9	97,88	79,60	65,91	150	50	4
			z-z	509,0	481,9	397,0	301,0	216,3	156,3	116,4	89,37	70,58	57,07	47,06	33,54	25,10	19,47	15,55	12,70			
150	50	5	y-y	651,6	651,6	630,9	586,9	540,2	489,8	436,6	383,4	333,3	288,6	250,0	189,9	147,6	117,5	95,51	79,06	150	50	5
			z-z	651,6	612,6	498,0	369,9	261,5	187,3	138,8	106,3	83,79	67,67	55,75	39,69	29,67	23,01	18,36	14,99			
150	50	6	y-y	768,0	768,0	742,2	689,5	633,5	573,0	509,4	446,0	386,8	334,1	289,0	219,1	170,1	135,3	109,9	90,93	150	50	6
			z-z	768,0	719,2	580,4	426,4	299,1	213,3	157,6	120,6	94,98	76,66	63,13	44,91	33,56	26,02	20,76	16,95			

Table 11.2.2 Buckling resistance values for rectangular hollow sections of steel grade S355J2H (f_y = 355 N/mm^2) *continued*



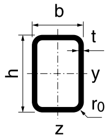
h = depth
b = width
t = wall thickness
L_{cr} = buckling length
$N_{b.Rd}$ = buckling resistance

The calculated resistance values are design values (see Chapter 2) based on recommended partial safety factor values **γ_{M0} = 1,0** and **γ_{M1} = 1,0 (for Class 4 circular hollow sections γ_{M1} = 1,1)** as given in Eurocode 3 (EN 1993). Partial safety factor values may differ in each country. National values must be checked from the National Annex of the relevant country.

h mm	b mm	t mm	axis	$N_{b.Rd}$ (kN) L_{cr} (m) 0	0,5	1	1,5	2	2,5	3	3,5	4	4,5	5	6	7	8	9	10	h mm	b mm	t mm
150	100	3	y-y	442,6	442,6	439,1	414,9	389,9	363,5	335,4	306,0	276,1	247,0	219,9	173,6	138,2	111,6	91,66	76,41	150	100	3
			z-z	442,6	442,6	421,5	387,6	350,9	311,3	270,7	231,9	197,4	168,1	143,9	107,5	82,82	65,52	53,04	43,77			
150	100	4	y-y	651,0	651,0	661,1	621,0	579,2	534,6	487,1	437,9	389,3	343,6	302,2	234,5	184,6	148,1	121,0	100,6	150	100	4
			z-z	651,0	651,0	613,2	559,3	500,4	437,2	373,8	315,6	265,6	224,2	190,7	141,4	108,4	85,50	69,07	56,91			
150	100	5	y-y	829,1	829,1	814,0	764,1	712,0	656,3	597,0	535,8	475,5	418,9	368,0	285,0	224,1	179,7	146,7	121,9	150	100	5
			z-z	829,1	829,1	777,3	706,3	628,6	545,4	463,1	388,6	325,5	274,0	232,5	171,9	131,5	103,6	83,65	68,89			
150	100	6	y-y	981,0	981,0	961,9	902,2	839,9	773,2	702,1	629,0	557,1	490,1	429,9	332,3	261,0	209,1	170,7	141,8	150	100	6
			z-z	981,0	981,0	917,8	832,6	739,3	639,7	541,6	453,3	379,1	318,6	270,1	199,5	152,5	120,1	96,94	79,82			
150	100	7,1	y-y	1127	1127	1103	1033	960,0	881,6	798,2	712,6	629,2	551,9	483,0	372,1	291,7	233,4	190,4	158,0	150	100	7,1
			z-z	1127	1127	1051	951,6	841,9	725,3	611,4	510,0	425,3	356,9	302,1	222,7	170,2	133,9	108,0	88,92			
150	100	8	y-y	1251	1251	1223	1144	1062	973,9	880,1	784,2	691,0	605,1	528,9	406,7	318,5	254,6	207,6	172,2	150	100	8
			z-z	1251	1251	1165	1052	928,7	797,6	670,4	557,9	464,5	389,2	329,2	242,5	185,1	145,6	117,4	96,64			
150	100	8,8	y-y	1358	1358	1325	1239	1149	1052	949,3	844,3	742,6	649,3	566,8	435,1	340,4	272,0	221,7	183,8	150	100	8,8
			z-z	1358	1358	1261	1138	1002	858,3	719,4	597,4	496,6	415,7	351,3	258,5	197,3	155,1	125,1	102,9			
150	100	10	y-y	1511	1511	1472	1375	1273	1163	1047	928,0	814,1	710,1	618,7	473,8	370,1	295,4	240,6	199,4	150	100	10
			z-z	1511	1511	1400	1260	1105	942,7	787,0	651,4	540,3	451,6	381,2	280,1	213,5	167,8	135,3	111,3			
160	70	4	y-y	576,3	576,3	568,9	535,9	501,6	465,2	426,5	386,1	345,6	306,9	271,4	212,2	167,9	135,1	110,6	92,06	160	70	4
			z-z	576,3	571,9	508,0	437,2	360,2	287,1	226,8	180,6	145,9	119,8	99,88	72,24	54,55	42,60	34,18	28,02			
160	70	5	y-y	758,1	758,1	744,6	699,1	651,6	600,9	546,9	491,1	436,0	384,4	337,8	261,7	205,9	165,1	134,9	112,1	160	70	5
			z-z	758,1	748,3	659,6	560,4	453,9	356,2	278,4	220,2	177,2	145,1	120,7	87,07	65,64	51,21	41,05	33,63			
160	70	6	y-y	895,8	895,8	878,6	824,2	767,3	706,6	641,8	575,2	509,6	448,4	393,5	304,2	239,0	191,5	156,4	129,8	160	70	6
			z-z	895,8	882,6	775,9	656,4	528,6	412,9	321,7	253,9	204,0	166,9	138,8	100,0	75,38	58,79	47,11	38,59			
160	70	8	y-y	1138	1138	1111	1039	964,3	883,6	798,0	710,4	625,5	547,3	478,1	367,3	287,5	229,8	187,3	155,4	160	70	8
			z-z	1138	1116	974,7	815,5	647,8	500,3	386,9	304,0	243,6	198,9	165,2	118,8	89,45	69,71	55,83	45,71			

h mm	b mm	t mm	axis	$N_{b.Rd}$ (kN) L_{cr} (m) 0	0,5	1	1,5	2	2,5	3	3,5	4	4,5	5	6	7	8	9	10	h mm	b mm	t mm
160	80	3	y-y	404,2	404,2	403,2	382,1	360,5	337,9	313,8	288,5	262,5	236,8	212,4	169,6	136,0	110,4	90,91	75,96	160	80	3
			z-z	404,2	404,2	373,2	335,0	292,8	248,6	206,6	170,5	141,0	117,7	99,24	72,82	55,46	43,57	35,10	28,86			
160	80	4	y-y	604,7	604,7	598,2	564,2	529,0	491,8	452,0	410,5	368,8	328,5	291,3	228,7	181,4	146,2	119,8	99,78	160	80	4
			z-z	604,7	604,7	549,4	486,3	416,6	345,6	281,5	228,8	187,4	155,3	130,3	95,01	72,10	56,50	45,43	37,31			
160	80	5	y-y	793,6	793,6	781,4	734,7	686,3	634,6	579,5	522,4	465,7	411,9	363,1	282,5	222,8	178,9	146,3	121,7	160	80	5
			z-z	793,6	793,6	713,8	626,1	529,4	433,1	348,8	281,2	229,1	189,2	158,4	115,1	87,19	68,24	54,82	44,99			
160	80	6	y-y	938,4	938,4	922,7	867,0	809,0	747,1	681,1	612,8	545,2	481,4	423,7	329,0	259,1	208,0	170,0	141,3	160	80	6
			z-z	938,4	938,0	841,3	735,7	619,6	504,7	405,1	325,9	265,1	218,7	182,9	132,8	100,6	78,68	63,19	51,85			
160	80	7,1	y-y	1077	1077	1056	990,9	922,8	849,9	772,3	692,3	613,6	540,1	474,0	366,7	288,1	230,9	188,5	156,6	160	80	7,1
			z-z	1077	1074	960,8	836,6	700,3	567,0	452,9	363,3	295,0	243,0	203,0	147,3	111,4	87,11	69,94	57,37			
160	80	8	y-y	1194	1194	1170	1097	1020	938,2	850,9	761,1	673,2	591,5	518,3	400,1	314,0	251,4	205,2	170,3	160	80	8
			z-z	1194	1190	1062	922,2	768,7	619,6	493,5	395,0	320,2	263,6	220,1	159,5	120,6	94,27	75,67	62,06			
160	80	10	y-y	1440	1440	1406	1315	1220	1118	1009	898,0	790,4	691,4	603,8	463,8	362,9	290,1	236,4	196,1	160	80	10
			z-z	1440	1430	1271	1095	903,6	721,2	570,3	454,4	367,3	301,7	251,6	182,0	137,4	107,4	86,13	70,61			
160	90	3	y-y	425,5	425,5	425,0	403,1	380,7	357,2	332,3	306,1	279,1	252,3	226,7	181,6	145,9	118,6	97,76	81,73	160	90	3
			z-z	425,5	425,5	400,6	365,2	326,5	285,0	243,5	205,4	172,8	145,8	123,9	91,89	70,42	55,54	44,86	36,96			
160	90	4	y-y	633,1	633,1	627,5	592,4	556,3	518,1	477,4	434,8	391,7	349,9	311,0	245,0	194,7	157,2	129,0	107,5	160	90	4
			z-z	633,1	633,1	588,2	530,6	467,2	400,2	335,4	278,5	231,6	193,8	163,8	120,5	91,98	72,33	58,31	47,98			
160	90	5	y-y	829,1	829,1	818,1	770,3	720,7	668,0	611,8	553,3	494,9	439,1	388,0	303,1	239,6	192,7	157,8	131,2	160	90	5
			z-z	829,1	829,1	764,0	684,6	596,9	505,2	418,8	344,8	284,9	237,5	200,1	146,7	111,7	87,70	70,63	58,07			
160	90	6	y-y	981,0	981,0	966,8	909,6	850,3	787,1	719,8	649,9	580,2	514,0	453,6	353,5	279,1	224,3	183,5	152,6	160	90	6
			z-z	981,0	981,0	901,7	806,3	700,9	591,1	488,5	401,2	330,9	275,5	232,0	169,9	129,3	101,5	81,69	67,15			
160	90	7,1	y-y	1127	1127	1109	1042	972,1	897,8	818,6	736,6	655,5	578,9	509,5	395,7	311,7	250,2	204,5	170,0	160	90	7,1
			z-z	1127	1127	1032	920,3	796,6	668,3	549,7	450,0	370,3	307,9	258,9	189,3	143,9	112,9	90,87	74,68			
160	90	8	y-y	1251	1251	1229	1154	1076	992,0	903,0	811,0	720,2	634,9	558,1	432,5	340,3	272,9	222,9	185,2	160	90	8
			z-z	1251	1251	1143	1017	877,2	733,3	601,3	491,1	403,5	335,1	281,6	205,7	156,3	122,6	98,62	81,03			
160	90	8,8	y-y	1358	1358	1332	1250	1164	1072	974,2	873,4	774,2	681,5	598,2	462,8	363,7	291,4	238,0	197,6	160	90	8,8
			z-z	1358	1358	1238	1098	945,0	787,4	643,8	524,7	430,6	357,2	300,0	219,0	166,3	130,4	104,9	86,15			

Table 11.2.2 Buckling resistance values for rectangular hollow sections of steel grade S355J2H (f_y = 355 N/mm^2) *continued*



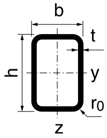
h = depth
b = width
t = wall thickness
L_{cr} = buckling length
$N_{b.Rd}$ = buckling resistance

The calculated resistance values are design values (see Chapter 2) based on recommended partial safety factor values **γ_{M0} = 1,0** and **γ_{M1} = 1,0 (for Class 4 circular hollow sections γ_{M1} = 1,1)** as given in Eurocode 3 (EN 1993). Partial safety factor values may differ in each country. National values must be checked from the National Annex of the relevant country.

h mm	b mm	t mm	axis	$N_{b.Rd}$ (kN) L_{cr} (m) 0	0,5	1	1,5	2	2,5	3	3,5	4	4,5	5	6	7	8	9	10	h mm	b mm	t mm
180	100	4	y-y	678,6	678,6	678,6	650,3	617,5	583,3	547,3	509,3	469,7	429,7	390,5	318,6	259,3	212,5	176,2	147,9	180	100	4
			z-z	678,6	678,6	645,5	593,0	536,3	475,0	412,2	352,5	299,7	255,0	218,0	162,8	125,3	99,10	80,20	66,17			
180	100	5	y-y	935,6	935,6	935,5	887,8	839,2	788,2	734,2	677,3	618,6	560,2	504,1	404,8	325,8	265,1	218,7	182,9	180	100	5
			z-z	935,6	935,6	880,4	802,2	716,8	625,3	533,7	449,9	378,2	319,1	271,2	201,0	154,0	121,4	98,06	80,79			
180	100	5,6	y-y	1040	1040	1039	986,1	931,7	874,7	814,2	750,5	685,0	619,8	557,3	447,0	359,4	292,3	241,1	201,6	180	100	5,6
			z-z	1040	1040	977,5	889,9	794,2	691,7	589,5	496,2	416,6	351,2	298,3	220,9	169,2	133,4	107,7	88,74			
180	100	6	y-y	1109	1109	1108	1051	992,4	931,4	866,6	798,4	728,3	658,6	591,9	474,3	381,2	309,9	255,5	213,6	180	100	6
			z-z	1109	1109	1041	947,4	844,8	734,9	625,6	526,1	441,4	372,0	315,8	233,8	179,0	141,1	113,9	93,84			
180	100	7,1	y-y	1278	1278	1275	1208	1140	1068	992,0	911,8	829,6	748,3	670,9	535,6	429,4	348,5	287,0	239,8	180	100	7,1
			z-z	1278	1278	1197	1087	966,4	837,5	710,1	595,2	498,1	419,0	355,3	262,6	200,9	158,2	127,7	105,2			
180	100	8	y-y	1422	1422	1416	1341	1265	1184	1098	1008	915,8	824,8	738,5	588,3	471,0	381,9	314,3	262,5	180	100	8
			z-z	1422	1422	1329	1205	1069	923,4	780,8	652,9	545,5	458,3	388,3	286,7	219,2	172,6	139,2	114,6			
180	100	8,8	y-y	1545	1545	1538	1456	1372	1283	1189	1090	988,8	889,3	795,3	632,3	505,5	409,6	336,9	281,3	180	100	8,8
			z-z	1545	1545	1442	1305	1155	996,0	840,0	701,0	584,9	490,9	415,6	306,5	234,2	184,3	148,7	122,4			
180	100	10	y-y	1724	1724	1714	1621	1526	1425	1318	1206	1092	980,0	874,6	693,3	553,3	447,7	368,0	307,0	180	100	10
			z-z	1724	1724	1605	1449	1279	1098	922,6	767,6	639,0	535,4	452,8	333,5	254,6	200,3	161,5	132,9			
180	120	4	y-y	735,4	735,4	735,4	707,1	672,4	636,4	598,5	558,5	516,8	474,4	432,5	355,0	290,1	238,5	198,1	166,5	180	120	4
			z-z	735,4	735,4	716,4	669,1	619,3	565,7	508,9	451,1	395,6	345,0	300,5	230,1	179,7	143,4	116,8	96,82			
180	120	5	y-y	1007	1007	1007	959,3	908,6	855,6	799,5	740,4	679,2	617,8	558,3	451,3	364,9	297,8	246,2	206,3	180	120	5
			z-z	1007	1007	972,6	903,4	829,9	750,5	666,9	583,8	506,0	437,1	377,9	286,4	222,4	176,8	143,6	118,8			
180	120	6	y-y	1194	1194	1194	1136	1076	1012	945,1	874,2	801,0	727,6	656,7	529,9	427,8	348,9	288,2	241,4	180	120	6
			z-z	1194	1194	1152	1069	980,9	885,6	785,4	686,1	593,7	512,0	442,3	334,7	259,6	206,4	167,6	138,6			
180	120	7,1	y-y	1379	1379	1379	1310	1239	1164	1085	1002	915,9	830,2	747,7	601,3	484,4	394,4	325,5	272,3	180	120	7,1
			z-z	1379	1379	1328	1231	1127	1015	897,8	782,0	674,9	580,9	501,0	378,4	293,2	232,8	188,9	156,2			
180	120	8	y-y	1535	1535	1534	1456	1376	1292	1203	1110	1013	917,0	824,9	662,0	532,6	433,2	357,4	298,9	180	120	8
			z-z	1535	1535	1476	1367	1250	1124	992,0	862,3	742,9	638,6	550,2	415,0	321,2	255,0	206,9	171,0			
180	120	8,8	y-y	1670	1670	1668	1582	1494	1402	1305	1202	1096	990,8	890,2	713,1	573,0	465,8	384,0	321,0	180	120	8,8
			z-z	1670	1670	1604	1484	1356	1217	1072	930,2	800,2	687,0	591,3	445,4	344,6	273,4	221,7	183,3			
180	120	10	y-y	1866	1866	1862	1764	1665	1561	1450	1333	1214	1095	982,4	784,8	629,4	511,0	421,0	351,7	180	120	10
			z-z	1866	1866	1789	1653	1507	1350	1186	1026	880,4	754,4	648,5	487,7	376,8	298,8	242,2	200,1			
180	120	12,5	y-y	2203	2203	2188	2069	1947	1818	1680	1536	1389	1246	1111	879,8	701,5	567,4	466,3	388,9	180	120	12,5
			z-z	2203	2203	2100	1933	1752	1557	1357	1164	992,7	846,4	724,7	542,4	417,9	330,8	267,8	221,1			

h mm	b mm	t mm	axis	$N_{b.Rd}$ (kN), L_{cr} (m): 0	0,5	1	1,5	2	2,5	3	3,5	4	4,5	5	6	7	8	9	10	h mm	b mm	t mm
200	80	4	y-y	635,4	635,4	635,4	617,0	589,2	560,7	530,8	499,3	466,4	432,5	398,4	333,1	276,1	229,2	191,7	161,9	200	80	4
			z-z	635,4	635,4	586,6	526,5	460,2	390,6	324,6	267,8	221,6	184,9	155,9	114,4	87,13	68,44	55,13	45,33			
200	80	5	y-y	873,8	873,8	873,8	841,2	800,3	758,0	713,4	666,4	617,4	567,5	518,1	426,1	348,8	287,0	238,6	200,7	200	80	5
			z-z	873,8	873,8	795,7	705,8	606,4	504,7	412,2	335,7	275,3	228,3	191,7	139,9	106,2	83,26	66,96	55,00			
200	80	6	y-y	1109	1109	1109	1061	1006	949,6	889,8	826,7	761,2	695,1	630,4	512,8	416,4	340,9	282,3	236,8	200	80	6
			z-z	1109	1109	999,1	877,9	744,2	610,3	492,5	397,8	324,4	268,0	224,4	163,2	123,7	96,81	77,79	63,85			
200	80	7,1	y-y	1278	1278	1278	1220	1156	1089	1018	944,0	867,0	789,5	714,2	578,5	468,3	382,5	316,4	265,2	200	80	7,1
			z-z	1278	1278	1147	1004	846,7	690,5	554,8	446,7	363,6	300,0	251,0	182,3	138,0	108,0	86,75	71,19			
200	80	8	y-y	1422	1422	1422	1354	1282	1207	1128	1044	957,6	870,7	786,5	635,5	513,6	419,1	346,4	290,1	200	80	8
			z-z	1422	1420	1272	1110	932,4	757,4	606,6	487,4	396,2	326,6	273,0	198,2	150,0	117,3	94,20	77,28			
200	80	8,8	y-y	1545	1545	1545	1470	1391	1309	1221	1129	1034	939,0	847,1	683,0	551,2	449,3	371,1	310,7	200	80	8,8
			z-z	1545	1541	1379	1200	1004	812,8	649,2	520,6	422,7	348,1	290,9	211,0	159,6	124,8	100,2	82,19			
200	80	10	y-y	1724	1724	1724	1637	1547	1454	1355	1250	1142	1035	931,7	748,7	602,8	490,7	404,9	338,7	200	80	10
			z-z	1724	1717	1532	1328	1105	889,2	707,3	565,6	458,4	377,1	314,8	228,1	172,4	134,8	108,2	88,71			
200	100	4	y-y	692,2	692,2	692,2	674,5	645,0	614,8	583,3	550,2	515,5	479,7	443,5	373,3	311,0	259,2	217,4	183,9	200	100	4
			z-z	692,2	692,2	662,2	610,9	555,7	496,1	434,3	374,5	320,6	274,1	235,3	176,5	136,3	108,0	87,48	72,25			
200	100	5	y-y	944,8	944,8	944,8	913,5	870,8	826,6	780,3	731,6	680,6	628,4	576,2	477,8	393,4	325,1	271,0	228,4	200	100	5
			z-z	944,8	944,8	895,4	820,3	738,8	651,0	561,7	478,0	404,7	343,2	292,8	218,1	167,6	132,4	107,0	88,28			
200	100	6	y-y	1194,0	1194	1194	1148	1092	1033	971,4	906,5	838,8	769,9	701,9	576,0	470,7	386,9	321,4	270,1	200	100	6
			z-z	1194,0	1194	1123	1024	914,9	798,1	681,4	574,4	482,8	407,4	346,2	256,6	196,6	155,0	125,2	103,2			
200	100	7,1	y-y	1379,0	1379	1379	1323	1257	1188	1116	1039	959,6	878,8	799,4	653,6	532,6	437,0	362,5	304,4	200	100	7,1
			z-z	1379,0	1379	1295	1177	1049	911,8	775,5	651,7	546,4	460,3	390,7	289,1	221,3	174,4	140,8	116,0			
200	100	8	y-y	1535,0	1535	1535	1471	1397	1319	1238	1151	1062	971,3	882,4	719,8	585,7	480,0	398,0	334,0	200	100	8
			z-z	1535,0	1535	1438	1306	1161	1007	853,9	716,0	599,4	504,2	427,6	316,1	241,8	190,5	153,7	126,6			
200	100	8,8	y-y	1670,0	1670	1670	1598	1517	1432	1342	1247	1149	1050	952,7	775,6	630,2	516,0	427,5	358,7	200	100	8,8
			z-z	1670,0	1670	1562	1417	1257	1087	920,0	769,8	643,4	540,8	458,3	338,4	258,7	203,7	164,4	135,4			
200	100	10	y-y	1866,0	1866	1866	1783	1690	1594	1492	1385	1274	1161	1052	853,8	692,3	566,1	468,5	392,8	200	100	10
			z-z	1866,0	1866	1741	1575	1394	1201	1013	844,6	704,5	591,1	500,4	369,0	281,9	221,8	178,9	147,3			
200	100	12,5	y-y	2203,0	2203	2203	2091	1977	1857	1730	1597	1459	1322	1190	955,9	769,6	626,4	516,9	432,4	200	100	12,5
			z-z	2203,0	2203	2040	1835	1609	1372	1145	947,1	785,3	656,2	553,9	406,9	310,2	243,8	196,4	161,6			

Table 11.2.2 Buckling resistance values for rectangular hollow sections of steel grade S355J2H (f_y = 355 N/mm^2) *continued*



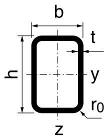
h = depth
b = width
t = wall thickness
L_{cr} = buckling length
$N_{b.Rd}$ = buckling resistance

The calculated resistance values are design values (see Chapter 2) based on recommended partial safety factor values **γ_{M0} = 1,0** and **γ_{M1} = 1,0 (for Class 4 circular hollow sections γ_{M1} = 1,1)** as given in Eurocode 3 (EN 1993). Partial safety factor values may differ in each country. National values must be checked from the National Annex of the relevant country.

h mm	b mm	t mm	axis	$N_{b.Rd}$ (kN) L_{cr} (m) 0	0,5	1	1,5	2	2,5	3	3,5	4	4,5	5	6	7	8	9	10	h mm	b mm	t mm
200	120	5	y-y	1016,0	1016	1016	985,5	940,8	894,7	846,5	795,8	742,7	688,1	633,3	528,5	437,4	362,7	303,2	255,9	200	120	5
			z-z	1016,0	1016	987,1	920,4	849,9	774,1	693,8	612,7	535,4	465,4	404,5	308,7	240,6	191,8	156,1	129,3			
200	120	6	y-y	1279,0	1279	1279	1235	1176	1116	1052	984,8	914,7	843,1	771,8	637,9	524,1	432,4	360,1	303,2	200	120	6
			z-z	1279,0	1279	1236	1149	1056	955,2	849,4	743,9	645,3	557,6	482,3	365,7	284,0	225,9	183,5	151,8			
200	120	7,1	y-y	1480,0	1480	1480	1426	1357	1286	1211	1132	1050	965,7	882,4	726,9	595,6	490,6	408,0	343,3	200	120	7,1
			z-z	1480,0	1480	1428	1325	1216	1097	973,2	850,0	735,5	634,3	547,8	414,6	321,6	255,5	207,5	171,6			
200	120	8	y-y	1649,0	1649	1649	1587	1510	1429	1345	1256	1164	1069	975,8	802,3	656,5	540,1	448,9	377,5	200	120	8
			z-z	1649,0	1649	1589	1473	1350	1216	1077	938,5	810,7	698,2	602,4	455,3	352,8	280,3	227,5	188,1			
200	120	8,8	y-y	1795,0	1795	1795	1726	1641	1553	1461	1363	1261	1158	1055	866,2	707,8	581,9	483,3	406,3	200	120	8,8
			z-z	1795,0	1795	1728	1601	1465	1318	1165	1014	874,2	752,0	648,2	489,3	379,0	300,9	244,2	201,9			
200	120	10	y-y	2008,0	2008	2008	1928	1832	1732	1627	1516	1401	1284	1169	956,6	780,1	640,4	531,5	446,4	200	120	10
			z-z	2008,0	2008	1929	1785	1631	1464	1290	1120	963,6	827,4	712,3	536,7	415,2	329,5	267,2	220,9			
200	120	12,5	y-y	2380,0	2380	2380	2273	2154	2031	1901	1763	1621	1477	1338	1085	879,1	718,6	594,6	498,5	200	120	12,5
			z-z	2380,0	2380	2275	2098	1907	1700	1487	1281	1095	935,5	802,4	601,8	464,3	367,7	297,9	246,0			
220	120	5	y-y	1038,0	1038	1038	1021	979,6	938,0	894,8	849,7	802,5	753,5	703,2	603,2	510,6	430,5	364,1	309,9	220	120	5
			z-z	1038,0	1038	1013	946,8	877,5	803,2	724,3	643,8	566,0	494,6	431,6	331,2	259,0	206,9	168,6	139,8			
220	120	6	y-y	1364,0	1364	1364	1332	1275	1217	1156	1092	1025	955,4	885,1	748,1	625,3	522,4	438,8	371,7	220	120	6
			z-z	1364,0	1364	1321	1228	1130	1025	913,1	801,6	696,6	603,0	522,2	396,6	308,3	245,4	199,4	165,1			
220	120	7,1	y-y	1581,0	1581	1581	1541	1474	1405	1334	1258	1179	1098	1015	855,5	713,2	594,6	498,8	422,2	220	120	7,1
			z-z	1581,0	1581	1528	1419	1304	1179	1048	917,8	795,8	687,4	594,5	450,7	349,9	278,3	226,0	187,0			
220	120	8	y-y	1762,0	1762	1762	1716	1641	1564	1483	1398	1310	1218	1125	946,2	787,7	656,0	549,9	465,1	220	120	8
			z-z	1762,0	1762	1701	1579	1449	1308	1161	1014	878,1	757,6	654,5	495,5	384,4	305,5	248,1	205,2			
220	120	8,8	y-y	1920,0	1920	1920	1868	1786	1701	1612	1519	1422	1321	1219	1024	851,0	708,0	593,0	501,4	220	120	8,8
			z-z	1920,0	1920	1851	1717	1574	1419	1257	1097	947,9	816,8	705,0	533,1	413,3	328,3	266,5	220,4			
220	120	10	y-y	2150,0	2150	2150	2089	1996	1900	1799	1694	1583	1469	1354	1134	940,6	781,4	653,8	552,4	220	120	10
			z-z	2150,0	2150	2069	1917	1754	1579	1395	1214	1047	900,2	775,9	585,7	453,6	360,1	292,2	241,6			

h mm	b mm	t mm	axis	$N_{b.Rd}$ (kN), L_{cr} (m): 0	0,5	1	1,5	2	2,5	3	3,5	4	4,5	5	6	7	8	9	10	h mm	b mm	t mm
250	100	5	y-y	992,7	992,7	992,7	990,0	955,4	920,7	885,1	848,3	810,0	770,1	728,8	643,8	560,2	483,1	415,5	358,1	250	100	5
			z-z	992,7	992,7	952,2	880,1	802,7	719,0	632,0	547,1	469,8	402,7	346,2	260,5	201,3	159,6	129,4	106,9			
250	100	6	y-y	1289	1289	1289	1279	1231	1184	1135	1084	1030	974,8	917,6	801,2	689,2	588,7	502,6	430,8	250	100	6
			z-z	1289	1289	1226	1126	1019	902,1	782,7	669,4	569,0	484,1	413,8	309,1	237,9	188,1	152,2	125,6			
250	100	8	y-y	1819	1819	1819	1791	1720	1647	1573	1494	1413	1328	1240	1066	904,3	763,7	646,5	550,8	250	100	8
			z-z	1819	1819	1711	1559	1392	1214	1036	872,8	733,3	618,6	525,6	389,5	298,4	235,3	190,0	156,5			
250	100	10	y-y	2221	2221	2221	2182	2094	2004	1910	1813	1711	1605	1496	1281	1082	911,2	769,8	654,9	250	100	10
			z-z	2221	2221	2081	1890	1681	1457	1236	1036	867,5	729,8	619,0	457,5	350,0	275,7	222,6	183,3			
250	150	5	y-y	1170	1170	1170	1170	1134	1095	1055	1014	971,4	927,2	881,4	786,4	691,2	601,3	520,9	451,5	250	150	5
			z-z	1170	1170	1170	1117	1058	997,1	932,5	864,5	794,0	723,1	654,2	529,9	429,0	350,5	289,9	242,9			
250	150	6	y-y	1502	1502	1502	1499	1446	1394	1340	1285	1227	1167	1105	976,4	850,0	733,4	631,0	544,1	250	150	6
			z-z	1502	1502	1500	1423	1344	1261	1173	1081	985,5	890,9	800,5	641,2	515,2	418,7	345,2	288,6			
250	150	7,1	y-y	1883	1883	1883	1869	1801	1731	1660	1586	1509	1429	1346	1177	1014	866,9	740,8	635,4	250	150	7,1
			z-z	1883	1883	1872	1770	1665	1555	1437	1314	1189	1066	951,1	753,2	600,7	486,0	399,3	333,1			
250	150	8	y-y	2103	2103	2103	2086	2009	1931	1851	1768	1681	1591	1497	1307	1125	960,8	820,4	703,3	250	150	8
			z-z	2103	2103	2088	1974	1856	1732	1599	1461	1320	1182	1053	833,0	663,7	536,5	440,7	367,5			
250	150	8,8	y-y	2295	2295	2295	2275	2190	2105	2017	1925	1830	1731	1628	1420	1220	1041	888,4	761,2	250	150	8,8
			z-z	2295	2295	2277	2151	2022	1885	1740	1587	1432	1282	1141	901,0	717,2	579,5	475,8	396,6			
250	150	10	y-y	2576	2576	2576	2551	2455	2359	2259	2155	2047	1934	1818	1583	1358	1157	985,8	843,9	250	150	10
			z-z	2576	2576	2554	2411	2264	2109	1943	1770	1595	1425	1267	998,0	793,4	640,4	525,5	437,9			
250	150	12,5	y-y	3090	3090	3090	3051	2933	2814	2690	2562	2427	2288	2144	1854	1581	1341	1139	972,7	250	150	12,5
			z-z	3090	3090	3054	2879	2697	2505	2299	2085	1870	1663	1473	1154	914,3	736,4	603,3	502,2			
260	140	6	y-y	1473	1473	1473	1473	1425	1375	1325	1272	1218	1162	1104	982,8	862,1	748,7	647,6	560,7	260	140	6
			z-z	1473	1473	1463	1383	1301	1214	1122	1026	927,4	831,4	741,3	586,8	467,8	378,3	310,8	259,2			
260	140	7,1	y-y	1883	1883	1883	1874	1807	1740	1671	1599	1524	1446	1366	1201	1040	893,6	766,3	659,0	260	140	7,1
			z-z	1883	1883	1859	1750	1638	1519	1392	1259	1127	1000	884,2	690,9	546,4	439,6	359,9	299,5			
260	140	8	y-y	2103	2103	2103	2091	2016	1940	1863	1782	1698	1610	1520	1335	1154	990,5	848,6	729,3	260	140	8
			z-z	2103	2103	2074	1952	1826	1691	1548	1398	1250	1108	978,4	763,4	603,1	484,9	396,9	330,1			
260	140	8,8	y-y	2295	2295	2295	2281	2198	2115	2030	1941	1849	1752	1653	1450	1253	1074	919,1	789,4	260	140	8,8
			z-z	2295	2295	2261	2127	1988	1840	1682	1518	1355	1200	1059	825,0	651,3	523,4	428,2	356,0			
260	140	10	y-y	2576	2576	2576	2558	2465	2370	2273	2173	2068	1959	1846	1616	1394	1193	1020	875,3	260	140	10
			z-z	2576	2576	2535	2383	2226	2057	1878	1692	1507	1332	1174	912,7	719,5	577,8	472,4	392,7			
260	140	12,5	y-y	3090	3090	3090	3059	2944	2828	2708	2583	2453	2317	2177	1894	1624	1383	1178	1008	260	140	12,5
			z-z	3090	3090	3031	2844	2649	2439	2217	1987	1761	1550	1361	1053	827,1	662,8	541,2	449,4			

Table 11.2.2 Buckling resistance values for rectangular hollow sections of steel grade S355J2H (f_y = 355 N/mm^2) *continued*



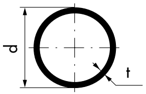
h = depth
b = width
t = wall thickness
L_{cr} = buckling length
$N_{b.Rd}$ = buckling resistance

The calculated resistance values are design values (see Chapter 2) based on recommended partial safety factor values γ_{M0} **= 1,0** and γ_{M1} **= 1,0 (for Class 4 circular hollow sections** γ_{M1} **= 1,1)** as given in Eurocode 3 (EN 1993). Partial safety factor values may differ in each country. National values must be checked from the National Annex of the relevant country.

h mm	b mm	t mm	axis	$N_{b.Rd}$ (kN) L_{cr} (m) 0	0,5	1	1,5	2	2,5	3	3,5	4	4,5	5	6	7	8	9	10	h mm	b mm	t mm
260	180	6	y-y	1643	1643	1643	1643	1597	1543	1489	1433	1375	1315	1252	1122	991,2	866,2	753,1	654,5	260	180	6
			z-z	1643	1643	1643	1598	1527	1454	1378	1297	1213	1127	1040	871,7	724,1	602,0	504,0	426,0			
260	180	7,1	y-y	2085	2085	2085	2083	2012	1940	1867	1791	1713	1631	1546	1371	1198	1036	893,6	771,8	260	180	7,1
			z-z	2085	2085	2085	2013	1918	1820	1716	1607	1494	1377	1261	1043	857,6	707,8	589,6	496,5			
260	180	8	y-y	2330	2330	2330	2327	2247	2166	2084	1999	1910	1818	1723	1526	1331	1151	991,7	856,0	260	180	8
			z-z	2330	2330	2330	2248	2141	2031	1914	1791	1663	1532	1402	1158	950,8	784,1	652,8	549,5			
260	180	8,8	y-y	2545	2545	2545	2540	2452	2363	2273	2180	2082	1981	1876	1660	1447	1250	1076	928,3	260	180	8,8
			z-z	2545	2545	2545	2454	2336	2214	2086	1951	1810	1666	1524	1257	1031	849,4	706,8	594,8			
260	180	10	y-y	2860	2860	2860	2852	2753	2653	2550	2444	2334	2219	2100	1855	1615	1392	1198	1032	260	180	10
			z-z	2860	2860	2860	2754	2621	2483	2337	2184	2024	1861	1700	1399	1146	943,0	784,1	659,5			
300	100	5	y-y	1024	1024	1024	1024	1013	984,5	955,7	926,4	896,3	865,4	833,4	766,3	696,2	625,6	557,5	494,5	300	100	5
			z-z	1024	1024	991,9	923,0	849,9	771,2	688,1	604,7	526,1	455,8	395,0	300,3	233,6	185,9	151,2	125,1			
300	100	6	y-y	1344	1344	1344	1344	1322	1282	1242	1201	1158	1115	1070	974,9	876,9	779,9	688,6	605,9	300	100	6
			z-z	1344	1344	1291	1195	1091	979,4	862,8	748,5	643,9	552,8	475,8	358,4	277,3	220,0	178,4	147,5			
300	100	7,1	y-y	1704	1704	1704	1704	1664	1611	1557	1501	1444	1384	1323	1194	1062	935,0	817,9	714,3	300	100	7,1
			z-z	1704	1704	1623	1492	1351	1199	1042	893,2	760,4	647,7	554,2	414,3	319,1	252,4	204,3	168,6			
300	100	8	y-y	2017	2017	2017	2017	2038	1968	1897	1824	1748	1669	1587	1417	1246	1085	940,2	815,2	300	100	8
			z-z	2017	2017	1909	1747	1571	1382	1190	1011	855,1	724,6	617,7	459,6	353,0	278,7	225,4	185,8			
300	100	8,8	y-y	2295	2295	2295	2295	2222	2146	2067	1987	1903	1816	1726	1539	1352	1176	1018	882,1	300	100	8,8
			z-z	2295	2295	2161	1971	1763	1540	1316	1111	934,5	789,0	670,8	497,4	381,3	300,7	242,9	200,2			
300	100	10	y-y	2576	2576	2576	2576	2492	2405	2316	2225	2129	2031	1928	1716	1505	1306	1129	977,5	300	100	10
			z-z	2576	2576	2421	2203	1966	1711	1458	1227	1030	868,1	737,3	545,9	418,1	329,6	266,1	219,2			
300	100	12,5	y-y	3090	3090	3090	3083	2976	2868	2758	2643	2525	2401	2273	2010	1750	1510	1300	1121	300	100	12,5
			z-z	3090	3090	2886	2615	2318	2000	1690	1412	1179	989,8	838,3	618,6	472,8	372,2	300,3	247,2			

h mm	b mm	t mm	axis	$N_{b.Rd}$ (kN)																h mm	b mm	t mm
				L_{cr} (m)																		
				0	0,5	1	1,5	2	2,5	3	3,5	4	4,5	5	6	7	8	9	10			
300	150	6	y-y	1557	1557	1557	1557	1540	1496	1451	1406	1360	1312	1263	1160	1052	943,9	839,9	744,0	300	150	6
			z-z	1557	1557	1557	1490	1414	1334	1250	1162	1070	977,2	886,4	721,3	585,8	479,5	397,2	333,2			
300	150	7,1	y-y	1956	1956	1956	1956	1923	1865	1807	1747	1686	1622	1556	1418	1276	1135	1002	881,3	300	150	7,1
			z-z	1956	1956	1956	1858	1757	1651	1539	1421	1299	1178	1061	852,9	687,1	559,4	461,7	386,3			
300	150	8	y-y	2301	2301	2301	2301	2333	2258	2183	2106	2026	1943	1858	1679	1495	1318	1154	1008	300	150	8
			z-z	2301	2301	2294	2173	2050	1921	1783	1639	1490	1344	1204	961,0	770,2	625,0	514,7	429,9			
300	150	10	y-y	2931	2931	2931	2931	2860	2767	2674	2577	2478	2375	2268	2045	1817	1597	1396	1218	300	150	10
			z-z	2931	2931	2912	2753	2590	2418	2235	2043	1847	1656	1477	1169	932,4	754,1	619,6	516,8			
300	150	12,5	y-y	3534	3534	3534	3534	3437	3323	3207	3088	2964	2836	2703	2426	2146	1878	1634	1422	300	150	12,5
			z-z	3534	3534	3502	3306	3104	2890	2662	2424	2183	1949	1732	1364	1084	874,7	717,7	598,0			
300	200	6	y-y	1770	1770	1770	1770	1757	1708	1660	1610	1560	1508	1454	1342	1224	1104	988,2	879,6	300	200	6
			z-z	1770	1770	1770	1752	1686	1619	1550	1479	1404	1326	1245	1083	927,6	789,7	672,5	575,4			
300	200	7,1	y-y	2208	2208	2208	2208	2181	2118	2055	1990	1924	1856	1785	1637	1483	1328	1180	1044	300	200	7,1
			z-z	2208	2208	2208	2175	2089	2002	1912	1817	1719	1616	1511	1301	1104	933,3	790,6	673,8			
300	200	8	y-y	2585	2585	2585	2585	2624	2545	2465	2383	2298	2211	2120	1931	1735	1542	1360	1195	300	200	8
			z-z	2585	2585	2585	2537	2434	2328	2219	2105	1985	1861	1734	1482	1250	1052	887,9	754,9			
300	200	8,8	y-y	2920	2920	2920	2920	2867	2780	2692	2602	2509	2413	2313	2105	1890	1678	1479	1300	300	200	8,8
			z-z	2920	2920	2920	2859	2739	2617	2491	2358	2219	2074	1927	1639	1376	1154	971,6	824,6			
300	200	10	y-y	3286	3286	3286	3286	3225	3126	3026	2924	2818	2709	2597	2360	2117	1877	1653	1451	300	200	10
			z-z	3286	3286	3286	3215	3079	2941	2797	2646	2488	2324	2158	1831	1535	1286	1082	917,6			
300	200	12,5	y-y	3978	3978	3978	3978	3894	3772	3648	3522	3391	3256	3116	2823	2522	2229	1956	1713	300	200	12,5
			z-z	3978	3978	3978	3882	3715	3543	3365	3178	2982	2779	2573	2173	1815	1515	1272	1077			
400	200	6	y-y	1837	1837	1837	1837	1837	1837	1807	1771	1735	1698	1661	1585	1505	1422	1335	1246	400	200	6
			z-z	1837	1837	1837	1837	1778	1716	1653	1588	1520	1450	1377	1226	1075	934,0	808,0	699,4			
400	200	7,1	y-y	2321	2321	2321	2321	2321	2318	2271	2223	2175	2126	2077	1975	1868	1756	1640	1522	400	200	7,1
			z-z	2321	2321	2321	2313	2231	2149	2065	1978	1888	1794	1696	1496	1299	1118	960,9	827,5			
400	200	8	y-y	2749	2749	2749	2749	2749	2736	2678	2620	2561	2501	2440	2315	2183	2045	1902	1758	400	200	8
			z-z	2749	2749	2749	2730	2630	2529	2426	2319	2207	2090	1970	1724	1487	1272	1088	933,4			
400	200	8,8	y-y	3143	3143	3143	3143	3143	3120	3051	2983	2913	2843	2772	2624	2468	2305	2137	1968	400	200	8,8
			z-z	3143	3143	3143	3111	2994	2876	2754	2627	2494	2357	2214	1926	1651	1406	1198	1025			
400	200	10	y-y	3749	3749	3749	3749	3749	3707	3623	3538	3453	3366	3277	3092	2898	2695	2487	2279	400	200	10
			z-z	3749	3749	3749	3696	3552	3405	3253	3095	2929	2757	2580	2226	1893	1603	1359	1159			
400	200	12,5	y-y	4865	4865	4865	4865	4865	4784	4669	4554	4436	4317	4195	3940	3671	3391	3107	2827	400	200	12,5
			z-z	4865	4865	4865	4769	4572	4371	4162	3943	3714	3476	3234	2755	2319	1947	1641	1394			

Table 11.2.3 Buckling resistance values for circular longitudinally welded hollow sections of steel grade S355J2H (f_y = 355 N/mm^2)



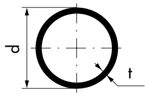
d = external diameter
t = wall thickness
L_{cr} = buckling length
$N_{b.Rd}$ = buckling resistance

The calculated resistance values are design values (see Chapter 2) based on recommended partial safety factor values γ_{M0} **= 1,0** and γ_{M1} **= 1,0 (for Class 4 circular hollow sections** γ_{M1} **= 1,1)** as given in Eurocode 3 (EN 1993). Partial safety factor values may differ in each country. National values must be checked from the National Annex of the relevant country.

d mm	t mm	$N_{b.Rd}$ (kN) L_{cr} (m) 0	0,5	1	1,5	2	2,5	3	3,5	4	4,5	5	6	7	8	9	10	d mm	t mm
26,9	2	55,54	38,84	17,80	9,07	5,40												26,9	2
26,9	2,5	68,03	46,99	21,20	10,76	6,40												26,9	2,5
26,9	2,6	70,46	48,54	21,83	11,07	6,58												26,9	2,6
33,7	2	70,71	56,25	31,86	17,41	10,62	7,10	5,06										33,7	2
33,7	2,5	86,99	68,77	38,46	20,90	12,72	8,50	6,06										33,7	2,5
33,7	2,6	90,18	71,20	39,72	21,56	13,12	8,76	6,25										33,7	2,6
33,7	3	102,7	80,67	44,55	24,09	14,64	9,77	6,97										33,7	3
33,7	3,2	108,9	85,26	46,85	25,28	15,35	10,24	7,30										33,7	3,2
42,4	2	90,11	78,08	53,22	32,40	20,55	13,98	10,08	7,59									42,4	2
42,4	2,5	111,2	96,06	64,97	39,31	24,88	16,91	12,18	9,17									42,4	2,5
42,4	2,6	115,4	99,58	67,25	40,64	25,71	17,47	12,58	9,48									42,4	2,6
42,4	2,9	127,8	110,0	73,93	44,52	28,12	19,10	13,75	10,35									42,4	2,9
42,4	3	131,8	113,4	76,12	45,78	28,90	19,62	14,13	10,64									42,4	3
42,4	3,2	139,9	120,2	80,41	48,25	30,43	20,65	14,87	11,19									42,4	3,2
42,4	4	171,3	146,4	96,65	57,46	36,11	24,47	17,60	13,24									42,4	4
48,3	2	103,3	92,70	68,46	44,89	29,43	20,32	14,76	11,18	8,74								48,3	2
48,3	2,5	127,7	114,3	83,99	54,75	35,79	24,68	17,92	13,56	10,61								48,3	2,5
48,3	2,6	132,5	118,6	87,02	56,66	37,02	25,52	18,52	14,02	10,97								48,3	2,6
48,3	3	151,6	135,4	98,90	64,10	41,79	28,78	20,88	15,80	12,35								48,3	3
48,3	3,2	161,0	143,6	104,7	67,70	44,08	30,34	22,01	16,65	13,02								48,3	3,2
48,3	4	197,6	175,6	126,9	81,30	52,72	36,22	26,24	19,84	15,51								48,3	4
60,3	2	130,0	122,3	99,44	73,91	52,29	37,47	27,75	21,26	16,76	13,54	11,15						60,3	2
60,3	2,5	161,2	151,3	122,7	90,83	64,06	45,82	33,92	25,97	20,47	16,52	13,61						60,3	2,5
60,3	2,9	185,6	174,1	140,9	103,9	73,12	52,23	38,63	29,57	23,30	18,81	15,49						60,3	2,9
60,3	3	191,7	179,8	145,4	107,1	75,33	53,80	39,78	30,44	23,99	19,36	15,95						60,3	3
60,3	3,2	203,8	191,0	154,3	113,5	79,71	56,88	42,05	32,17	25,35	20,46	16,85						60,3	3,2
60,3	4	251,2	234,8	188,8	138,0	96,43	68,64	50,68	38,74	30,50	24,61	20,26						60,3	4
60,3	5	308,4	287,4	229,8	166,5	115,7	82,08	60,50	46,21	36,36	29,32	24,14						60,3	5

d	t	$N_{b.Rd}$ (kN)																d	t
mm	mm	L_{cr} (m)																mm	mm
		0	0,5	1	1,5	2	2,5	3	3,5	4	4,5	5	6	7	8	9	10		
76,1	2	165,3	161,1	139,4	114,7	89,36	67,98	52,09	40,70	32,50	26,47	21,94	15,75					76,1	2
76,1	2,5	205,2	199,9	172,7	141,8	110,2	83,65	64,02	49,99	39,89	32,49	26,93	19,32					76,1	2,5
76,1	2,9	236,7	230,4	198,9	163,0	126,4	95,81	73,26	57,17	45,61	37,13	30,77	22,08					76,1	2,9
76,1	3	244,6	238,0	205,4	168,3	130,4	98,80	75,53	58,93	47,01	38,27	31,72	22,76					76,1	3
76,1	4	321,6	312,4	268,9	219,3	169,0	127,6	97,32	75,84	60,45	49,18	40,74	29,21					76,1	4
76,1	5	396,5	384,4	329,9	267,8	205,3	154,4	117,5	91,47	72,85	59,24	49,05	35,16					76,1	5
76,1	6	469,1	454,0	388,5	313,8	239,3	179,4	136,2	105,9	84,27	68,49	56,69	40,61					76,1	6
76,1	6,3	490,4	474,4	405,6	327,1	249,1	186,5	141,6	110,0	87,51	71,11	58,85	42,15					76,1	6,3
88,9	2	193,8	192,6	171,3	147,7	122,1	97,57	77,23	61,57	49,79	40,91	34,12	24,69	18,65				88,9	2
88,9	2,5	240,9	239,2	212,5	183,1	151,0	120,5	95,23	75,86	61,31	50,36	41,99	30,37	22,94				88,9	2,5
88,9	3	287,4	285,2	253,2	217,8	179,3	142,8	112,7	89,72	72,48	59,51	49,61	35,87	27,09				88,9	3
88,9	3,2	305,9	303,4	269,3	231,5	190,4	151,5	119,6	95,15	76,85	63,08	52,58	38,02	28,71				88,9	3,2
88,9	4	378,7	375,3	332,7	285,3	234,0	185,7	146,2	116,2	93,77	76,93	64,10	46,33	34,97				88,9	4
88,9	5	467,9	463,0	409,7	350,4	286,2	226,3	177,8	141,0	113,7	93,23	77,65	56,08	42,31				88,9	5
88,9	6	554,7	548,3	484,3	412,9	335,9	264,7	207,4	164,3	132,4	108,4	90,28	65,16	49,15				88,9	6
88,9	6,3	580,4	573,4	506,2	431,2	350,4	275,8	216,0	171,0	137,7	112,8	93,89	67,76	51,10				88,9	6,3
101,6	2	222,2	222,2	202,7	180,0	155,0	129,3	105,9	86,33	70,87	58,82	49,41	36,08	27,40	21,48			101,6	2
101,6	2,5	276,3	276,3	251,8	223,5	192,2	160,1	130,9	106,6	87,48	72,57	60,94	44,48	33,78	26,48			101,6	2,5
101,6	3	329,9	329,9	300,3	266,3	228,7	190,2	155,3	126,4	103,7	85,96	72,16	52,65	39,97	31,33			101,6	3
101,6	3,6	393,5	393,5	357,7	316,8	271,7	225,6	183,9	149,5	122,5	101,5	85,21	62,15	47,17	36,97			101,6	3,6
101,6	4	435,4	435,4	395,5	350,0	299,8	248,6	202,5	164,5	134,7	111,6	93,67	68,30	51,83	40,61			101,6	4
101,6	5	538,7	538,7	488,3	431,3	368,4	304,6	247,4	200,7	164,2	135,9	114,0	83,05	62,99	49,35			101,6	5
101,6	6	639,7	639,7	578,6	510,1	434,5	358,1	290,2	235,0	192,0	158,8	133,1	96,93	73,49	57,56			101,6	6
101,6	6,3	669,6	669,6	605,2	533,2	453,8	373,7	302,6	244,9	200,1	165,5	138,7	100,9	76,52	59,92			101,6	6,3
108	2	236,4	236,4	218,4	196,2	171,6	145,8	121,3	100,1	82,87	69,17	58,33	42,81	32,62	25,62	20,64		108	2
108	2,5	294,2	294,2	271,5	243,6	212,9	180,6	150,1	123,8	102,4	85,42	72,02	52,84	40,24	31,61	25,46		108	2,5
108	3	351,3	351,3	324,0	290,5	253,5	214,8	178,3	146,9	121,4	101,3	85,36	62,60	47,67	37,44	30,15		108	3
108	3,6	419,2	419,2	386,1	345,9	301,5	255,1	211,4	174,0	143,7	119,8	100,9	73,97	56,31	44,22	35,61		108	3,6
108	4	464,0	464,0	427,1	382,3	333,0	281,4	232,9	191,6	158,2	131,8	111,0	81,35	61,92	48,61	39,14		108	4
108	5	574,4	574,4	527,7	471,7	409,8	345,4	285,3	234,2	193,1	160,8	135,3	99,10	75,39	59,17	47,64		108	5
108	6	682,5	682,5	626,0	558,6	484,2	407,0	335,3	274,8	226,3	188,3	158,4	115,9	88,12	69,15	55,65		108	6
108	6,3	714,6	714,6	654,9	584,2	506,0	425,0	349,9	286,6	236,0	196,2	165,1	120,7	91,80	72,02	57,97		108	6,3
114,3	2	250,5	250,5	234,0	211,9	187,8	162,1	136,9	114,4	95,48	80,18	67,91	50,11	38,30	30,15	24,32	20,02	114,3	2
114,3	2,5	311,7	311,7	290,9	263,4	233,2	201,0	169,6	141,5	118,1	99,11	83,91	61,89	47,29	37,22	30,02	24,72	114,3	2,5
114,3	3	372,4	372,4	347,3	314,2	277,9	239,3	201,6	168,1	140,2	117,6	99,54	73,38	56,06	44,12	35,58	29,29	114,3	3
114,3	3,6	444,5	444,5	414,1	374,3	330,7	284,4	239,3	199,3	166,1	139,2	117,8	86,80	66,28	52,15	42,06	34,62	114,3	3,6
114,3	4	492,1	492,1	458,1	414,0	365,4	314,0	263,9	219,7	182,9	153,3	129,7	95,52	72,93	57,38	46,27	38,08	114,3	4
114,3	5	609,5	609,5	566,5	511,2	450,5	386,1	323,8	269,0	223,7	187,3	158,4	116,6	88,95	69,95	56,40	46,41	114,3	5
114,3	6	724,7	724,7	672,5	606,1	533,0	455,7	381,4	316,3	262,7	219,7	185,6	136,5	104,1	81,87	65,99	54,29	114,3	6
114,3	6,3	758,8	758,8	703,8	634,0	557,3	476,1	398,2	330,0	274,0	229,2	193,5	142,3	108,5	85,32	68,77	56,57	114,3	6,3

Table 11.2.3 Buckling resistance values for circular longitudinally welded hollow sections of steel grade S355J2H (f_y = 355 N/mm^2) *continued*



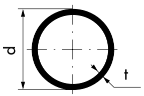
d = external diameter
t = wall thickness
L_{cr} = buckling length
$N_{b.Rd}$ = buckling resistance

The calculated resistance values are design values (see Chapter 2) based on recommended partial safety factor values **γ_{M0} = 1,0** and **γ_{M1} = 1,0 (for Class 4 circular hollow sections γ_{M1} = 1,1)** as given in Eurocode 3 (EN 1993). Partial safety factor values may differ in each country. National values must be checked from the National Annex of the relevant country.

d mm	t mm	$N_{b.Rd}$ (kN) L_{cr} (m) 0	0,5	1	1,5	2	2,5	3	3,5	4	4,5	5	6	7	8	9	10	d mm	t mm
127	2	223,7	223,7	214,8	198,7	181,4	162,7	143,3	124,2	106,8	91,68	78,88	59,41	45,95	36,45	29,56	24,43	127	2
127	2,5	347,1	347,1	330,0	303,0	273,8	242,2	210,0	179,4	152,4	129,6	110,7	82,66	63,60	50,28	40,69	33,57	127	2,5
127	3	414,9	414,9	394,1	361,7	326,6	288,7	250,1	213,5	181,2	154,0	131,6	98,15	75,49	59,68	48,28	39,83	127	3
127	4	548,7	548,7	520,6	477,3	430,4	379,9	328,3	279,8	237,2	201,4	171,9	128,1	98,49	77,83	62,95	51,93	127	4
127	5	680,3	680,3	644,6	590,5	531,8	468,5	404,1	343,8	291,0	246,8	210,5	156,8	120,5	95,15	76,95	63,46	127	5
127	6	809,7	809,7	766,2	701,2	630,6	554,5	477,4	405,5	342,8	290,4	247,5	184,1	141,4	111,7	90,28	74,44	127	6
127	6,3	848,1	848,1	802,2	733,9	659,7	579,8	498,9	423,5	357,9	303,1	258,3	192,1	147,5	116,5	94,16	77,63	127	6,3
133	2	232,7	232,7	225,1	209,3	192,4	174,2	155,1	136,0	118,0	102,1	88,33	67,03	52,07	41,43	33,66	27,86	133	2
133	2,5	363,9	363,9	348,4	321,6	292,8	261,7	229,3	198,0	169,7	145,2	124,7	93,63	72,30	57,29	46,43	38,35	133	2,5
133	3	435,0	435,0	416,2	384,0	349,4	312,1	273,3	235,8	201,9	172,6	148,2	111,2	85,87	68,04	55,13	45,53	133	3
133	4	575,5	575,5	550,1	507,1	460,9	411,0	359,3	309,4	264,6	226,0	193,9	145,4	112,2	88,83	71,96	59,42	133	4
133	5	713,8	713,8	681,5	627,7	569,9	507,3	442,7	380,7	325,0	277,4	237,7	178,1	137,3	108,7	88,06	72,70	133	5
133	6	849,8	849,8	810,5	745,9	676,3	601,1	523,7	449,5	383,3	326,8	279,9	209,4	161,4	127,7	103,4	85,38	133	6
133	6,3	890,2	890,2	848,7	780,9	707,8	628,8	547,5	469,7	400,3	341,2	292,2	218,6	168,4	133,3	107,9	89,07	133	6,3
139,7	3	457,4	457,4	440,9	408,9	374,8	338,0	299,4	261,1	225,7	194,5	167,8	126,9	98,37	78,15	63,44	52,47	139,7	3
139,7	4	605,4	605,4	583,0	540,3	494,8	445,6	394,0	343,2	296,2	254,9	219,8	166,0	128,6	102,2	82,91	68,55	139,7	4
139,7	5	751,1	751,1	722,6	669,3	612,2	550,6	486,1	422,7	364,3	313,3	269,9	203,7	157,7	125,2	101,6	83,96	139,7	5
139,7	6	894,7	894,7	859,8	795,7	727,2	653,1	575,7	499,8	430,2	369,5	318,2	239,8	185,5	147,2	119,4	98,71	139,7	6
139,7	6,3	937,3	937,3	900,5	833,2	761,2	683,3	602,1	522,5	449,6	386,0	332,3	250,3	193,7	153,7	124,6	103,0	139,7	6,3
139,7	8	1175	1175	1127	1041	949,7	850,5	747,4	646,9	555,3	475,9	409,1	307,7	237,8	188,6	152,9	126,3	139,7	8
139,7	10	1446	1446	1384	1277	1162	1038	909,2	784,4	671,6	574,5	493,1	370,2	285,8	226,4	183,5	151,5	139,7	10
152,4	3	499,9	499,9	487,7	456,0	422,7	386,8	348,8	310,0	272,5	238,2	207,8	159,5	124,7	99,62	81,18	67,31	152,4	3
152,4	4	662,0	662,0	645,4	603,1	558,6	510,7	459,9	408,3	358,5	312,9	272,8	209,1	163,4	130,5	106,3	88,11	152,4	4
152,4	5	822,0	822,0	800,6	747,7	692,0	632,1	568,5	504,0	441,9	385,3	335,7	257,0	200,7	160,2	130,4	108,1	152,4	5
152,4	6	979,6	979,6	953,4	889,9	822,9	750,9	674,6	597,2	523,0	455,5	396,5	303,2	236,6	188,8	153,7	127,3	152,4	6
152,4	6,3	1027	1027	998,8	932,1	861,7	786,1	705,9	624,6	546,8	476,2	414,3	316,7	247,1	197,1	160,5	133,0	152,4	6,3
159	3	521,9	521,9	512,1	480,4	447,4	412,1	374,5	335,7	297,5	261,9	229,9	177,8	139,7	112,0	91,43	75,93	159	3
159	4	691,5	691,5	677,8	635,6	591,5	544,4	494,1	442,4	391,7	344,4	302,0	233,3	183,2	146,8	119,8	99,47	159	4
159	5	858,8	858,8	841,2	788,4	733,2	674,1	611,3	546,6	483,4	424,6	372,0	287,1	225,3	180,3	147,2	122,2	159	5
159	6	1024	1024	1002	938,7	872,5	801,4	725,9	648,3	572,6	502,4	439,8	339,0	265,8	212,7	173,5	144,0	159	6
159	6,3	1073	1073	1050	983,4	913,7	839,1	759,8	678,3	598,9	525,3	459,8	354,2	277,7	222,2	181,3	150,4	159	6,3

d mm	t mm	$N_{b.Rd}$ (kN), L_{cr} (m): 0	0,5	1	1,5	2	2,5	3	3,5	4	4,5	5	6	7	8	9	10	d mm	t mm
168,3	3	553,1	553,1	546,3	514,8	482,1	447,4	410,4	371,9	333,2	296,2	262,1	205,2	162,4	130,8	107,1	89,14	168,3	3
168,3	3,2	589,2	589,2	582,0	548,3	513,5	476,4	437,0	395,9	354,6	315,2	278,9	218,2	172,7	139,0	113,9	94,77	168,3	3,2
168,3	4	733,0	733,0	723,5	681,5	637,9	591,5	542,1	490,7	439,2	390,0	344,9	269,6	213,2	171,6	140,5	116,9	168,3	4
168,3	4,5	822,1	822,1	811,2	763,9	714,8	662,6	607,0	549,2	491,3	436,0	385,4	301,1	238,1	191,5	156,8	130,4	168,3	4,5
168,3	5	910,6	910,6	898,3	845,7	791,2	733,1	671,3	607,0	542,7	481,4	425,3	332,1	262,5	211,1	172,8	143,7	168,3	5
168,3	6	1086	1086	1071	1007	942,0	872,2	797,9	720,7	643,7	570,4	503,5	392,6	310,1	249,3	204,0	169,6	168,3	6
168,3	6,3	1138	1138	1122	1056	986,8	913,5	835,4	754,3	673,5	596,6	526,5	410,4	324,1	260,5	213,1	177,2	168,3	6,3
168,3	8	1430	1430	1408	1324	1236	1143	1044	940,5	838,2	741,3	653,2	508,1	400,7	321,8	263,2	218,8	168,3	8
168,3	10	1765	1765	1735	1630	1521	1404	1279	1150	1023	902,8	794,3	616,4	485,3	389,4	318,2	264,4	168,3	10
193,7	4	846,3	846,3	846,3	806,4	763,8	719,2	672,0	622,3	570,8	519,2	469,1	379,2	306,6	250,2	206,8	173,2	193,7	4
193,7	5	1052	1052	1052	1002	948,5	892,8	833,7	771,4	707,1	642,6	580,1	468,3	378,3	308,6	255,0	213,5	193,7	5
193,7	6	1256	1256	1256	1195	1131	1064	992,9	918,1	840,8	763,5	688,7	555,3	448,2	365,3	301,8	252,6	193,7	6
193,7	6,3	1317	1317	1317	1252	1185	1115	1040	961,6	880,4	799,2	720,9	581,0	468,7	382,0	315,5	264,1	193,7	6,3
193,7	8	1657	1657	1657	1574	1488	1399	1304	1204	1101	997,7	898,7	722,7	582,3	474,1	391,3	327,4	193,7	8
193,7	10	2049	2049	2047	1943	1836	1724	1605	1479	1350	1222	1099	881,6	709,1	576,7	475,6	397,8	193,7	10
193,7	12,5	2526	2526	2521	2390	2257	2116	1967	1810	1649	1489	1336	1069	857,7	696,6	574,0	479,8	193,7	12,5
219,1	4	959,6	959,6	959,6	931,2	889,1	845,6	800,2	752,4	702,4	650,9	599,2	500,3	414,2	343,6	287,2	242,5	219,1	4
219,1	4,5	1077	1077	1077	1045	997,4	948,6	897,4	843,6	787,3	729,4	671,2	560,1	463,5	384,4	321,2	271,1	219,1	4,5
219,1	5	1194	1194	1194	1158	1105	1051	994,0	934,1	871,6	807,2	742,5	619,3	512,2	424,6	354,8	299,4	219,1	5
219,1	6	1426	1426	1426	1382	1319	1254	1185	1113	1038	960,9	883,4	735,9	608,1	503,8	420,7	355,0	219,1	6
219,1	6,3	1495	1495	1495	1449	1382	1314	1242	1167	1088	1007	925,1	770,4	636,5	527,2	440,2	371,3	219,1	6,3
219,1	8	1883	1883	1883	1823	1739	1652	1560	1464	1364	1261	1158	961,9	793,5	656,5	547,8	461,8	219,1	8
219,1	10	2332	2332	2332	2255	2149	2040	1926	1805	1679	1550	1422	1179	970,4	801,9	668,5	563,3	219,1	10
219,1	12,5	2880	2880	2880	2780	2648	2512	2368	2217	2060	1898	1738	1437	1180	973,6	810,8	682,7	219,1	12,5
244,5	6	1596	1596	1596	1569	1506	1442	1376	1306	1234	1158	1081	927,1	784,7	661,6	559,5	476,2	244,5	6
244,5	8	2110	2110	2110	2073	1989	1903	1815	1722	1625	1524	1421	1216	1027	865,0	730,8	621,6	244,5	8
244,5	10	2615	2615	2615	2567	2462	2355	2244	2127	2006	1880	1751	1495	1261	1060	894,8	760,5	244,5	10
244,5	12,5	3234	3234	3234	3170	3039	2905	2766	2620	2468	2310	2148	1830	1540	1293	1090	925,5	244,5	12,5
273	4	951,9	951,9	951,9	951,9	924,3	893,1	861,4	828,7	794,9	759,7	723,3	647,5	571,2	498,5	433,0	376,1	273	4
273	5	1494	1494	1494	1489	1437	1384	1330	1274	1215	1155	1092	962,7	836,1	719,9	618,4	532,6	273	5
273	6	1787	1787	1787	1779	1717	1653	1588	1521	1451	1378	1302	1148	995,9	856,8	735,7	633,3	273	6
273	6,3	1874	1874	1874	1866	1800	1733	1665	1595	1521	1444	1365	1203	1043	897,4	770,4	663,1	273	6,3
273	8	2364	2364	2364	2353	2269	2185	2098	2008	1914	1817	1716	1510	1308	1124	963,8	829,0	273	8
273	10	2933	2933	2933	2917	2812	2706	2598	2485	2368	2246	2120	1862	1610	1382	1184	1017	273	10
273	12,5	3632	3632	3632	3608	3477	3345	3209	3068	2921	2768	2610	2287	1974	1691	1447	1242	273	12,5
323,9	4	1099	1099	1099	1099	1091	1062	1032	1002	970,8	939,0	906,1	837,4	765,2	691,8	620,2	553,0	323,9	4
323,9	5	1422	1422	1422	1422	1409	1370	1331	1290	1249	1207	1163	1072	975,7	878,7	784,8	697,4	323,9	5
323,9	6	2127	2127	2127	2127	2092	2029	1965	1900	1834	1765	1693	1544	1389	1235	1091	959,8	323,9	6
323,9	6,3	2232	2232	2232	2232	2194	2128	2061	1993	1923	1851	1775	1618	1456	1295	1143	1006	323,9	6,3
323,9	8	2818	2818	2818	2818	2769	2685	2601	2514	2425	2333	2237	2038	1831	1627	1435	1262	323,9	8
323,9	10	3501	3501	3501	3501	3437	3332	3226	3118	3007	2891	2772	2521	2263	2008	1770	1555	323,9	10
323,9	12,5	4341	4341	4341	4341	4258	4127	3995	3859	3720	3575	3425	3112	2789	2472	2175	1909	323,9	12,5

Table 11.2.4 Buckling resistance values for circular spirally welded hollow sections of steel grade S355J2H (f_y = 355 N/mm^2)
(Technical delivery conditions to be agreed when ordering)



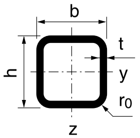
d = external diameter
t = wall thickness
L_{cr} = buckling length
$N_{b.Rd}$ = buckling resistance

The calculated resistance values are design values (see Chapter 2) based on recommended partial safety factor values **γ_{M0} = 1,0** and **γ_{M1} = 1,0 (for Class 4 circular hollow sections γ_{M1} = 1,1)** as given in Eurocode 3 (EN 1993). Partial safety factor values may differ in each country. National values must be checked from the National Annex of the relevant country.

d mm	t mm	$N_{b.Rd}$ (kN) L_{cr} (m) 0	1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	d mm	t mm
406,4	6,3	2250	2250	2250	2181	2081	1978	1870	1757	1638	1516	1394	1275	1161	1056	959,8	872,9	795,1	406,4	6,3
406,4	8	3555	3555	3555	3412	3243	3066	2880	2684	2480	2273	2069	1875	1694	1529	1381	1250	1134	406,4	8
406,4	10	4421	4421	4421	4241	4029	3808	3575	3330	3075	2816	2561	2319	2094	1889	1706	1543	1400	406,4	10
406,4	12,5	5491	5491	5491	5263	4998	4721	4430	4123	3804	3480	3162	2860	2581	2327	2101	1900	1722	406,4	12,5
457	6,3	2487	2487	2487	2450	2353	2255	2154	2048	1937	1822	1704	1585	1467	1354	1246	1146	1054	457	6,3
457	8	4006	4006	4006	3910	3741	3569	3389	3201	3003	2799	2592	2386	2188	2001	1828	1670	1526	457	8
457	10	4985	4985	4985	4863	4652	4436	4212	3975	3728	3473	3214	2958	2711	2478	2262	2065	1887	457	10
457	12	5956	5956	5956	5806	5553	5294	5024	4740	4443	4136	3826	3519	3223	2945	2687	2453	2241	457	12
457	12,5	6197	6197	6197	6040	5777	5507	5226	4930	4621	4301	3977	3657	3350	3060	2792	2548	2328	457	12,5
508	6,3	2716	2716	2716	2716	2617	2523	2426	2327	2223	2115	2003	1889	1773	1658	1545	1437	1335	508	6,3
508	8	3579	3579	3579	3579	3437	3309	3179	3043	2902	2756	2604	2449	2292	2138	1988	1844	1709	508	8
508	10	5554	5554	5554	5489	5279	5067	4847	4619	4380	4131	3876	3617	3360	3109	2871	2646	2438	508	10
508	12,5	6908	6908	6908	6823	6561	6295	6021	5735	5436	5124	4804	4481	4160	3848	3550	3271	3013	508	12,5
559	6,3	2937	2937	2937	2937	2871	2780	2687	2592	2494	2393	2288	2179	2068	1956	1843	1732	1625	559	6,3
559	8	3884	3884	3884	3884	3785	3661	3535	3406	3272	3134	2990	2842	2690	2538	2386	2237	2093	559	8
559	10	6123	6123	6123	6116	5906	5695	5480	5257	5025	4784	4535	4279	4020	3762	3509	3266	3035	559	10
559	12,5	7619	7619	7619	7606	7344	7081	6811	6532	6242	5941	5628	5308	4984	4662	4347	4044	3757	559	12,5
610	8	4181	4181	4181	4181	4122	4001	3880	3756	3628	3496	3360	3219	3074	2927	2778	2629	2481	610	8
610	10	5401	5401	5401	5401	5312	5152	4991	4827	4658	4483	4302	4115	3922	3727	3530	3334	3141	610	10
610	12,5	8330	8330	8330	8330	8127	7865	7598	7325	7043	6750	6446	6132	5812	5488	5166	4848	4541	610	12,5
610	14,2	9436	9436	9436	9436	9203	8905	8602	8292	7970	7637	7292	6936	6571	6204	5837	5477	5129	610	14,2
660	8	4463	4463	4463	4463	4463	4325	4207	4087	3964	3838	3708	3574	3436	3295	3151	3005	2859	660	8
660	10	5781	5781	5781	5781	5742	5585	5428	5269	5105	4938	4765	4586	4402	4213	4022	3828	3635	660	10
660	12,5	9027	9027	9027	9027	8895	8633	8369	8099	7822	7536	7240	6934	6620	6300	5976	5653	5334	660	12,5
660	14,2	10227	10227	10227	10227	10075	9778	9477	9171	8856	8531	8195	7847	7490	7125	6757	6391	6029	660	14,2
711	8	4742	4742	4742	4742	4742	4645	4530	4413	4295	4174	4050	3922	3791	3656	3518	3377	3235	711	8
711	10	6160	6160	6160	6160	6160	6016	5862	5707	5549	5388	5222	5051	4875	4694	4509	4321	4131	711	10
711	12,5	9738	9738	9738	9738	9678	9416	9153	8887	8614	8334	8045	7746	7439	7124	6803	6480	6156	711	12,5
711	14,2	11035	11035	11035	11035	10965	10667	10369	10066	9756	9437	9109	8769	8420	8062	7698	7330	6962	711	14,2

d mm	t mm	$N_{b.Rd}$ (kN) L_{cr} (m) 0	1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	d mm	t mm
762	8	5013	5013	5013	5013	5013	4956	4843	4729	4615	4498	4379	4257	4131	4003	3871	3736	3598	762	8
762	10	6529	6529	6529	6529	6529	6437	6286	6134	5981	5824	5664	5500	5332	5158	4981	4799	4615	762	10
762	12,5	8434	8434	8434	8434	8434	8294	8095	7894	7691	7484	7272	7054	6830	6600	6364	6123	5878	762	12,5
762	14,2	11843	11843	11843	11843	11843	11557	11259	10958	10652	10339	10017	9685	9344	8993	8634	8270	7902	762	14,2
813	8	5275	5275	5275	5275	5275	5275	5146	5036	4924	4811	4696	4579	4459	4336	4209	4080	3948	813	8
813	10	6890	6890	6890	6890	6890	6848	6699	6551	6401	6249	6094	5936	5774	5608	5437	5263	5085	813	10
813	12,5	8920	8920	8920	8920	8920	8844	8647	8451	8252	8050	7844	7634	7418	7196	6969	6736	6499	813	12,5
813	14,2	12650	12650	12650	12650	12650	12447	12149	11850	11546	11237	10921	10596	10261	9918	9566	9207	8842	813	14,2
813	16	14222	14222	14222	14222	14222	13989	13654	13316	12975	12626	12269	11903	11526	11139	10742	10337	9926	813	16
914	10	7579	7579	7579	7579	7579	7579	7488	7345	7202	7057	6911	6762	6611	6456	6298	6137	5971	914	10
914	12,5	9855	9855	9855	9855	9855	9855	9710	9520	9328	9135	8940	8741	8538	8331	8119	7902	7681	914	12,5
914	14,2	11409	11409	11409	11409	11409	11409	11225	11001	10777	10551	10321	10088	9849	9606	9356	9101	8840	914	14,2
914	16	16024	16024	16024	16024	16024	15974	15639	15303	14965	14623	14276	13920	13557	13184	12802	12411	12012	914	16
1016	10	8241	8241	8241	8241	8241	8241	8241	8109	7970	7832	7692	7551	7408	7263	7115	6965	6811	1016	10
1016	12,5	10763	10763	10763	10763	10763	10763	10763	10558	10372	10186	9999	9809	9617	9422	9222	9019	8812	1016	12,5
1016	14,2	12487	12487	12487	12487	12487	12487	12487	12228	12011	11792	11571	11348	11122	10891	10656	10416	10172	1016	14,2
1016	16	14316	14316	14316	14316	14316	14316	14316	14000	13747	13493	13237	12978	12715	12447	12174	11895	11610	1016	16
1219	10	9461	9461	9461	9461	9461	9461	9461	9461	9388	9259	9130	9000	8870	8739	8606	8472	8337	1219	10
1219	12,5	12469	12469	12469	12469	12469	12469	12469	12469	12331	12156	11980	11804	11627	11449	11268	11085	10900	1219	12,5
1219	14,2	14526	14526	14526	14526	14526	14526	14526	14526	14342	14134	13927	13719	13509	13298	13084	12867	12647	1219	14,2
1219	16	16712	16712	16712	16712	16712	16712	16712	16712	16476	16234	15993	15750	15505	15258	15009	14755	14498	1219	16

Table 11.2.5 Buckling resistance values for square hollow sections of steel grade S420MH (f_y = 420 N/mm^2)



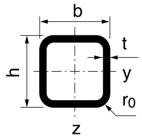
h = depth
b = width
t = wall thickness
L_{cr} = buckling length
$N_{b.Rd}$ = buckling resistance

The calculated resistance values are design values (see Chapter 2) based on recommended partial safety factor values **γ_{M0} = 1,0** and **γ_{M1} = 1,0 (for Class 4 circular hollow sections γ_{M1} = 1,1)** as given in Eurocode 3 (EN 1993). Partial safety factor values may differ in each country. National values must be checked from the National Annex of the relevant country.

h mm	b mm	t mm	$N_{b.Rd}$ (kN) L_{cr} (m) 0	0,5	1	1,5	2	2,5	3	3,5	4	4,5	5	6	7	8	9	10	h mm	b mm	t mm
25	25	2	72,95	49,67	22,02	11,12	6,61	4,36											25	25	2
25	25	2,5	87,74	58,57	25,37	12,75	7,56	4,98											25	25	2,5
25	25	3	101,1	66,08	27,96	13,97	8,27	5,45											25	25	3
30	30	2	89,75	68,85	36,40	19,36	11,70	7,79	5,54										30	30	2
30	30	2,5	108,7	82,49	42,77	22,59	13,62	9,06	6,45										30	30	2,5
30	30	3	126,3	94,71	48,13	25,25	15,19	10,09	7,18										30	30	3
40	40	2	123,4	106,5	72,03	43,58	27,58	18,74	13,50	10,17	7,93								40	40	2
40	40	2,5	150,7	129,5	86,66	52,01	32,81	22,26	16,03	12,07	9,41								40	40	2,5
40	40	3	176,7	151,1	99,98	59,51	37,42	25,36	18,24	13,73	10,70								40	40	3
40	40	4	224,6	190,0	122,7	71,84	44,89	30,34	21,79	16,38	12,76								40	40	4
50	50	2	157,0	143,7	110,7	76,22	51,28	35,83	26,20	19,91	15,62	12,57							50	50	2
50	50	2,5	192,7	175,9	134,8	92,19	61,78	43,09	31,47	23,91	18,75	15,08							50	50	2,5
50	50	3	227,1	206,8	157,5	107,0	71,40	49,71	36,27	27,54	21,58	17,36							50	50	3
50	50	4	291,8	264,1	198,7	133,0	88,07	61,09	44,49	33,74	26,43	21,24							50	50	4
50	50	5	351,0	315,7	234,3	154,4	101,5	70,15	50,99	38,62	30,23	24,29							50	50	5
60	60	2	190,6	180,6	149,1	113,5	81,81	59,21	44,11	33,90	26,78	21,66	17,86						60	60	2
60	60	2,5	234,7	222,1	182,8	138,4	99,40	71,79	53,41	41,01	32,39	26,19	21,59						60	60	2,5
60	60	3	277,5	262,2	215,1	162,1	115,9	83,51	62,05	47,62	37,59	30,38	25,04						60	60	3
60	60	4	359,0	337,9	275,4	205,3	145,6	104,5	77,43	59,34	46,79	37,80	31,14						60	60	4
60	60	5	435,0	407,9	330,0	243,3	171,1	122,2	90,37	69,16	54,49	43,99	36,23						60	60	5
70	70	2,5	276,7	268,3	230,2	186,8	143,1	107,6	81,92	63,75	50,76	41,28	34,18	24,50					70	70	2,5
70	70	3	327,9	317,6	271,9	219,9	167,9	126,0	95,73	74,43	59,24	48,15	39,86	28,56					70	70	3
70	70	4	426,2	411,7	351,1	282,0	213,8	159,6	120,9	93,85	74,62	60,61	50,14	35,90					70	70	4
70	70	5	519,0	500,0	424,6	338,5	254,7	189,2	142,9	110,8	87,97	71,40	59,04	42,24					70	70	5
80	80	2,5	318,7	314,5	277,1	235,2	190,3	149,2	116,5	92,11	74,09	60,66	50,46	36,39	27,43				80	80	2,5
80	80	3	378,3	373,0	328,2	278,0	224,2	175,4	136,8	108,0	86,83	71,05	59,09	42,60	32,11				80	80	3
80	80	4	493,4	485,5	426,0	359,2	288,1	224,3	174,3	137,4	110,3	90,19	74,97	54,01	40,68				80	80	4
80	80	5	603,0	592,2	518,2	434,8	346,7	268,5	208,0	163,7	131,2	107,2	89,06	64,11	48,27				80	80	5
80	80	6	707,0	692,9	604,5	504,8	399,9	308,2	238,0	186,9	149,7	122,2	101,4	72,96	54,91				80	80	6

h mm	b mm	t mm	$N_{b.Rd}$ (kN) L_{cr} (m) 0	0,5	1	1,5	2	2,5	3	3,5	4	4,5	5	6	7	8	9	10	h mm	b mm	t mm
90	90	2,5	334,0	334,0	302,3	266,7	227,3	187,6	152,1	123,2	100,7	83,34	69,86	50,88	38,58	30,22			90	90	2,5
90	90	3	428,7	428,4	384,0	335,5	282,2	229,5	184,1	148,0	120,3	99,24	82,99	60,25	45,61	35,68			90	90	3
90	90	4	560,6	559,4	500,5	435,9	365,1	295,7	236,3	189,6	153,9	126,8	106,0	76,87	58,15	45,48			90	90	4
90	90	5	687,0	684,4	611,1	530,6	442,4	356,7	284,1	227,4	184,4	151,8	126,7	91,86	69,46	54,29			90	90	5
90	90	6	807,8	803,6	716,1	619,6	514,2	412,7	327,6	261,7	211,9	174,2	145,4	105,3	79,56	62,17			90	90	6
100	100	2,5	347,7	347,7	324,0	293,0	258,9	222,7	187,5	156,2	130,2	109,2	92,37	68,08	51,99	40,91	32,99		100	100	2,5
100	100	3	479,1	479,1	439,7	392,5	340,5	286,4	236,1	193,6	159,5	132,7	111,7	81,72	62,15	48,77	39,26		100	100	3
100	100	4	627,8	627,8	574,6	511,9	442,6	371,0	304,9	249,4	205,1	170,5	143,3	104,8	79,65	62,48	50,28		100	100	4
100	100	5	771,0	771,0	703,8	625,5	539,1	450,2	368,7	300,9	247,1	205,2	172,4	125,9	95,63	74,99	60,33		100	100	5
100	100	6	908,6	908,6	827,2	733,5	630,0	524,0	427,8	348,3	285,6	236,8	198,8	145,1	110,1	86,33	69,44		100	100	6
100	100	7,1	1035	1035	938,2	828,5	707,4	584,6	474,7	385,0	314,9	260,6	218,5	159,2	120,8	94,60	76,05		100	100	7,1
100	100	8	1144	1144	1034	910,4	774,3	637,2	515,7	417,3	340,7	281,7	236,1	171,8	130,2	102,0	81,97		100	100	8
100	100	10	1368	1368	1227	1073	904,5	737,2	592,0	476,5	387,7	319,9	267,6	194,3	147,1	115,1	92,46		100	100	10
110	110	2,5	358,7	358,7	341,8	314,4	284,9	253,0	220,2	188,8	160,9	137,1	117,3	87,76	67,61	53,50	43,31	35,75	110	110	2,5
110	110	3	484,6	484,6	456,6	416,6	372,9	326,0	279,0	235,6	198,4	167,5	142,5	105,7	81,04	63,93	51,65	42,56	110	110	3
110	110	4	695,0	695,0	648,6	587,2	519,9	448,1	378,0	315,5	263,2	220,9	187,0	138,0	105,4	82,96	66,92	55,09	110	110	4
110	110	5	855,0	855,0	796,2	719,7	635,6	546,2	459,4	382,5	318,6	267,1	225,9	166,4	127,1	99,99	80,63	66,36	110	110	5
110	110	6	1009	1009	938,0	846,4	745,6	638,8	535,7	444,9	369,9	309,7	261,8	192,7	147,0	115,6	93,21	76,70	110	110	6
120	120	3	500,6	500,6	480,8	444,7	406,0	364,3	320,7	278,1	239,2	205,2	176,6	133,0	102,9	81,61	66,18	54,70	120	120	3
120	120	4	762,2	762,2	722,5	662,1	596,5	525,8	453,9	386,4	327,2	277,6	236,9	176,4	135,6	107,1	86,63	71,45	120	120	4
120	120	5	939,0	939,0	888,6	813,2	731,3	643,1	553,8	470,3	397,6	336,9	287,2	213,6	164,1	129,6	104,7	86,36	120	120	5
120	120	5,6	1042	1042	985,4	901,1	809,5	710,8	611,1	518,3	437,7	370,6	315,7	234,7	180,1	142,2	115,0	94,78	120	120	5,6
120	120	6	1110	1110	1049	958,6	860,4	754,8	648,3	549,3	463,5	392,2	334,0	248,2	190,5	150,3	121,5	100,2	120	120	6
120	120	7,1	1274	1274	1200	1094	979,3	855,6	731,7	617,7	519,7	438,9	373,2	276,8	212,2	167,3	135,2	111,4	120	120	7,1
120	120	8	1413	1413	1329	1210	1080	941,5	802,9	676,2	568,0	479,0	406,9	301,4	230,9	182,1	147,0	121,1	120	120	8
120	120	8,8	1532	1532	1438	1308	1166	1013	862,1	724,6	607,7	511,9	434,5	321,5	246,1	194,0	156,6	129,0	120	120	8,8
120	120	10	1704	1704	1595	1448	1286	1114	944,0	790,8	661,6	556,3	471,7	348,5	266,6	209,9	169,4	139,5	120	120	10
140	140	4	844,9	844,9	823,4	769,2	712,1	650,8	585,8	519,6	456,0	397,8	346,7	265,6	207,5	165,6	134,9	111,8	140	140	4
140	140	5	1107	1107	1073	999,2	920,8	836,3	747,1	657,4	572,6	496,5	430,6	327,7	255,0	203,1	165,1	136,7	140	140	5
140	140	5,6	1231	1231	1192	1109	1022	927,1	827,2	727,1	632,7	548,1	475,1	361,2	280,9	223,7	181,8	150,5	140	140	5,6
140	140	6	1312	1312	1270	1182	1088	986,4	879,6	772,6	671,8	581,7	503,9	382,9	297,7	237,0	192,6	159,4	140	140	6
140	140	7,1	1513	1513	1462	1358	1248	1129	1004	878,8	762,0	658,4	569,4	431,7	335,2	266,6	216,6	179,2	140	140	7,1
140	140	8	1682	1682	1623	1507	1383	1249	1108	968,6	838,5	723,4	625,0	473,2	367,1	291,8	237,0	196,0	140	140	8
140	140	8,8	1828	1828	1762	1635	1498	1351	1197	1044	902,7	777,9	671,4	507,7	393,6	312,7	253,9	210,0	140	140	8,8
140	140	10	2040	2040	1963	1819	1664	1497	1323	1151	992,5	853,6	735,8	555,3	430,1	341,4	277,1	229,1	140	140	10

Table 11.2.5 Buckling resistance values for square hollow sections of steel grade S420MH (f_y = 420 N/mm^2) *continued*



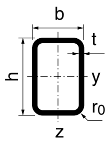
h = depth
b = width
t = wall thickness
L_{cr} = buckling length
$N_{b.Rd}$ = buckling resistance

The calculated resistance values are design values (see Chapter 2) based on recommended partial safety factor values **γ_{M0} = 1,0** and **γ_{M1} = 1,0 (for Class 4 circular hollow sections γ_{M1} = 1,1)** as given in Eurocode 3 (EN 1993). Partial safety factor values may differ in each country. National values must be checked from the National Annex of the relevant country.

h mm	b mm	t mm	$N_{b.Rd}$ (kN), L_{cr} (m): 0	0,5	1	1,5	2	2,5	3	3,5	4	4,5	5	6	7	8	9	10	h mm	b mm	t mm
150	150	4	869,1	869,1	856,8	806,3	753,9	698,1	638,7	576,9	515,3	456,6	403,1	314,4	248,3	199,6	163,3	135,8	150	150	4
150	150	5	1191	1191	1166	1092	1015	932,0	844,1	753,8	665,7	584,1	511,3	394,0	309,0	247,2	201,7	167,4	150	150	5
150	150	6	1413	1413	1381	1293	1200	1101	995,9	887,9	783,0	686,0	599,8	461,5	361,6	289,1	235,8	195,6	150	150	6
150	150	7,1	1632	1632	1593	1489	1381	1264	1141	1014	892,2	780,0	680,8	522,6	408,8	326,7	266,2	220,8	150	150	7,1
150	150	8	1816	1816	1771	1655	1533	1402	1263	1121	984,3	859,3	749,3	574,3	448,9	358,4	292,0	242,1	150	150	8
150	150	8,8	1976	1976	1925	1797	1663	1519	1367	1211	1062	926,2	806,8	617,6	482,3	385,0	313,5	259,9	150	150	8,8
150	150	10	2208	2208	2147	2003	1852	1688	1515	1340	1172	1020	887,5	678,0	528,8	421,8	343,3	284,5	150	150	10
150	150	12,5	2606	2606	2522	2346	2158	1956	1743	1530	1330	1151	996,8	757,0	588,4	468,3	380,6	315,0	150	150	12,5
160	160	4	890,0	890,0	886,1	838,9	790,5	739,5	685,3	628,4	570,2	513,0	458,8	364,9	291,8	236,5	194,5	162,4	160	160	4
160	160	5	1275	1275	1258	1184	1108	1027	940,8	850,9	761,0	675,3	596,7	466,1	368,5	296,4	242,6	201,8	160	160	5
160	160	6	1513	1513	1492	1404	1313	1215	1112	1004	896,9	794,8	701,6	547,1	432,1	347,3	284,2	236,4	160	160	6
160	160	7,1	1751	1751	1724	1621	1514	1399	1278	1151	1026	907,3	799,5	621,9	490,4	393,8	322,0	267,7	160	160	7,1
160	160	8	1951	1951	1918	1803	1682	1554	1417	1275	1134	1002	881,8	684,9	539,5	433,0	354,0	294,2	160	160	8
160	160	8,8	2124	2124	2087	1960	1828	1686	1536	1380	1227	1082	951,6	738,0	580,9	466,0	380,7	316,3	160	160	8,8
160	160	10	2376	2376	2332	2188	2038	1878	1707	1531	1358	1196	1050	812,6	638,8	512,0	418,1	347,3	160	160	10
160	160	12,5	2816	2816	2752	2576	2391	2193	1983	1767	1557	1364	1192	917,2	718,4	574,4	468,4	388,5	160	160	12,5
180	180	5	1336	1336	1336	1276	1209	1140	1067	989,4	909,3	828,7	750,2	608,4	492,9	402,9	333,4	279,4	180	180	5
180	180	6	1715	1715	1714	1626	1536	1442	1342	1237	1129	1021	918,2	736,2	592,0	481,4	397,0	332,0	180	180	6
180	180	7,1	1990	1990	1986	1883	1778	1667	1549	1426	1299	1173	1053	842,2	676,0	549,1	452,5	378,2	180	180	7,1
180	180	8	2219	2219	2214	2098	1980	1855	1723	1584	1441	1300	1166	930,9	746,4	605,9	499,0	417,0	180	180	8
180	180	8,8	2419	2419	2412	2285	2155	2018	1873	1720	1563	1409	1262	1007	806,3	654,1	538,5	449,8	180	180	8,8
180	180	10	2712	2712	2700	2557	2410	2255	2090	1917	1739	1565	1400	1114	891,1	722,2	594,2	496,1	180	180	10
180	180	12,5	3236	3236	3213	3036	2855	2663	2459	2246	2028	1817	1619	1280	1019	823,9	676,6	564,2	180	180	12,5
200	200	5	1391	1391	1391	1355	1296	1235	1172	1105	1036	963,7	890,9	750,0	624,9	520,7	436,7	369,5	200	200	5
200	200	6	1917	1917	1917	1847	1759	1667	1570	1468	1362	1253	1146	944,5	774,4	638,0	530,8	446,7	200	200	6
200	200	7,1	2228	2228	2228	2145	2041	1933	1819	1699	1574	1446	1320	1086	888,7	731,3	607,9	511,2	200	200	7,1
200	200	8	2488	2488	2488	2393	2276	2154	2026	1891	1751	1608	1466	1204	984,1	809,2	672,3	565,1	200	200	8
200	200	8,8	2715	2715	2715	2609	2481	2347	2206	2058	1904	1746	1591	1305	1066	875,7	727,2	611,1	200	200	8,8
200	200	10	3048	3048	3048	2925	2780	2628	2469	2300	2125	1947	1772	1450	1183	970,7	805,5	676,6	200	200	10
200	200	12,5	3656	3656	3656	3496	3317	3130	2932	2724	2508	2289	2076	1688	1371	1122	929,0	779,2	200	200	12,5

h	b	t	$N_{b.Rd}$ (kN)																h	b	t
mm	mm	mm	L_{cr} (m)																mm	mm	mm
			0	0,5	1	1,5	2	2,5	3	3,5	4	4,5	5	6	7	8	9	10			
220	220	6	1938	1938	1938	1904	1827	1748	1666	1581	1492	1399	1304	1116	942,5	793,4	670,2	570,0	220	220	6
220	220	7,1	2467	2467	2467	2407	2303	2197	2086	1970	1848	1722	1594	1345	1124	937,8	787,3	666,7	220	220	7,1
220	220	8	2757	2757	2757	2688	2572	2452	2327	2196	2059	1917	1774	1495	1247	1040	872,7	738,7	220	220	8
220	220	8,8	3011	3011	3011	2934	2806	2674	2537	2393	2243	2087	1929	1624	1353	1128	945,8	800,3	220	220	8,8
220	220	10	3384	3384	3384	3294	3149	3000	2845	2681	2510	2334	2155	1811	1507	1254	1051	888,8	220	220	10
220	220	12,5	4076	4076	4076	3957	3778	3594	3402	3199	2987	2769	2550	2131	1765	1465	1224	1034	220	220	12,5
250	250	6	2030	2030	2030	2030	1966	1898	1828	1757	1682	1605	1525	1359	1193	1037	897,6	777,4	250	250	6
250	250	7,1	2651	2651	2651	2637	2543	2448	2350	2249	2144	2034	1921	1688	1462	1255	1076	925,4	250	250	7,1
250	250	8	3160	3160	3160	3131	3015	2897	2776	2649	2518	2380	2239	1951	1676	1430	1219	1044	250	250	8
250	250	8,8	3454	3454	3454	3421	3293	3164	3030	2892	2747	2596	2441	2125	1824	1554	1325	1134	250	250	8,8
250	250	10	3888	3888	3888	3848	3703	3556	3405	3247	3083	2912	2735	2378	2038	1735	1478	1264	250	250	10
250	250	12,5	4706	4706	4706	4648	4469	4288	4101	3906	3702	3490	3271	2832	2417	2051	1742	1488	250	250	12,5
260	260	6	2056	2056	2056	2056	2005	1940	1874	1806	1736	1663	1588	1430	1270	1115	973,9	849,3	260	260	6
260	260	7,1	2694	2694	2694	2694	2605	2513	2420	2324	2224	2120	2013	1790	1568	1360	1175	1017	260	260	7,1
260	260	8	3295	3295	3295	3279	3163	3045	2924	2799	2669	2534	2393	2106	1825	1569	1346	1158	260	260	8
260	260	8,8	3602	3602	3602	3584	3456	3326	3194	3057	2914	2765	2611	2295	1987	1707	1463	1258	260	260	8,8
260	260	10	4056	4056	4056	4032	3887	3741	3591	3435	3273	3103	2929	2571	2223	1907	1634	1404	260	260	10
260	260	12,5	4916	4916	4916	4878	4699	4519	4333	4140	3938	3728	3512	3071	2645	2262	1933	1658	260	260	12,5
300	300	6	2141	2141	2141	2141	2137	2081	2026	1970	1913	1854	1794	1667	1534	1398	1263	1133	300	300	6
300	300	7,1	2838	2838	2838	2838	2812	2733	2654	2574	2492	2407	2320	2136	1945	1751	1563	1389	300	300	7,1
300	300	8	3453	3453	3453	3453	3404	3304	3203	3100	2994	2885	2772	2536	2290	2044	1811	1599	300	300	8
300	300	8,8	4193	4193	4193	4193	4106	3977	3847	3714	3576	3434	3286	2977	2660	2351	2063	1807	300	300	8,8
300	300	10	4728	4728	4728	4728	4626	4480	4332	4181	4025	3863	3696	3346	2986	2636	2312	2023	300	300	10
300	300	12,5	5756	5756	5756	5756	5621	5441	5258	5071	4877	4677	4469	4035	3591	3161	2765	2415	300	300	12,5

Table 11.2.6 Buckling resistance values for rectangular hollow sections of steel grade S420MH (f_y = 420 N/mm²)



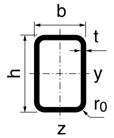
h = depth
b = width
t = wall thickness
L_{cr} = buckling length
$N_{b.Rd}$ = buckling resistance

The calculated resistance values are design values (see Chapter 2) based on recommended partial safety factor values γ_{M0} **= 1,0** and γ_{M1} **= 1,0 (for Class 4 circular hollow sections** γ_{M1} **= 1,1)** as given in Eurocode 3 (EN 1993). Partial safety factor values may differ in each country. National values must be checked from the National Annex of the relevant country.

h mm	b mm	t mm	axis	$N_{b.Rd}$ (kN) L_{cr} (m) 0	0,5	1	1,5	2	2,5	3	3,5	4	4,5	5	6	7	8	9	10	h mm	b mm	t mm
40	20	2	y-y	89,75	74,80	46,71	26,79	16,62	11,19	8,02	6,03									40	20	2
			z-z	89,75	53,95	21,13	10,39	6,11	4,02	2,84	2,11											
40	20	2,5	y-y	108,7	89,91	55,19	31,35	19,38	13,03	9,33	7,01									40	20	2,5
			z-z	108,7	63,65	24,41	11,96	7,02	4,61	3,25	2,42											
40	20	3	y-y	126,3	103,6	62,41	35,10	21,62	14,52	10,39	7,79									40	20	3
			z-z	126,3	71,87	27,00	13,17	7,73	5,07	3,58	2,66											
40	30	2	y-y	106,6	90,79	59,58	35,27	22,13	14,98	10,77	8,10	6,31								40	30	2
			z-z	106,6	83,35	45,68	24,62	14,95	9,97	7,11	5,32	4,13										
40	30	2,5	y-y	129,7	109,9	71,20	41,78	26,13	17,67	12,69	9,54	7,43								40	30	2,5
			z-z	129,7	100,5	54,15	28,99	17,56	11,70	8,34	6,24	4,84										
40	30	3	y-y	151,5	127,6	81,55	47,43	29,57	19,96	14,33	10,77	8,38								40	30	3
			z-z	151,5	116,3	61,52	32,72	19,78	13,16	9,37	7,01	5,44										
50	30	2	y-y	123,4	111,0	82,45	54,40	35,77	24,73	17,98	13,62	10,66	8,56							50	30	2
			z-z	123,4	97,70	54,85	29,85	18,18	12,15	8,67	6,49	5,04	4,02									
50	30	2,5	y-y	150,7	135,2	99,59	65,14	42,65	29,43	21,37	16,18	12,66	10,17							50	30	2,5
			z-z	150,7	118,4	65,40	35,36	21,49	14,34	10,22	7,65	5,94	4,74									
50	30	3	y-y	176,7	157,9	115,3	74,77	48,74	33,57	24,35	18,43	14,41	11,57							50	30	3
			z-z	176,7	137,6	74,74	40,15	24,34	16,23	11,56	8,65	6,71	5,36									
50	30	4	y-y	224,6	199,0	142,7	90,78	58,68	40,25	29,15	22,02	17,21	13,81							50	30	4
			z-z	224,6	171,5	89,99	47,72	28,82	19,17	13,65	10,20	7,91	6,31									
60	40	2	y-y	157,0	147,3	119,4	88,25	62,19	44,46	32,90	25,19	19,85	16,02	13,20						60	40	2
			z-z	157,0	137,6	96,31	59,94	38,38	26,21	18,94	14,29	11,15	8,95	7,33								
60	40	2,5	y-y	192,7	180,5	145,7	107,0	75,09	53,56	39,58	30,28	23,85	19,25	15,85						60	40	2,5
			z-z	192,7	168,3	116,8	72,14	46,03	31,40	22,67	17,10	13,34	10,70	8,76								
60	40	3	y-y	227,1	212,3	170,6	124,5	86,96	61,88	45,68	34,91	27,49	22,18	18,26						60	40	3
			z-z	227,1	197,6	135,8	83,27	52,97	36,08	26,03	19,62	15,31	12,27	10,05								
60	40	4	y-y	291,8	271,4	216,1	155,7	107,7	76,29	56,17	42,87	33,72	27,19	22,38						60	40	4
			z-z	291,8	251,7	169,7	102,5	64,78	44,01	31,69	23,87	18,61	14,91	12,21								
60	40	5	y-y	351,0	324,8	256,0	181,8	124,6	87,82	64,51	49,17	38,64	31,13	25,61						60	40	5
			z-z	351,0	299,9	198,0	117,7	74,01	50,14	36,06	27,14	21,14	16,93	13,86								

h mm	b mm	t mm	axis	$N_{b.Rd}$ (kN) L_{cr} (m) 0	0,5	1	1,5	2	2,5	3	3,5	4	4,5	5	6	7	8	9	10	h mm	b mm	t mm
70	50	2	y-y	184,1	183,8	156,3	124,9	94,23	70,11	53,02	41,11	32,66	26,52	21,93	15,70					70	50	2
			z-z	184,1	170,6	134,8	96,00	65,93	46,53	34,20	26,07	20,49	16,51	13,58	9,65							
70	50	2,5	y-y	234,7	226,0	191,8	152,7	114,7	85,13	64,29	49,80	39,55	32,10	26,54	18,98					70	50	2,5
			z-z	234,7	216,3	168,9	118,5	80,59	56,60	41,49	31,59	24,80	19,97	16,42	11,66							
70	50	3	y-y	277,5	266,9	225,9	179,1	134,0	99,18	74,79	57,88	45,94	37,27	30,81	22,03					70	50	3
			z-z	277,5	255,1	198,3	138,2	93,65	65,66	48,09	36,59	28,72	23,12	19,01	13,49							
70	50	4	y-y	359,0	344,2	289,8	227,8	169,0	124,4	93,55	72,28	57,30	46,45	38,38	27,43					70	50	4
			z-z	359,0	328,4	252,8	173,9	116,9	81,67	59,70	45,37	35,59	28,64	23,53	16,69							
70	50	5	y-y	435,0	415,6	348,0	271,0	199,3	146,0	109,4	84,41	66,85	54,15	44,71	31,93					70	50	5
			z-z	435,0	395,9	301,4	204,6	136,5	95,03	69,33	52,63	41,26	33,18	27,25	19,32							
80	40	2	y-y	172,9	169,7	148,4	124,4	98,97	76,56	59,26	46,60	37,35	30,50	25,34	18,23	13,73				80	40	2
			z-z	172,9	154,7	113,4	73,80	48,20	33,22	24,11	18,25	14,27	11,46	9,40	6,66	4,96						
80	40	2,5	y-y	234,7	228,7	197,7	162,6	126,4	96,10	73,59	57,48	45,88	37,37	30,97	22,23	16,71				80	40	2,5
			z-z	234,7	206,8	146,4	92,03	59,17	40,50	29,28	22,11	17,27	13,85	11,35	8,03	5,97						
80	40	3	y-y	277,5	270,0	233,0	190,9	147,9	112,0	85,63	66,82	53,30	43,39	35,95	25,80	19,38				80	40	3
			z-z	277,5	243,6	171,0	106,7	68,41	46,76	33,78	25,50	19,91	15,96	13,08	9,25	6,88						
80	40	4	y-y	359,0	348,3	299,2	243,2	186,8	140,7	107,2	83,44	66,47	54,06	44,77	32,10	24,10				80	40	4
			z-z	359,0	312,7	215,8	132,7	84,52	57,61	41,56	31,34	24,45	19,60	16,06	11,34	8,44						
80	40	5	y-y	435,0	420,7	359,7	290,0	220,7	165,2	125,4	97,42	77,51	62,98	52,12	37,33	28,02				80	40	5
			z-z	435,0	375,8	254,6	154,2	97,65	66,37	47,82	36,02	28,09	22,51	18,44	13,02	9,68						
80	60	2,5	y-y	276,7	271,7	237,7	199,4	158,8	123,0	95,25	74,91	60,06	49,06	40,75	29,33	22,08	17,21			80	60	2,5
			z-z	276,7	263,5	219,3	169,1	123,2	89,78	67,11	51,67	40,88	33,09	27,31	19,49	14,59	11,33					
80	60	3	y-y	327,9	321,7	281,0	235,0	186,6	144,1	111,4	87,56	70,16	57,28	47,57	34,22	25,76	20,07			80	60	3
			z-z	327,9	311,8	258,8	198,6	144,2	104,8	78,27	60,23	47,62	38,54	31,80	22,68	16,98	13,18					
80	60	4	y-y	426,2	417,1	363,2	302,1	238,3	183,0	141,1	110,6	88,52	72,21	59,93	43,08	32,41	25,25			80	60	4
			z-z	426,2	404,0	333,4	253,6	182,7	132,2	98,48	75,67	59,78	48,35	39,87	28,43	21,27	16,51					
80	60	5	y-y	519,0	506,8	439,7	363,6	284,8	217,6	167,1	130,8	104,5	85,20	70,66	50,76	38,17	29,72			80	60	5
			z-z	519,0	490,2	402,2	303,0	216,6	156,1	116,0	89,00	70,25	56,77	46,80	33,35	24,94	19,35					
90	50	2,5	y-y	263,4	261,7	232,8	200,9	166,1	132,8	105,1	83,83	67,81	55,71	46,47	33,63	25,40	19,85	15,92		90	50	2,5
			z-z	263,4	245,2	195,6	141,2	97,89	69,39	51,12	39,03	30,70	24,76	20,38	14,48	10,82	8,38	6,69				
90	50	3	y-y	327,9	324,6	287,3	245,8	200,9	158,9	124,9	99,13	79,93	65,54	54,59	39,43	29,75	23,22	18,62		90	50	3
			z-z	327,9	303,3	238,6	169,1	115,7	81,51	59,85	45,60	35,83	28,87	23,74	16,86	12,59	9,75	7,77				
90	50	4	y-y	426,2	421,1	371,6	316,4	257,0	202,2	158,3	125,3	100,9	82,65	68,79	49,64	37,43	29,21	23,42		90	50	4
			z-z	426,2	392,4	306,0	214,2	145,5	102,1	74,84	56,96	44,72	36,01	29,61	21,02	15,68	12,14	9,68				
90	50	5	y-y	519,0	511,6	450,2	381,4	307,7	240,7	187,7	148,3	119,2	97,53	81,12	58,48	44,08	34,38	27,55		90	50	5
			z-z	519,0	475,6	367,4	253,9	171,2	119,7	87,59	66,59	52,25	42,05	34,56	24,52	18,28	14,16	11,28				

Table 11.2.6 Buckling resistance values for rectangular hollow sections of steel grade S420MH (f_y = 420 N/mm^2) *continued*



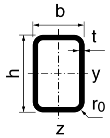
h = depth
b = width
t = wall thickness
L_{cr} = buckling length
$N_{b.Rd}$ = buckling resistance

The calculated resistance values are design values (see Chapter 2) based on recommended partial safety factor values γ_{M0} **= 1,0** and γ_{M1} **= 1,0 (for Class 4 circular hollow sections** γ_{M1} **= 1,1)** as given in Eurocode 3 (EN 1993). Partial safety factor values may differ in each country. National values must be checked from the National Annex of the relevant country.

h mm	b mm	t mm	axis	$N_{b.Rd}$ (kN) L_{cr} (m) 0	0,5	1	1,5	2	2,5	3	3,5	4	4,5	5	6	7	8	9	10	h mm	b mm	t mm
100	40	2,5	y-y	249,2	249,2	225,0	198,1	168,3	138,4	112,0	90,56	73,93	61,12	51,20	37,26	28,24	22,11	17,77	14,59	100	40	2,5
			z-z	249,2	223,6	165,0	108,1	70,80	48,87	35,50	26,88	21,03	16,89	13,86	9,81	7,31	5,65	4,50	3,67			
100	40	3	y-y	327,9	326,8	291,8	253,5	211,4	170,5	135,9	108,8	88,22	72,61	60,64	43,96	33,24	25,98	20,86	17,11	100	40	3
			z-z	327,9	289,5	206,0	130,1	83,80	57,40	41,53	31,36	24,50	19,66	16,11	11,39	8,48	6,55	5,22	4,25			
100	40	4	y-y	426,2	423,9	377,6	326,5	270,7	217,1	172,2	137,5	111,3	91,50	76,35	55,28	41,77	32,64	26,19	21,48	100	40	4
			z-z	426,2	373,6	261,5	162,8	104,2	71,17	51,41	38,80	30,28	24,29	19,90	14,07	10,46	8,09	6,44	5,24			
100	40	5	y-y	519,0	515,0	457,5	393,7	324,3	258,5	204,2	162,6	131,4	107,9	89,92	65,04	49,11	38,35	30,77	25,22	100	40	5
			z-z	519,0	451,5	310,6	190,5	121,2	82,57	59,56	44,90	35,03	28,08	23,00	16,25	12,08	9,33	7,43	6,05			
100	50	2,5	y-y	270,2	270,2	245,6	217,4	186,4	154,6	126,0	102,4	83,92	69,55	58,36	42,56	32,30	25,31	20,35	16,72	100	50	2,5
			z-z	270,2	253,1	204,4	150,4	105,5	75,31	55,67	42,59	33,55	27,08	22,30	15,87	11,86	9,19	7,34	5,99			
100	50	3	y-y	353,1	353,1	316,9	277,3	233,8	190,6	153,2	123,3	100,3	82,79	69,26	50,31	38,09	29,80	23,94	19,64	100	50	3
			z-z	353,1	327,3	258,7	184,4	126,7	89,41	65,71	50,10	39,38	31,74	26,11	18,55	13,85	10,73	8,55	6,98			
100	50	4	y-y	459,8	459,0	411,0	358,4	300,6	243,8	195,1	156,7	127,3	104,9	87,67	63,62	48,14	37,65	30,23	24,80	100	50	4
			z-z	459,8	424,3	332,5	234,2	159,7	112,3	82,39	62,75	49,29	39,70	32,64	23,18	17,30	13,40	10,68	8,71			
100	50	5	y-y	561,0	559,0	499,3	433,7	361,8	291,9	232,6	186,3	151,1	124,4	103,9	75,29	56,93	44,50	35,72	29,30	100	50	5
			z-z	561,0	515,4	400,1	278,4	188,5	132,1	96,70	73,56	57,74	46,48	38,21	27,11	20,22	15,66	12,48	10,18			
100	50	6	y-y	656,6	653,1	581,8	503,3	417,5	334,9	265,8	212,3	171,9	141,3	117,9	85,38	64,52	50,42	40,46	33,18	100	50	6
			z-z	656,6	600,4	461,6	317,2	213,1	148,8	108,8	82,66	64,83	52,17	42,86	30,40	22,67	17,55	13,99	11,41			
100	60	2,5	y-y	291,2	291,2	266,0	236,6	204,1	170,6	139,9	114,2	93,83	77,91	65,47	47,83	36,34	28,50	22,93	18,84	100	60	2,5
			z-z	291,2	280,3	237,6	189,0	141,8	105,2	79,38	61,48	48,81	39,61	32,75	23,42	17,57	13,65	10,92	8,92			
100	60	3	y-y	378,3	378,3	341,7	300,8	255,7	210,3	170,1	137,6	112,3	92,89	77,82	56,64	42,93	33,61	27,01	22,18	100	60	3
			z-z	378,3	361,2	302,1	234,7	172,3	126,0	94,40	72,79	57,63	46,67	38,53	27,51	20,61	16,00	12,78	10,45			
100	60	4	y-y	493,4	493,4	444,1	389,7	329,8	270,0	217,6	175,5	143,1	118,2	98,91	71,91	54,47	42,63	34,25	28,11	100	60	4
			z-z	493,4	469,7	390,8	301,1	219,4	159,8	119,4	91,94	72,73	58,87	48,58	34,66	25,95	20,15	16,09	13,15			
100	60	5	y-y	603,0	602,8	540,7	472,9	398,4	324,5	260,5	209,7	170,6	140,7	117,7	85,48	64,71	50,63	40,67	33,37	100	60	5
			z-z	603,0	572,3	473,6	361,7	261,6	189,7	141,4	108,7	85,94	69,52	57,35	40,90	30,61	23,76	18,97	15,50			
100	60	6	y-y	707,0	705,6	631,5	550,5	461,4	374,0	299,1	240,0	195,0	160,6	134,3	97,41	73,70	57,63	46,28	37,96	100	60	6
			z-z	707,0	668,9	550,4	416,6	299,0	215,9	160,6	123,3	97,37	78,72	64,91	46,26	34,61	26,85	21,44	17,51			

h mm	b mm	t mm	axis	$N_{b.Rd}$ (kN) L_{cr} (m) 0	0,5	1	1,5	2	2,5	3	3,5	4	4,5	5	6	7	8	9	10	h mm	b mm	t mm
100	80	2,5	y-y	332,1	332,1	305,8	273,8	238,5	201,6	166,9	137,3	113,4	94,46	79,57	58,31	44,38	34,85	28,06	23,07	100	80	2,5
			z-z	332,1	330,3	294,2	254,4	210,9	169,2	134,2	107,1	86,74	71,31	59,50	43,08	32,55	25,44	20,41	16,74			
100	80	3	y-y	428,7	428,7	390,8	347,0	298,5	248,8	203,4	165,8	136,1	112,9	94,81	69,21	52,56	41,21	33,15	27,23	100	80	3
			z-z	428,7	424,1	375,0	320,3	261,1	206,2	161,8	128,3	103,4	84,74	70,56	50,95	38,44	30,00	24,05	19,71			
100	80	4	y-y	560,6	560,6	509,6	451,2	386,8	321,1	261,7	212,7	174,3	144,4	121,2	88,40	67,09	52,58	42,28	34,72	100	80	4
			z-z	560,6	553,6	488,3	415,4	337,0	264,8	207,2	164,0	132,0	108,1	89,94	64,89	48,93	38,17	30,60	25,07			
100	80	5	y-y	687,0	687,0	622,6	549,8	469,5	388,1	315,2	255,6	209,1	173,1	145,1	105,7	80,21	62,83	50,51	41,48	100	80	5
			z-z	687,0	677,2	595,9	504,8	407,3	318,6	248,5	196,2	157,7	129,1	107,4	77,40	58,33	45,50	36,46	29,87			
100	80	6	y-y	807,8	807,8	729,8	642,7	546,7	449,9	364,1	294,6	240,6	198,9	166,7	121,3	91,96	72,01	57,88	47,51	100	80	6
			z-z	807,8	794,9	697,6	588,4	472,1	367,6	285,8	225,3	180,9	147,9	122,9	88,55	66,71	52,01	41,67	34,13			
120	40	2,5	y-y	259,2	259,2	244,2	222,6	199,2	174,0	148,8	125,6	105,7	89,21	75,86	56,26	43,12	34,01	27,47	22,64	120	40	2,5
			z-z	259,2	236,1	180,0	122,3	81,72	56,90	41,53	31,53	24,72	19,88	16,33	11,58	8,63	6,68	5,32	4,34			
120	40	3	y-y	338,7	338,7	315,9	285,8	252,8	217,8	183,5	153,1	127,6	107,1	90,63	66,82	51,05	40,17	32,40	26,67	120	40	3
			z-z	338,7	304,3	225,1	147,9	97,03	67,01	48,70	36,88	28,86	23,18	19,02	13,47	10,03	7,76	6,18	5,04			
120	40	4	y-y	493,4	493,4	453,9	406,1	353,3	298,3	246,7	202,8	167,4	139,4	117,4	86,00	65,44	51,37	41,36	34,00	120	40	4
			z-z	493,4	434,3	307,0	192,7	123,8	84,72	61,25	46,24	36,11	28,97	23,75	16,78	12,49	9,65	7,68	6,26			
120	40	5	y-y	603,0	603,0	552,8	493,1	427,2	358,9	295,6	242,1	199,4	165,8	139,5	102,0	77,58	60,87	48,99	40,26	120	40	5
			z-z	603,0	526,9	366,3	226,6	144,7	98,73	71,28	53,77	41,96	33,64	27,6	19,48	14,49	11,19	8,91	7,26			
120	40	6	y-y	707,0	707,0	645,7	574,2	495,3	413,9	339,3	277,0	227,6	189,0	158,8	116,0	88,13	69,11	55,60	45,68	120	40	6
			z-z	707,0	613,1	418,7	255,3	162,0	110,3	79,49	59,91	46,73	37,45	30,68	21,66	16,11	12,44	9,90	8,06			
120	50	2,5	y-y	280,2	280,2	265,0	242,5	217,9	191,5	164,8	139,8	118,2	100,1	85,27	63,42	48,69	38,44	31,08	25,62	120	50	2,5
			z-z	280,2	265,5	218,9	166,3	119,7	86,59	64,47	49,53	39,12	31,64	26,09	18,60	13,92	10,80	8,62	7,04			
120	50	3	y-y	363,9	363,9	341,2	310,0	275,9	239,4	203,3	170,6	142,9	120,3	102,0	75,46	57,74	45,49	36,72	30,24	120	50	3
			z-z	363,9	341,5	276,8	204,6	144,2	103,1	76,29	58,40	46,02	37,16	30,61	21,78	16,28	12,63	10,08	8,23			
120	50	4	y-y	527,0	527,0	488,1	439,2	385,3	328,5	274,1	226,9	188,1	157,2	132,7	97,51	74,33	58,42	47,08	38,72	120	50	4
			z-z	527,0	488,1	385,2	274,1	188,1	132,7	97,48	74,31	58,40	47,06	38,71	27,50	20,53	15,90	12,68	10,35			
120	50	5	y-y	645,0	645,0	595,6	534,6	467,3	396,7	329,8	272,1	225,1	187,9	158,4	116,2	88,55	69,56	56,03	46,07	120	50	5
			z-z	645,0	594,8	465,4	327,2	222,9	156,7	114,9	87,48	68,70	55,33	45,50	32,30	24,10	18,67	14,88	12,14			
120	50	6	y-y	757,4	757,4	697,2	624,2	543,6	459,4	380,4	312,8	258,3	215,2	181,3	132,8	101,1	79,39	63,93	52,55	120	50	6
			z-z	757,4	695,4	539,2	374,5	253,2	177,3	129,8	98,74	77,49	62,38	51,27	36,38	27,14	21,01	16,75	13,66			
120	60	2,5	y-y	301,2	301,2	285,9	262,2	236,5	208,8	180,5	153,9	130,5	110,8	94,63	70,55	54,24	42,87	34,68	28,60	120	60	2,5
			z-z	301,2	292,5	251,7	205,1	158,0	119,3	90,95	70,86	56,48	45,95	38,06	27,29	20,50	15,95	12,76	10,44			
120	60	3	y-y	389,1	389,1	366,3	333,9	298,6	260,7	222,7	187,9	158,0	133,3	113,4	84,04	64,40	50,79	41,03	33,81	120	60	3
			z-z	389,1	375,3	319,2	255,2	192,6	143,4	108,4	84,09	66,82	54,25	44,87	32,11	24,09	18,73	14,97	12,24			
120	60	4	y-y	560,6	560,6	522,0	471,7	416,5	357,9	300,9	250,5	208,6	174,8	147,9	108,9	83,17	65,44	52,77	43,43	120	60	4
			z-z	560,6	535,4	447,9	348,2	255,8	187,2	140,3	108,2	85,64	69,37	57,27	40,89	30,63	23,79	19,01	15,53			
120	60	5	y-y	687,0	687,0	638,0	575,3	506,4	433,5	363,2	301,5	250,5	209,7	177,2	130,4	99,45	78,20	63,04	51,87	120	60	5
			z-z	687,0	654,2	544,6	419,9	306,2	223,1	166,8	128,4	101,6	82,25	67,88	48,44	36,27	28,16	22,49	18,38			
120	60	6	y-y	807,8	807,8	748,2	673,2	590,7	503,7	420,4	347,9	288,5	241,1	203,6	149,6	114,0	89,61	72,21	59,40	120	60	6
			z-z	807,8	767,0	635,1	485,6	351,5	255,0	190,2	146,3	115,6	93,52	77,15	55,02	41,18	31,96	25,52	20,85			

Table 11.2.6 Buckling resistance values for rectangular hollow sections of steel grade S420MH (f_y = 420 N/mm^2) *continued*



h = depth
b = width
t = wall thickness
L_{cr} = buckling length
$N_{b.Rd}$ = buckling resistance

The calculated resistance values are design values (see Chapter 2) based on recommended partial safety factor values γ_{M0} **= 1,0** and γ_{M1} **= 1,0 (for Class 4 circular hollow sections** γ_{M1} **= 1,1)** as given in Eurocode 3 (EN 1993). Partial safety factor values may differ in each country. National values must be checked from the National Annex of the relevant country.

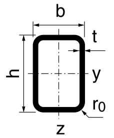
h mm	b mm	t mm	axis	$N_{b.Rd}$ (kN) L_{cr} (m) 0	0,5	1	1,5	2	2,5	3	3,5	4	4,5	5	6	7	8	9	10	h mm	b mm	t mm
120	80	2,5	y-y	342,1	342,1	326,4	300,5	272,6	242,4	211,3	181,5	154,8	132,1	113,1	84,68	65,27	51,66	41,84	34,54	120	80	2,5
			z-z	342,1	342,1	307,7	269,9	228,2	186,7	150,3	121,2	98,75	81,54	68,25	49,61	37,57	29,41	23,62	19,39			
120	80	3	y-y	439,5	439,5	416,3	381,3	343,3	302,4	260,8	221,8	187,7	159,2	135,8	101,1	77,66	61,35	49,61	40,91	120	80	3
			z-z	439,5	437,9	391,0	339,5	283,0	228,2	181,8	145,5	118,0	97,10	81,09	58,77	44,44	34,74	27,88	22,87			
120	80	4	y-y	627,8	627,8	589,3	535,9	477,5	415,1	353,0	296,6	248,8	209,5	177,8	131,6	100,7	79,39	64,10	52,80	120	80	4
			z-z	627,8	621,6	550,3	471,1	385,2	305,0	239,8	190,3	153,5	125,9	104,9	75,75	57,16	44,62	35,78	29,32			
120	80	5	y-y	771,0	771,0	722,1	655,6	582,8	505,0	428,2	358,9	300,3	252,6	214,2	158,3	121,1	95,40	77,00	63,41	120	80	5
			z-z	771,0	762,1	673,2	574,1	467,1	368,2	288,5	228,6	184,1	150,9	125,6	90,67	68,38	53,36	42,78	35,06			
120	80	6	y-y	908,6	908,6	849,1	769,6	682,4	589,4	498,2	416,4	347,8	292,2	247,5	182,7	139,7	110,0	88,71	73,04	120	80	6
			z-z	908,6	896,7	790,2	671,2	543,3	426,3	333,1	263,4	211,9	173,5	144,3	104,1	78,48	61,23	49,08	40,21			
120	80	7,1	y-y	1035	1035	963,4	870,2	767,9	659,3	553,9	460,8	383,6	321,4	271,8	200,2	152,8	120,2	96,94	79,77	120	80	7,1
			z-z	1035	1019	894,6	755,0	606,2	472,4	367,4	289,7	232,6	190,2	158,1	113,9	85,84	66,93	53,63	43,93			
120	80	8	y-y	1144	1144	1062	957,5	842,4	720,7	603,4	500,6	415,9	348,0	294,0	216,3	165,0	129,7	104,6	86,03	120	80	8
			z-z	1144	1124	984,6	827,4	660,7	512,5	397,4	312,8	250,9	205,0	170,3	122,6	92,35	71,99	57,67	47,23			
120	80	8,8	y-y	1237	1237	1145	1030	904,1	770,8	643,3	532,4	441,5	368,9	311,4	228,8	174,4	137,1	110,5	90,87	120	80	8,8
			z-z	1237	1213	1060	887,0	704,7	544,4	421,1	330,9	265,1	216,5	179,8	129,4	97,37	75,89	60,78	49,76			
120	80	10	y-y	1368	1368	1262	1132	988,7	838,4	696,2	573,9	474,6	395,9	333,8	244,8	186,4	146,4	117,9	96,98	120	80	10
			z-z	1368	1338	1165	968,5	763,4	586,0	451,5	354,0	283,2	231,0	191,7	137,8	103,7	80,76	64,66	52,93			
120	100	2,5	y-y	357,7	358	344,3	318,9	291,9	262,6	232,1	201,9	174,2	149,8	129,1	97,47	75,49	59,93	48,64	40,21	120	100	2,5
			z-z	357,7	358	336,8	307,1	274,7	239,9	205,0	173,0	145,5	122,9	104,5	77,46	59,37	46,82	37,82	31,17			
120	100	3	y-y	482,8	483	460,0	423,0	383,1	340,1	295,8	253,6	215,9	184,0	157,4	117,7	90,67	71,74	58,08	47,93	120	100	3
			z-z	482,8	483	449,1	405,6	357,7	306,9	257,7	214,3	178,3	149,4	126,3	93,02	71,00	55,85	45,03	37,06			
120	100	4	y-y	695,0	695	656,0	599,2	537,4	470,9	403,9	341,8	288,2	243,7	207,5	154,1	118,2	93,28	75,38	62,13	120	100	4
			z-z	695,0	695	639,3	572,0	497,7	420,2	347,5	285,6	235,7	196,3	165,3	121,1	92,17	72,36	58,26	47,89			
120	100	5	y-y	855,0	855	805,5	734,7	657,5	574,6	491,5	415,0	349,3	295,0	250,8	186,1	142,6	112,5	90,89	74,90	120	100	5
			z-z	855,0	855	784,6	700,5	607,7	511,3	421,6	345,7	284,9	237,0	199,4	146,0	111,0	87,11	70,12	57,63			
120	100	6	y-y	1009	1009	949,2	864,5	772,0	672,8	573,9	483,4	406,1	342,5	291,0	215,6	165,1	130,2	105,1	86,62	120	100	6
			z-z	1009	1009	924,0	823,3	712,0	597,0	490,7	401,4	330,2	274,5	230,8	168,7	128,3	100,6	80,97	66,53			

$N_{b.Rd}$ (kN)

L_{cr} (m)

h mm	b mm	t mm	axis	0	0,5	1	1,5	2	2,5	3	3,5	4	4,5	5	6	7	8	9	10	h mm	b mm	t mm
140	60	2,5	y-y	308,2	308	300,3	280,4	259,6	237,1	213,3	189,1	165,9	144,7	126,0	96,51	75,37	60,16	49,00	40,62	140	60	2,5
			z-z	308,2	302	262,5	218,3	172,1	132,2	101,8	79,84	63,88	52,11	43,24	31,09	23,39	18,22	14,59	11,94			
140	60	3	y-y	401,5	402	388,4	361,1	332,1	300,9	267,9	235,0	204,1	176,5	152,8	116,0	90,13	71,71	58,28	48,23	140	60	3
			z-z	401,5	390	335,9	274,3	211,7	159,9	122,0	95,14	75,84	61,72	51,13	36,67	27,54	21,44	17,15	14,03			
140	60	4	y-y	602,0	602	598,5	550,7	499,1	443,4	386,0	331,2	282,3	240,6	206,0	154,1	118,7	93,96	76,07	62,79	140	60	4
			z-z	602,0	578	488,1	385,5	287,4	212,2	159,8	123,6	98,05	79,52	65,72	46,98	35,22	27,37	21,87	17,88			
140	60	5	y-y	771,0	771	733,5	673,9	609,6	540,1	468,8	401,1	341,1	290,2	248,2	185,4	142,7	112,8	91,33	75,36	140	60	5
			z-z	771,0	736	615,3	477,8	350,7	256,4	192,1	148,1	117,2	94,96	78,40	55,97	41,92	32,56	26,01	21,25			
140	60	6	y-y	908,6	909	862,7	791,4	714,3	631,1	546,1	465,9	395,2	335,7	286,7	213,8	164,4	130,0	105,2	86,74	140	60	6
			z-z	908,6	865	719,5	554,2	403,7	294,0	219,7	169,1	133,8	108,3	89,37	63,77	47,74	37,07	29,61	24,19			
140	70	2,5	y-y	329,2	329	321,3	300,5	278,6	255,1	230,1	204,6	180,0	157,4	137,4	105,4	82,48	65,90	53,71	44,54	140	70	2,5
			z-z	329,2	327	291,6	252,1	209,0	167,6	132,9	106,1	85,91	70,63	58,93	42,67	32,24	25,19	20,22	16,58			
140	70	3	y-y	426,7	427	413,8	385,4	355,2	322,8	288,5	254,0	221,3	192,0	166,6	126,8	98,69	78,61	63,92	52,93	140	70	3
			z-z	426,7	422	373,0	318,4	259,4	204,7	160,5	127,3	102,5	84,03	69,97	50,52	38,10	29,74	23,84	19,54			
140	70	4	y-y	635,6	636	632,8	583,8	531,0	474,1	415,0	357,9	306,4	262,0	224,8	168,7	130,2	103,2	83,61	69,05	140	70	4
			z-z	635,6	623	542,9	452,8	358,2	275,8	212,9	167,1	133,8	109,2	90,63	65,17	49,04	38,21	30,60	25,05			
140	70	5	y-y	813,0	813	776,4	715,3	649,6	578,5	505,1	434,5	371,1	316,8	271,5	203,5	156,9	124,2	100,6	83,09	140	70	5
			z-z	813,0	793	686,6	566,0	441,7	336,4	258,0	201,7	161,1	131,2	108,8	78,13	58,73	45,73	36,60	29,95			
140	70	6	y-y	959,0	959	914,2	841,1	762,4	677,3	589,6	505,9	431,1	367,5	314,6	235,4	181,3	143,5	116,2	95,90	140	70	6
			z-z	959,0	933	805,7	660,6	512,3	388,4	297,0	231,8	185,0	150,6	124,8	89,54	67,28	52,37	41,91	34,29			
140	80	3	y-y	451,9	452	439,2	409,5	378,2	344,5	308,9	272,8	238,4	207,3	180,2	137,5	107,2	85,48	69,56	57,63	140	80	3
			z-z	451,9	452	407,2	357,7	303,2	248,7	200,7	162,0	132,1	109,2	91,41	66,48	50,37	39,43	31,68	26,01			
140	80	4	y-y	669,2	669	667,0	616,6	562,6	504,3	443,6	384,3	330,1	283,1	243,5	183,3	141,7	112,4	91,12	75,29	140	80	4
			z-z	669,2	665	592,3	511,7	423,8	339,5	269,1	214,7	173,8	142,8	119,2	86,26	65,17	50,92	40,86	33,50			
140	80	5	y-y	855,0	855	819,1	756,4	689,1	616,3	540,7	467,3	400,7	343,1	294,7	221,5	171,1	135,6	109,9	90,79	140	80	5
			z-z	855,0	847	750,3	642,9	526,5	417,4	328,4	260,8	210,4	172,6	143,8	103,90	78,42	61,22	49,10	40,24			
140	80	6	y-y	1009	1009	965,5	890,5	809,8	722,7	632,4	545,2	466,5	398,8	342,2	256,7	198,1	157,0	127,2	105,0	140	80	6
			z-z	1009	998	882,4	753,4	613,9	484,5	380,1	301,3	242,8	199,0	165,7	119,6	90,24	70,43	56,47	46,27			
150	50	2,5	y-y	290,0	290	284,9	267,5	249,4	230,0	209,4	188,1	167,1	147,4	129,5	100,4	79,02	63,37	51,77	43,01	150	50	2,5
			z-z	290,0	278	235,1	185,7	138,4	102,2	76,96	59,52	47,22	38,29	31,65	22,62	16,96	13,18	10,53	8,61			
150	50	3	y-y	381,1	381	371,8	347,6	322,1	294,7	265,6	236,0	207,4	181,2	158,1	121,2	94,79	75,72	61,69	51,15	150	50	3
			z-z	381,1	363	301,4	231,8	168,6	122,7	91,65	70,55	55,80	45,16	37,26	26,58	19,90	15,45	12,34	10,08			
150	50	4	y-y	580,5	581	559,5	518,9	475,5	428,7	379,6	331,1	286,1	246,5	212,7	160,8	124,6	99,02	80,38	66,48	150	50	4
			z-z	580,5	543	438,2	321,6	225,4	160,7	118,7	90,81	71,53	57,72	47,53	33,82	25,27	19,59	15,63	12,76			
150	50	5	y-y	771,0	771	737,4	680,2	618,6	552,1	483,2	416,6	356,4	304,7	261,5	196,2	151,4	120,0	97,20	80,27	150	50	5
			z-z	771,0	714	562,9	400,1	274,4	193,5	142,1	108,3	85,14	68,60	56,43	40,08	29,92	23,18	18,48	15,08			
150	50	6	y-y	908,6	909	867,3	798,8	725,0	645,2	562,9	483,8	413,0	352,4	302,0	226,2	174,4	138,0	111,8	92,30	150	50	6
			z-z	908,6	838	654,9	460,0	313,1	220,0	161,3	122,8	96,46	77,69	63,88	45,35	33,84	26,21	20,89	17,05			

Table 11.2.6 Buckling resistance values for rectangular hollow sections of steel grade S420MH (f_y = 420 N/mm^2) *continued*



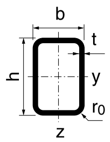
h = depth
b = width
t = wall thickness
L_{cr} = buckling length
$N_{b.Rd}$ = buckling resistance

The calculated resistance values are design values (see Chapter 2) based on recommended partial safety factor values γ_{M0} **= 1,0** and γ_{M1} **= 1,0 (for Class 4 circular hollow sections** γ_{M1} **= 1,1)** as given in Eurocode 3 (EN 1993). Partial safety factor values may differ in each country. National values must be checked from the National Annex of the relevant country.

h mm	b mm	t mm	axis	$N_{b.Rd}$ (kN) L_{cr} (m) 0	0,5	1	1,5	2	2,5	3	3,5	4	4,5	5	6	7	8	9	10	h mm	b mm	t mm
150	100	3	y-y	500,1	500	492,7	463,5	433,2	400,9	366,5	330,8	295,2	261,4	230,6	179,7	141,8	114,0	93,25	77,54	150	100	3
			z-z	500,1	500	471,5	430,3	385,5	337,3	288,9	244,2	205,7	173,8	147,8	109,7	84,14	66,38	53,63	44,20			
150	100	4	y-y	748,5	749	730,8	683,5	633,9	580,7	524,1	466,3	410,3	358,9	313,4	240,7	188,3	150,5	122,7	101,7	150	100	4
			z-z	748,5	749	696,3	628,8	554,7	476,0	399,8	332,5	276,7	231,8	196,0	144,3	110,2	86,66	69,87	57,50			
150	100	5	y-y	981,0	981	952,8	888,1	819,7	746,1	668,3	589,7	514,9	447,4	388,7	296,4	231,0	184,1	149,8	124,1	150	100	5
			z-z	981,0	981	905,3	812,2	709,6	601,9	500,0	412,3	341,0	284,5	239,8	175,9	134,0	105,3	84,78	69,71			
150	100	6	y-y	1161	1161	1126	1048	966,6	878,4	785,2	691,5	602,7	522,9	453,7	345,4	268,9	214,2	174,2	144,2	150	100	6
			z-z	1161	1161	1069	957,0	833,8	705,0	583,9	480,4	396,7	330,6	278,5	204,1	155,4	122,0	98,23	80,76			
150	100	7,1	y-y	1334	1334	1291	1200	1104	1000	891,1	781,9	679,2	587,7	508,9	386,4	300,3	238,9	194,2	160,7	150	100	7,1
			z-z	1334	1334	1224	1093	948,2	797,8	657,9	539,5	444,5	369,9	311,2	227,8	173,2	136,0	109,4	89,94			
150	100	8	y-y	1480	1480	1430	1329	1221	1104	981,5	859,4	745,1	643,7	556,7	422,0	327,6	260,5	211,7	175,1	150	100	8
			z-z	1480	1480	1355	1208	1045	876,2	720,4	589,5	485,0	403,1	339,0	247,9	188,4	147,8	118,9	97,75			
150	100	8,8	y-y	1606	1606	1550	1439	1320	1192	1058	924,1	799,8	690,0	596,1	451,2	350,0	278,2	225,9	186,9	150	100	8,8
			z-z	1606	1606	1468	1305	1126	941,6	772,2	630,6	518,2	430,3	361,6	264,2	200,7	157,4	126,7	104,1			
150	100	10	y-y	1788	1788	1722	1596	1462	1316	1164	1014	875,3	753,5	649,8	490,8	380,3	302,0	245,1	202,7	150	100	10
			z-z	1788	1788	1628	1444	1241	1032	843,1	686,6	563,0	467,0	392,1	286,1	217,2	170,3	136,9	112,5			
160	70	4	y-y	658,1	658	644,6	604,1	561,7	516,4	468,2	418,6	370,1	325,0	284,7	219,7	172,3	138,0	112,6	93,45	160	70	4
			z-z	658,1	648	569,6	481,5	387,4	302,3	235,4	185,8	149,2	122,1	101,5	73,14	55,11	42,98	34,44	28,21			
160	70	5	y-y	897,0	897	871,5	812,6	750,3	683,3	612,4	540,7	472,4	410,7	356,9	272,3	212,3	169,2	137,7	114,1	160	70	5
			z-z	897,0	876	760,9	630,1	494,3	378,0	290,6	227,5	181,9	148,3	123,0	88,35	66,44	51,75	41,43	33,91			
160	70	6	y-y	1060	1060	1028	957,8	883,2	802,8	717,9	632,5	551,4	478,5	415,3	316,3	246,3	196,2	159,6	132,1	160	70	6
			z-z	1060	1033	894,5	736,9	574,6	437,4	335,3	262,1	209,3	170,5	141,3	101,5	76,28	59,39	47,54	38,90			
160	70	8	y-y	1346	1346	1300	1207	1108	1002	889,5	778,0	674,0	581,9	503,0	381,0	295,7	235,1	191,0	158,0	160	70	8
			z-z	1346	1306	1122	912,3	701,0	528,1	402,2	313,2	249,5	202,9	168,1	120,5	90,48	70,40	56,32	46,07			

h mm	b mm	t mm	axis	$N_{b.Rd}$ (kN) L_{cr} (m) 0	0,5	1	1,5	2	2,5	3	3,5	4	4,5	5	6	7	8	9	10	h mm	b mm	t mm
160	80	3	y-y	461,0	461	456,5	430,8	404,3	376,2	346,3	315,0	283,4	252,8	224,5	176,6	140,2	113,1	92,74	77,26	160	80	3
			z-z	461,0	461	420,0	372,6	320,3	266,7	217,9	177,5	145,6	120,8	101,4	74,02	56,20	44,06	35,44	29,11			
160	80	4	y-y	691,7	692	678,9	637,2	593,6	547,0	497,5	446,3	395,9	348,7	306,2	237,1	186,4	149,4	122,0	101,3	160	80	4
			z-z	691,7	691	618,9	540,2	453,7	368,6	295,2	237,2	192,8	158,9	132,9	96,44	72,99	57,09	45,84	37,61			
160	80	5	y-y	939,0	939	914,8	854,5	791,0	722,7	650,2	576,6	505,8	441,2	384,4	294,3	229,9	183,5	149,5	123,9	160	80	5
			z-z	939,0	932	827,1	711,4	585,5	466,3	368,1	293,0	236,7	194,3	162,0	117,1	88,44	69,07	55,41	45,42			
160	80	6	y-y	1110	1110	1080	1008	932,0	850,1	763,4	675,5	591,4	515,0	448,1	342,5	267,3	213,2	173,6	143,8	160	80	6
			z-z	1110	1100	974,5	835,2	684,2	542,5	427,0	339,2	273,6	224,5	187,0	135,1	102,0	79,62	63,86	52,34			
160	80	7,1	y-y	1274	1274	1236	1152	1062	965,8	864,0	761,4	664,1	576,5	500,4	381,2	296,9	236,5	192,4	159,3	160	80	7,1
			z-z	1274	1259	1112	948,3	771,6	608,1	476,6	377,6	304,1	249,2	207,4	149,7	112,9	88,13	70,66	57,90			
160	80	8	y-y	1413	1413	1369	1274	1174	1065	950,9	836,0	727,6	630,5	546,6	415,7	323,3	257,4	209,3	173,3	160	80	8
			z-z	1413	1395	1229	1044	845,6	663,6	518,6	410,1	330,0	270,2	224,8	162,1	122,2	95,36	76,44	62,62			
160	80	10	y-y	1704	1704	1645	1527	1402	1267	1125	983,3	851,6	735,0	635,1	481,0	373,3	296,7	241,0	199,4	160	80	10
			z-z	1704	1676	1469	1237	990,3	769,7	597,7	470,9	377,9	308,9	256,7	184,9	139,2	108,6	86,97	71,23			
160	90	3	y-y	486,2	486	482,1	455,3	427,7	398,5	367,5	334,9	301,9	269,9	240,1	189,3	150,6	121,6	99,80	83,18	160	90	3
			z-z	486,2	486	452,3	408,4	360,2	309,1	259,6	215,9	179,6	150,5	127,2	93,70	71,52	56,26	45,36	37,33			
160	90	4	y-y	725,3	725	713,2	670,2	625,3	577,5	526,5	473,7	421,5	372,2	327,6	254,4	200,4	160,8	131,4	109,2	160	90	4
			z-z	725,3	725	664,9	593,1	513,9	431,7	355,5	291,2	239,8	199,4	167,7	122,7	93,30	73,20	58,92	48,42			
160	90	5	y-y	981,0	981	958,0	896,2	831,3	761,6	687,6	612,0	538,7	471,3	411,6	316,2	247,5	197,8	161,2	133,7	160	90	5
			z-z	981,0	981	888,1	783,5	668,2	551,5	447,3	362,5	296,3	245,2	205,6	149,7	113,5	88,92	71,48	58,69			
160	90	6	y-y	1161	1161	1132	1058	980,4	896,9	808,3	718,0	630,8	551,0	480,6	368,6	288,1	230,1	187,5	155,5	160	90	6
			z-z	1161	1161	1048	922,2	783,6	644,3	521,0	421,3	343,9	284,3	238,2	173,3	131,4	102,9	82,67	67,86			
160	90	7,1	y-y	1334	1334	1298	1211	1120	1022	917,6	812,1	711,1	619,3	539,0	412,0	321,5	256,5	208,8	173,0	160	90	7,1
			z-z	1334	1334	1199	1052	889,0	726,9	585,2	471,9	384,4	317,4	265,6	193,1	146,2	114,4	91,94	75,45			
160	90	8	y-y	1480	1480	1438	1341	1239	1128	1011	893,0	780,4	678,5	589,7	450,0	350,8	279,6	227,6	188,5	160	90	8
			z-z	1480	1480	1327	1161	977,7	796,4	639,3	514,4	418,5	345,2	288,8	209,7	158,8	124,2	99,76	81,86			
160	90	8,8	y-y	1606	1606	1559	1452	1340	1218	1090	960,7	838,0	727,5	631,5	481,2	374,7	298,5	242,8	201,1	160	90	8,8
			z-z	1606	1604	1436	1253	1052	854,0	683,7	549,2	446,3	367,9	307,5	223,2	168,9	132,1	106,1	87,02			

Table 11.2.6 Buckling resistance values for rectangular hollow sections of steel grade S420MH (f_y = 420 N/mm^2) *continued*



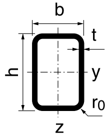
h = depth
b = width
t = wall thickness
L_{cr} = buckling length
$N_{b.Rd}$ = buckling resistance

The calculated resistance values are design values (see Chapter 2) based on recommended partial safety factor values **γ_{M0} = 1,0** and **γ_{M1} = 1,0 (for Class 4 circular hollow sections γ_{M1} = 1,1)** as given in Eurocode 3 (EN 1993). Partial safety factor values may differ in each country. National values must be checked from the National Annex of the relevant country.

h mm	b mm	t mm	axis	$N_{b.Rd}$ (kN) L_{cr} (m) 0	0,5	1	1,5	2	2,5	3	3,5	4	4,5	5	6	7	8	9	10	h mm	b mm	t mm
180	100	4	y-y	776,1	776	775,6	736,0	695,5	653,1	608,0	560,6	511,8	463,2	416,6	334,2	268,9	218,7	180,4	150,8	180	100	4
			z-z	776,1	776	730,1	665,1	594,2	518,2	442,2	372,7	313,2	264,2	224,5	166,4	127,5	100,5	81,17	66,88			
180	100	5	y-y	1053	1053	1047	989,7	931,1	869,3	803,5	734,4	664,2	595,6	531,2	420,5	335,3	271,2	222,9	185,9	180	100	5
			z-z	1053	1053	980,80	886,2	782,4	672,1	565,0	470,2	391,6	328,2	277,6	204,5	156,1	122,8	99,03	81,50			
180	100	5,6	y-y	1231	1231	1219	1150	1079	1004	924,1	840,5	756,1	674,5	598,9	471,0	374,0	301,6	247,4	206,1	180	100	5,6
			z-z	1231	1231	1139	1024	897,7	764,6	637,5	527,2	436,9	365,0	308,0	226,3	172,5	135,5	109,2	89,81			
180	100	6	y-y	1312	1312	1299	1225	1149	1069	983,3	893,8	803,6	716,5	635,8	499,6	396,5	319,7	262,2	218,4	180	100	6
			z-z	1312	1312	1213	1090	954,5	812,0	676,2	558,7	462,8	386,5	326,0	239,4	182,4	143,3	115,5	94,97			
180	100	7,1	y-y	1513	1513	1495	1409	1320	1225	1124	1019	913,7	812,5	719,4	563,4	446,2	359,3	294,3	245,0	180	100	7,1
			z-z	1513	1513	1394	1250	1091	923,6	766,1	631,0	521,6	434,9	366,5	268,7	204,6	160,7	129,4	106,4			
180	100	8	y-y	1682	1682	1660	1564	1463	1357	1244	1126	1008	894,6	791,0	618,3	489,1	393,5	322,2	268,1	180	100	8
			z-z	1682	1682	1547	1384	1205	1017	841,3	691,5	570,7	475,4	400,4	293,3	223,2	175,2	141,1	116,0			
180	100	8,8	y-y	1828	1828	1803	1697	1587	1470	1346	1216	1087	963,6	851,0	664,1	524,7	421,9	345,3	287,2	180	100	8,8
			z-z	1828	1828	1678	1499	1301	1096	904,1	741,8	611,4	508,8	428,2	313,5	238,4	187,1	150,6	123,8			
180	100	10	y-y	2040	2040	2009	1889	1764	1632	1491	1344	1198	1060	934,6	727,4	573,8	460,9	376,9	313,4	180	100	10
			z-z	2040	2040	1867	1664	1439	1206	991,3	811,0	667,2	554,5	466,2	340,9	259,1	203,2	163,6	134,4			
180	120	4	y-y	843,3	843	843,3	802,4	759,5	714,6	667,0	616,9	565,1	513,2	463,1	373,5	301,5	245,8	203,0	170,0	180	120	4
			z-z	843,3	843	813,9	755,4	693,1	625,9	555,3	485,2	419,9	362,2	312,9	236,9	183,8	146,1	118,6	98,11			
180	120	5	y-y	1137	1137	1133	1073	1012	947,2	878,5	806,3	732,4	659,6	590,5	470,3	376,5	305,3	251,2	209,8	180	120	5
			z-z	1137	1137	1089	1006	915,7	818,6	717,7	619,8	531,2	454,6	390,4	293,3	226,5	179,5	145,5	120,2			
180	120	6	y-y	1413	1413	1403	1326	1247	1163	1074	981,3	886,6	794,3	707,8	559,7	445,9	360,5	296,1	246,9	180	120	6
			z-z	1413	1413	1346	1238	1122	996,1	866,8	743,3	633,2	539,5	461,8	345,4	266,1	210,5	170,4	140,7			
180	120	7,1	y-y	1632	1632	1618	1528	1435	1337	1232	1123	1012	904,7	804,5	634,2	504,3	407,2	334,2	278,5	180	120	7,1
			z-z	1632	1632	1552	1425	1288	1141	989,3	845,7	718,7	611,3	522,5	390,2	300,2	237,4	192,1	158,5			
180	120	8	y-y	1816	1816	1799	1698	1594	1483	1366	1243	1119	998,4	886,7	697,8	554,2	447,1	366,8	305,6	180	120	8
			z-z	1816	1816	1725	1582	1428	1262	1092	931,6	790,4	671,4	573,4	427,7	328,9	260,0	210,3	173,5			
180	120	8,8	y-y	1976	1976	1956	1845	1730	1609	1480	1345	1209	1078	956,1	751,1	596,0	480,5	394,0	328,1	180	120	8,8
			z-z	1976	1976	1874	1717	1548	1365	1179	1004	850,5	721,7	615,9	458,9	352,7	278,7	225,4	185,9			
180	120	10	y-y	2208	2208	2182	2057	1928	1790	1644	1491	1337	1190	1054	825,7	654,2	526,9	431,7	359,4	180	120	10
			z-z	2208	2208	2089	1912	1719	1512	1302	1105	934,5	791,7	674,9	502,1	385,5	304,5	246,1	203,0			
180	120	12,5	y-y	2606	2606	2564	2411	2251	2080	1899	1710	1524	1347	1187	922,6	727,3	584,0	477,5	396,9	180	120	12,5
			z-z	2606	2606	2451	2232	1994	1738	1483	1249	1050	885,5	752,4	557,4	427,1	336,7	271,9	224,0			

h mm	b mm	t mm	axis	$N_{b.Rd}$ (kN), L_{cr} (m): 0	0,5	1	1,5	2	2,5	3	3,5	4	4,5	5	6	7	8	9	10	h mm	b mm	t mm
200	80	4	y-y	722,4	722	722,4	695,3	661,5	626,4	589,5	550,5	510,0	468,6	427,7	351,7	287,8	236,8	196,8	165,5	200	80	4
			z-z	722,4	722	658,3	584,2	502,3	418,4	342,0	278,6	228,6	189,6	159,2	116,2	88,26	69,19	55,65	45,71			
200	80	5	y-y	996,8	997	996,8	950,2	900,0	847,7	792,3	733,8	673,4	612,7	553,8	447,9	362,2	295,7	244,5	204,8	200	80	5
			z-z	996,8	997	894,4	782,8	659,9	538,1	432,2	348,0	283,2	233,7	195,5	142,0	107,5	84,11	67,56	55,44			
200	80	6	y-y	1312	1312	1307	1238	1167	1093	1014	930,7	845,6	761,7	682,1	543,5	435,1	352,9	290,4	242,6	200	80	6
			z-z	1312	1303	1158	998,2	823,8	657,8	520,3	414,6	335,2	275,3	229,6	166,1	125,5	98,00	78,62	64,46			
200	80	7,1	y-y	1512	1512	1504	1423	1340	1252	1159	1061	961,3	863,4	771,1	611,9	488,7	395,6	325,3	271,4	200	80	7,1
			z-z	1512	1499	1329	1140	935,4	742,7	585,1	465,0	375,3	308,0	256,6	185,5	140,0	109,3	87,67	71,86			
200	80	8	y-y	1682	1682	1671	1580	1487	1388	1283	1173	1061	951,1	848,2	671,6	535,6	433,2	355,9	296,9	200	80	8
			z-z	1682	1664	1473	1259	1029	813,3	638,9	506,9	408,7	335,1	279,1	201,5	152,1	118,7	95,18	78,00			
200	80	8,8	y-y	1828	1828	1815	1715	1612	1504	1388	1267	1144	1025	912,6	721,2	574,3	464,2	381,2	317,8	200	80	8,8
			z-z	1828	1807	1596	1360	1106	871,7	683,0	541,1	435,8	357,0	297,2	214,5	161,8	126,3	101,2	82,94			
200	80	10	y-y	2040	2040	2022	1909	1793	1669	1538	1401	1262	1127	1002	789,5	627,6	506,5	415,6	346,3	200	80	10
			z-z	2040	2012	1772	1503	1215	951,7	742,9	587,1	472,2	386,5	321,5	231,8	174,7	136,3	109,3	89,50			
200	100	4	y-y	789,6	790	789,6	762,6	726,6	689,3	650,2	609,0	565,9	521,9	478,0	395,5	325,1	268,4	223,6	188,3	200	100	4
			z-z	789,6	790	747,6	684,5	615,9	542,0	467,0	396,9	335,7	284,6	242,6	180,6	138,7	109,5	88,58	73,04			
200	100	5	y-y	1081	1081	1081	1035	982,3	927,5	869,7	808,7	745,4	681,3	618,6	504,0	409,8	335,7	278,2	233,4	200	100	5
			z-z	1081	1081	1013	919,6	817,8	709,0	601,4	504,3	422,2	355,2	301,2	222,7	170,4	134,2	108,3	89,20			
200	100	6	y-y	1413	1413	1413	1341	1267	1191	1109	1024	935,2	847,2	762,6	612,6	493,2	401,4	331,2	277,0	200	100	6
			z-z	1413	1413	1309	1178	1035	882,9	737,5	610,7	506,7	423,6	357,6	262,9	200,4	157,5	127,0	104,4			
200	100	7,1	y-y	1632	1632	1629	1545	1459	1369	1273	1172	1068	965,3	867,0	694,0	557,4	452,9	373,3	312,0	200	100	7,1
			z-z	1632	1632	1508	1354	1185	1007	837,9	691,8	572,8	478,1	403,3	296,0	225,5	177,2	142,7	117,4			
200	100	8	y-y	1816	1816	1812	1717	1621	1519	1411	1298	1181	1066	956,0	763,7	612,5	497,3	409,6	342,3	200	100	8
			z-z	1816	1816	1675	1502	1311	1111	921,5	759,2	627,7	523,4	441,2	323,5	246,3	193,5	155,8	128,1			
200	100	8,8	y-y	1976	1976	1969	1866	1760	1648	1530	1405	1277	1151	1031	822,2	658,6	534,3	439,9	367,4	200	100	8,8
			z-z	1976	1976	1819	1628	1418	1198	991,6	815,5	673,3	561,0	472,5	346,2	263,5	206,9	166,6	137,0			
200	100	10	y-y	2208	2208	2198	2081	1961	1834	1699	1558	1413	1271	1137	904,0	722,9	585,7	481,8	402,2	200	100	10
			z-z	2208	2208	2026	1809	1570	1321	1090	893,5	736,3	612,6	515,6	377,3	287,0	225,2	181,3	149,0			
200	100	12,5	y-y	2606	2606	2583	2438	2290	2132	1965	1789	1612	1440	1280	1008	801,2	646,7	530,6	442,1	200	100	12,5
			z-z	2606	2606	2372	2103	1806	1502	1226	998,0	818,3	678,5	569,6	415,6	315,5	247,3	198,9	163,4			

Table 11.2.6 Buckling resistance values for rectangular hollow sections of steel grade S420MH (f_y = 420 N/mm^2) *continued*



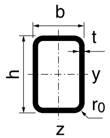
h = depth
b = width
t = wall thickness
L_{cr} = buckling length
$N_{b.Rd}$ = buckling resistance

The calculated resistance values are design values (see Chapter 2) based on recommended partial safety factor values **γ_{M0} = 1,0** and **γ_{M1} = 1,0 (for Class 4 circular hollow sections γ_{M1} = 1,1)** as given in Eurocode 3 (EN 1993). Partial safety factor values may differ in each country. National values must be checked from the National Annex of the relevant country.

h mm	b mm	t mm	axis	$N_{b.Rd}$ (kN) L_{cr} (m) 0	0,5	1	1,5	2	2,5	3	3,5	4	4,5	5	6	7	8	9	10	h mm	b mm	t mm
200	120	5	y-y	1165	1165	1165	1119	1064	1007	946,3	882,6	816,3	748,9	682,3	559,4	456,8	375,3	311,7	261,9	200	120	5
			z-z	1165	1165	1121	1039	950,6	855,4	755,8	657,7	567,2	488,0	420,6	317,5	245,9	195,2	158,4	131,0			
200	120	6	y-y	1513	1513	1513	1443	1367	1287	1203	1115	1023	930,7	841,3	680,5	550,5	449,4	371,6	311,3	200	120	6
			z-z	1513	1513	1445	1331	1208	1076	938,7	807,1	689,1	588,2	504,1	377,6	291,1	230,5	186,7	154,1			
200	120	7,1	y-y	1751	1751	1751	1666	1577	1483	1384	1280	1172	1064	960,3	774,3	624,9	509,4	420,8	352,3	200	120	7,1
			z-z	1751	1751	1669	1535	1390	1234	1074	920,8	784,3	668,2	571,9	427,7	329,5	260,7	211,1	174,2			
200	120	8	y-y	1951	1951	1951	1853	1753	1648	1537	1419	1298	1178	1061	853,9	688,3	560,6	462,8	387,3	200	120	8
			z-z	1951	1951	1856	1705	1543	1367	1187	1016	863,7	735,0	628,5	469,5	361,4	285,8	231,3	190,9			
200	120	8,8	y-y	2124	2124	2124	2016	1906	1790	1668	1539	1406	1274	1147	921,2	741,7	603,6	498,1	416,7	200	120	8,8
			z-z	2124	2124	2018	1853	1674	1480	1283	1096	930,5	791,0	675,9	504,4	388,1	306,8	248,2	204,8			
200	120	10	y-y	2376	2376	2373	2251	2126	1996	1857	1711	1560	1411	1268	1016	816,7	663,9	547,4	457,7	200	120	10
			z-z	2376	2376	2253	2066	1862	1643	1419	1209	1024	869,4	742,0	553,0	425,0	335,8	271,6	224,0			
200	120	12,5	y-y	2816	2816	2802	2652	2499	2337	2164	1983	1798	1617	1445	1148	917,8	743,4	611,4	510,4	200	120	12,5
			z-z	2816	2816	2656	2424	2172	1901	1628	1377	1160	980,1	833,9	619,0	474,7	374,5	302,6	249,4			
220	120	5	y-y	1187	1187	1187	1157	1107	1056	1002	945,8	886,8	825,9	764,1	644,2	537,5	448,3	376,2	318,5	220	120	5
			z-z	1187	1187	1148	1067	980,6	887,7	789,9	692,3	600,8	519,4	449,5	341,0	264,9	210,7	171,2	141,7			
220	120	6	y-y	1524	1524	1524	1477	1410	1340	1267	1190	1110	1027	944,2	786,6	650,0	538,5	449,7	379,4	220	120	6
			z-z	1524	1524	1464	1354	1237	1110	977,1	847,5	728,8	625,5	538,3	405,5	313,6	248,8	201,8	166,8			
220	120	7,1	y-y	1870	1870	1870	1803	1716	1626	1531	1432	1328	1222	1117	920,3	754,3	621,4	516,9	434,9	220	120	7,1
			z-z	1870	1870	1785	1644	1492	1328	1158	995,5	849,7	725,0	621,2	465,3	358,7	284,0	230,0	189,9			
220	120	8	y-y	2085	2085	2085	2007	1910	1809	1702	1590	1473	1354	1236	1017	832,5	685,1	569,6	479,0	220	120	8
			z-z	2085	2085	1987	1829	1657	1472	1281	1099	936,7	798,3	683,4	511,3	393,9	311,7	252,4	208,3			
220	120	8,8	y-y	2272	2272	2272	2185	2078	1967	1850	1727	1599	1468	1339	1099	898,8	739,0	614,0	516,2	220	120	8,8
			z-z	2272	2272	2163	1988	1799	1596	1386	1187	1010	860,1	735,7	549,9	423,4	334,9	271,1	223,7			
220	120	10	y-y	2544	2544	2544	2443	2322	2196	2064	1924	1778	1631	1485	1216	992,5	815,0	676,6	568,4	220	120	10
			z-z	2544	2544	2417	2219	2004	1773	1536	1312	1114	946,9	809,0	603,7	464,5	367,2	297,1	245,1			

h mm	b mm	t mm	axis	$N_{b.Rd}$ (kN) L_{cr} (m) 0	0,5	1	1,5	2	2,5	3	3,5	4	4,5	5	6	7	8	9	10	h mm	b mm	t mm
250	100	5	y-y	1129	1129	1129	1118	1076	1034	989,9	944,5	897,1	847,7	796,8	693,6	595,0	507,0	432,1	369,9	250	100	5
			z-z	1129	1129	1072	983,7	888,1	784,9	679,5	579,9	492,1	418,2	357,2	266,4	204,9	162,0	131,0	108,1			
250	100	6	y-y	1470	1470	1470	1446	1389	1330	1270	1207	1140	1072	1001	860,3	729,4	615,8	521,2	444,0	250	100	6
			z-z	1470	1470	1383	1260	1126	982,1	838,4	706,7	594,0	501,2	425,9	315,6	241,8	190,7	154,0	126,9			
250	100	8	y-y	2152	2152	2152	2097	2005	1911	1812	1709	1601	1489	1376	1158	964,4	803,4	673,6	569,8	250	100	8
			z-z	2152	2152	1994	1794	1574	1343	1121	927,7	769,5	643,1	542,9	398,9	304,1	239,0	192,6	158,5			
250	100	10	y-y	2628	2628	2628	2554	2440	2323	2200	2071	1936	1797	1657	1388	1152	957,2	801,1	676,9	250	100	10
			z-z	2628	2628	2423	2173	1897	1607	1334	1099	908,5	757,6	638,6	468,3	356,6	280,1	225,6	185,5			
250	150	5	y-y	1339	1339	1339	1332	1285	1237	1187	1136	1083	1028	970,9	853,8	739,5	635,2	544,7	468,4	250	150	5
			z-z	1339	1339	1334	1264	1191	1115	1035	949,9	863,1	777,4	696,1	554,7	444,1	360,1	296,4	247,6			
250	150	6	y-y	1722	1722	1722	1705	1641	1576	1509	1440	1367	1292	1214	1057	906,3	772,1	657,9	563,2	250	150	6
			z-z	1722	1722	1707	1612	1514	1410	1300	1185	1068	954,4	848,7	669,1	532,1	429,6	352,5	293,8			
250	150	7,1	y-y	2141	2141	2141	2190	2102	2012	1918	1821	1719	1613	1505	1289	1090	918,3	776,1	660,4	250	150	7,1
			z-z	2141	2141	2112	1989	1861	1724	1579	1428	1277	1133	1001	781,6	617,8	496,9	406,8	338,4			
250	150	8	y-y	2488	2488	2488	2444	2344	2243	2139	2029	1914	1795	1673	1431	1209	1017	859,1	730,6	250	150	8
			z-z	2488	2488	2447	2299	2145	1981	1807	1626	1446	1277	1124	873,1	687,8	552,0	451,2	375,0			
250	150	8,8	y-y	2715	2715	2715	2665	2556	2445	2330	2209	2083	1952	1818	1553	1310	1102	929,9	790,5	250	150	8,8
			z-z	2715	2715	2668	2506	2337	2156	1964	1765	1569	1384	1217	943,9	743,0	596,0	487,0	404,6			
250	150	10	y-y	3048	3048	3048	2988	2865	2739	2609	2472	2329	2180	2029	1730	1456	1223	1031	875,9	250	150	10
			z-z	3048	3048	2992	2808	2616	2410	2192	1967	1745	1537	1349	1045	821,4	658,4	537,7	446,6			
250	150	12,5	y-y	3656	3656	3656	3573	3421	3266	3104	2934	2756	2572	2385	2020	1691	1414	1189	1008	250	150	12,5
			z-z	3656	3656	3577	3350	3113	2858	2588	2310	2039	1788	1565	1206	945,2	756,2	616,8	511,9			
260	140	6	y-y	1684	1684	1684	1674	1614	1553	1490	1425	1358	1287	1215	1066	921,4	790,1	676,6	581,3	260	140	6
			z-z	1684	1684	1660	1562	1461	1353	1238	1118	999,0	885,4	781,7	609,7	481,6	387,2	316,8	263,5			
260	140	7,1	y-y	2104	2104	2104	2080	2001	1921	1839	1753	1663	1570	1474	1279	1094	930,6	791,9	677,1	260	140	7,1
			z-z	2104	2104	2062	1934	1800	1657	1504	1347	1193	1049	920,2	711,1	558,5	447,4	365,2	303,2			
260	140	8	y-y	2488	2488	2488	2451	2354	2256	2154	2048	1936	1821	1702	1465	1244	1051	890,5	759,0	260	140	8
			z-z	2488	2488	2428	2271	2105	1927	1739	1546	1360	1188	1037	796,2	622,8	497,6	405,5	336,3			
260	140	8,8	y-y	2715	2715	2715	2673	2566	2458	2347	2230	2108	1981	1850	1590	1349	1139	964,0	821,2	260	140	8,8
			z-z	2715	2715	2648	2474	2292	2097	1889	1677	1473	1286	1122	860,0	672,2	536,9	437,4	362,6			
260	140	10	y-y	3048	3048	3048	2997	2877	2755	2628	2495	2357	2213	2065	1771	1499	1264	1069	909,9	260	140	10
			z-z	3048	3048	2968	2772	2565	2342	2106	1866	1636	1426	1242	950,7	742,3	592,4	482,4	399,9			
260	140	12,5	y-y	3656	3656	3656	3583	3435	3284	3127	2962	2790	2611	2429	2069	1741	1461	1232	1046	260	140	12,5
			z-z	3656	3656	3548	3305	3049	2772	2480	2186	1906	1655	1436	1094	852,2	678,9	552,2	457,3			

Table 11.2.6 Buckling resistance values for rectangular hollow sections of steel grade S420MH (f_y = 420 N/mm^2) *continued*



h = depth
b = width
t = wall thickness
L_{cr} = buckling length
$N_{b.Rd}$ = buckling resistance

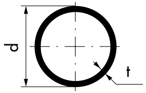
The calculated resistance values are design values (see Chapter 2) based on recommended partial safety factor values **γ_{M0} = 1,0** and **γ_{M1} = 1,0 (for Class 4 circular hollow sections γ_{M1} = 1,1)** as given in Eurocode 3 (EN 1993). Partial safety factor values may differ in each country. National values must be checked from the National Annex of the relevant country.

h mm	b mm	t mm	axis	$N_{b.Rd}$ (kN) L_{cr} (m) 0	0,5	1	1,5	2	2,5	3	3,5	4	4,5	5	6	7	8	9	10	h mm	b mm	t mm
260	180	6	y-y	1886	1886	1886	1880	1815	1749	1681	1611	1539	1463	1385	1223	1065	918,1	789,7	680,8	260	180	6
			z-z	1886	1886	1886	1816	1729	1638	1542	1441	1336	1229	1123	924,5	757,4	623,7	518,7	436,3			
260	180	7,1	y-y	2342	2342	2342	2325	2241	2155	2067	1976	1880	1781	1678	1469	1267	1084	927,2	795,7	260	180	7,1
			z-z	2342	2342	2342	2243	2129	2010	1885	1753	1615	1477	1341	1092	888,1	727,5	602,9	505,9			
260	180	8	y-y	2757	2757	2757	2728	2625	2520	2413	2301	2184	2062	1937	1683	1441	1227	1044	893,3	260	180	8
			z-z	2757	2757	2757	2627	2488	2343	2189	2027	1859	1691	1528	1235	998,4	814,8	673,5	564,2			
260	180	8,8	y-y	3011	3011	3011	2977	2864	2750	2632	2509	2380	2246	2108	1830	1566	1331	1133	968,5	260	180	8,8
			z-z	3011	3011	3011	2866	2714	2554	2385	2207	2022	1838	1659	1339	1082	882,3	729,0	610,5			
260	180	10	y-y	3384	3384	3384	3344	3216	3086	2952	2812	2666	2515	2358	2043	1745	1482	1260	1076	260	180	10
			z-z	3384	3384	3384	3217	3044	2863	2671	2469	2260	2051	1849	1490	1201	979,0	808,5	676,7			
300	100	5	y-y	1160	1160	1160	1160	1139	1105	1070	1034	997,5	959,5	920,1	837,7	752,5	668,4	589,4	518,1	300	100	5
			z-z	1160	1160	1114	1030	940,5	843,7	742,7	644,0	553,7	475,1	408,8	307,9	238,1	188,9	153,2	126,6			
300	100	6	y-y	1525	1525	1525	1525	1487	1439	1390	1340	1288	1235	1179	1063	944,0	829,6	724,7	632,3	300	100	6
			z-z	1525	1525	1451	1333	1205	1068	927,2	793,3	674,5	574,0	490,8	366,6	282,2	223,2	180,6	149,0			
300	100	7,1	y-y	1937	1937	1937	1937	1875	1810	1744	1676	1605	1532	1456	1298	1139	990,6	857,6	742,8	300	100	7,1
			z-z	1937	1937	1825	1665	1491	1304	1116	942,7	793,6	670,4	570,2	423,1	324,4	255,9	206,7	170,4			
300	100	8	y-y	2299	2299	2299	2294	2214	2134	2052	1968	1879	1788	1693	1497	1304	1126	969,1	835,9	300	100	8
			z-z	2299	2299	2151	1952	1734	1501	1271	1065	890,6	748,8	634,8	468,9	358,6	282,5	227,9	187,7			
300	100	8,8	y-y	2715	2715	2715	2696	2598	2498	2396	2290	2180	2065	1946	1704	1469	1258	1075	922,9	300	100	8,8
			z-z	2715	2715	2519	2269	1994	1704	1426	1182	981,2	820,7	693,1	509,7	388,7	305,6	246,3	202,6			
300	100	10	y-y	3048	3048	3048	3023	2912	2799	2684	2563	2438	2307	2172	1898	1633	1396	1192	1022	300	100	10
			z-z	3048	3048	2820	2535	2222	1892	1577	1303	1080	902,2	761,3	559,1	426,1	334,8	269,8	221,9			
300	100	12,5	y-y	3656	3656	3656	3614	3476	3336	3192	3042	2885	2722	2554	2215	1893	1608	1368	1169	300	100	12,5
			z-z	3656	3656	3360	3004	2612	2202	1820	1494	1233	1026	864,1	632,8	481,5	377,9	304,3	250,1			

$N_{b.Rd}$ (kN)

h mm	b mm	t mm	axis	L_{cr} (m) 0	0,5	1	1,5	2	2,5	3	3,5	4	4,5	5	6	7	8	9	10
300	150	6	y-y	1777	1777	1777	1777	1743	1689	1635	1579	1522	1463	1402	1273	1141	1011	889,9	780,9
			z-z	1777	1777	1774	1682	1588	1490	1386	1276	1163	1051	943,8	755,4	606,7	493,0	406,3	339,6
300	150	7,1	y-y	2235	2235	2235	2235	2179	2108	2036	1962	1886	1807	1725	1553	1379	1211	1057	921,8
			z-z	2235	2235	2220	2099	1975	1843	1703	1557	1407	1262	1125	890,3	709,8	574,0	471,6	393,3
300	150	8	y-y	2635	2635	2635	2635	2558	2472	2384	2293	2200	2102	2001	1791	1580	1379	1197	1040
			z-z	2635	2635	2608	2460	2307	2146	1973	1793	1612	1437	1275	1002	794,6	640,7	525,3	437,5
300	150	10	y-y	3468	3468	3468	3464	3345	3226	3104	2978	2846	2710	2569	2277	1988	1720	1483	1280
			z-z	3468	3468	3412	3208	2995	2768	2526	2275	2026	1791	1577	1226	966,5	776,0	634,5	527,4
300	150	12,5	y-y	4181	4181	4181	4166	4019	3872	3721	3564	3401	3231	3056	2695	2341	2016	1732	1492
			z-z	4181	4181	4102	3849	3585	3303	3002	2692	2387	2101	1845	1427	1122	899,2	734,3	609,8
300	200	6	y-y	2000	2000	2000	2000	1971	1913	1854	1794	1733	1669	1604	1466	1323	1181	1046	922,5
			z-z	2000	2000	2000	1966	1886	1805	1721	1634	1542	1447	1349	1155	976,6	822,6	695,1	591,4
300	200	7,1	y-y	2533	2533	2533	2533	2481	2404	2326	2246	2163	2077	1989	1803	1612	1426	1252	1097
			z-z	2533	2533	2533	2474	2368	2260	2147	2029	1905	1777	1647	1393	1165	973,7	818,1	693,2
300	200	8	y-y	2971	2971	2971	2971	2900	2806	2711	2614	2514	2410	2302	2076	1846	1624	1420	1239
			z-z	2971	2971	2971	2891	2763	2632	2495	2350	2200	2044	1887	1584	1317	1096	917,8	775,9
300	200	8,8	y-y	3454	3454	3454	3454	3356	3243	3129	3011	2889	2762	2631	2357	2081	1818	1580	1373
			z-z	3454	3454	3454	3345	3191	3032	2865	2689	2506	2317	2128	1770	1461	1209	1009	851,0
300	200	10	y-y	3888	3888	3888	3888	3774	3646	3517	3383	3244	3100	2951	2641	2329	2032	1764	1532
			z-z	3888	3888	3888	3761	3586	3406	3216	3017	2808	2594	2380	1976	1629	1347	1123	946,7
300	200	12,5	y-y	4706	4706	4706	4706	4556	4399	4238	4073	3901	3722	3537	3153	2769	2407	2084	1805
			z-z	4706	4706	4706	4541	4325	4101	3866	3618	3360	3095	2832	2340	1921	1584	1319	1110
400	200	6	y-y	2067	2067	2067	2067	2067	2062	2018	1975	1932	1888	1843	1751	1654	1553	1448	1341
			z-z	2067	2067	2067	2057	1983	1909	1833	1754	1672	1587	1498	1317	1141	979,6	839,9	722,2
400	200	7,1	y-y	2646	2646	2646	2646	2646	2624	2566	2508	2449	2390	2329	2203	2070	1931	1788	1645
			z-z	2646	2646	2646	2618	2519	2418	2315	2207	2095	1977	1857	1613	1380	1174	999,6	854,9
400	200	8	y-y	3135	3135	3135	3135	3135	3098	3027	2956	2884	2810	2735	2580	2416	2244	2069	1894
			z-z	3135	3135	3135	3090	2968	2845	2717	2584	2445	2300	2151	1854	1575	1332	1129	962,6
400	200	8,8	y-y	3587	3587	3587	3587	3587	3534	3451	3367	3282	3195	3107	2923	2728	2526	2320	2116
			z-z	3587	3587	3587	3524	3381	3235	3085	2927	2762	2590	2415	2067	1746	1470	1242	1056
400	200	10	y-y	4287	4287	4287	4287	4287	4206	4103	3999	3894	3786	3676	3445	3202	2950	2695	2446
			z-z	4287	4287	4287	4193	4016	3835	3646	3449	3242	3027	2809	2382	1997	1672	1406	1192
400	200	12,5	y-y	5756	5756	5756	5756	5749	5601	5452	5302	5150	4993	4832	4495	4140	3776	3413	3067
			z-z	5756	5756	5756	5581	5326	5064	4789	4500	4197	3886	3574	2979	2463	2042	1705	1439

Table 11.2.7 Buckling resistance values for circular longitudinally welded hollow sections of steel grade S420MH (f_y = 420 N/mm^2)



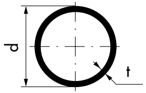
d = external diameter
t = wall thickness
L_{cr} = buckling length
$N_{b.Rd}$ = buckling resistance

The calculated resistance values are design values (see Chapter 2) based on recommended partial safety factor values γ_{M0} **= 1,0** and γ_{M1} **= 1,0 (for Class 4 circular hollow sections** γ_{M1} **= 1,1)** as given in Eurocode 3 (EN 1993). Partial safety factor values may differ in each country. National values must be checked from the National Annex of the relevant country.

d mm	t mm	$N_{b.Rd}$ (kN) L_{cr} (m) 0	0,5	1	1,5	2	2,5	3	3,5	4	4,5	5	6	7	8	9	10	d mm	t mm
26,9	2	65,71	43,27	18,46	9,24	5,47												26,9	2
26,9	2,5	80,49	52,25	21,96	10,96	6,48												26,9	2,5
26,9	2,6	83,36	53,96	22,61	11,27	6,67												26,9	2,6
33,7	2	83,65	64,01	33,69	17,89	10,81	7,19	5,12										33,7	2
33,7	2,5	102,9	78,17	40,61	21,47	12,95	8,61	6,13										33,7	2,5
33,7	2,6	106,7	80,91	41,93	22,15	13,35	8,88	6,32										33,7	2,6
33,7	3	121,5	91,60	46,98	24,73	14,89	9,89	7,04										33,7	3
33,7	3,2	128,8	96,77	49,38	25,94	15,61	10,37	7,38										33,7	3,2
42,4	2	106,6	90,01	57,85	33,77	21,08	14,24	10,22	7,69									42,4	2
42,4	2,5	131,6	110,7	70,52	40,95	25,51	17,21	12,35	9,28									42,4	2,5
42,4	2,6	136,5	114,7	72,97	42,32	26,36	17,78	12,76	9,59									42,4	2,6
42,4	2,9	151,1	126,7	80,16	46,34	28,82	19,44	13,94	10,48									42,4	2,9
42,4	3	156,0	130,6	82,51	47,65	29,62	19,97	14,33	10,76									42,4	3
42,4	3,2	165,5	138,4	87,11	50,20	31,19	21,02	15,07	11,32									42,4	3,2
42,4	4	202,7	168,4	104,5	59,70	36,98	24,89	17,84	13,39									42,4	4
48,3	2	122,2	107,4	75,64	47,34	30,38	20,77	15,01	11,33	8,85								48,3	2
48,3	2,5	151,1	132,4	92,69	57,69	36,93	25,23	18,22	13,75	10,73								48,3	2,5
48,3	2,6	156,8	137,3	96,01	59,69	38,19	26,08	18,84	14,22	11,10								48,3	2,6
48,3	3	179,3	156,7	109,0	67,47	43,09	29,41	21,23	16,01	12,50								48,3	3
48,3	3,2	190,4	166,3	115,3	71,23	45,45	31,00	22,38	16,88	13,17								48,3	3,2
48,3	4	233,8	203,2	139,5	85,41	54,31	36,98	26,67	20,11	15,69								48,3	4
60,3	2	153,9	142,5	112,4	79,96	54,86	38,69	28,43	21,67	17,03	13,72	11,29						60,3	2
60,3	2,5	190,7	176,3	138,7	98,17	67,16	47,30	34,73	26,46	20,79	16,75	13,78						60,3	2,5
60,3	2,9	219,6	202,8	159,1	112,2	76,61	53,90	39,55	30,13	23,66	19,06	15,68						60,3	2,9
60,3	3	226,8	209,4	164,1	115,7	78,92	55,51	40,73	31,02	24,36	19,63	16,14						60,3	3
60,3	3,2	241,1	222,4	174,1	122,5	83,48	58,69	43,04	32,78	25,74	20,73	17,05						60,3	3,2
60,3	4	297,1	273,4	212,9	148,7	100,9	70,77	51,84	39,45	30,97	24,94	20,50						60,3	4
60,3	5	364,8	334,5	258,7	179,1	120,8	84,55	61,86	47,04	36,91	29,71	24,41						60,3	5

d mm	t mm	$N_{b.Rd}$ (kN), L_{cr} (m): 0	0,5	1	1,5	2	2,5	3	3,5	4	4,5	5	6	7	8	9	10	d mm	t mm
76,1	2	195,5	188,4	160,1	127,7	96,12	71,42	53,98	41,83	33,23	26,97	22,30	15,96					76,1	2
76,1	2,5	242,8	233,7	198,2	157,7	118,4	87,83	66,31	51,36	40,78	33,10	27,36	19,57					76,1	2,5
76,1	2,9	280,1	269,4	228,2	181,2	135,8	100,6	75,86	58,73	46,62	37,83	31,27	22,36					76,1	2,9
76,1	3	289,4	278,3	235,7	187,0	140,0	103,7	78,20	60,54	48,05	38,99	32,23	23,05					76,1	3
76,1	4	380,5	365,3	308,3	243,3	181,2	133,7	100,7	77,86	61,76	50,08	41,38	29,58					76,1	4
76,1	5	469,1	449,4	378,1	296,7	219,8	161,7	121,5	93,86	74,40	60,30	49,81	35,59					76,1	5
76,1	6	555,0	530,7	444,9	347,2	255,9	187,6	140,7	108,6	86,02	69,69	57,55	41,11					76,1	6
76,1	6,3	580,2	554,5	464,4	361,8	266,2	195,0	146,2	112,8	89,32	72,36	59,75	42,67					76,1	6,3
88,9	2	229,3	225,6	198,0	166,9	133,9	104,2	80,97	63,82	51,24	41,89	34,82	25,08	18,89				88,9	2
88,9	2,5	285,0	280,2	245,6	206,8	165,5	128,5	99,80	78,60	63,08	51,55	42,84	30,85	23,24				88,9	2,5
88,9	3	340,0	334,1	292,6	245,9	196,3	152,3	118,1	92,94	74,55	60,91	50,61	36,44	27,44				88,9	3
88,9	3,2	361,9	355,5	311,2	261,3	208,5	161,6	125,2	98,54	79,03	64,57	53,64	38,62	29,08				88,9	3,2
88,9	4	448,1	439,7	384,3	321,8	255,9	197,8	153,0	120,3	96,39	78,72	65,37	47,04	35,41				88,9	4
88,9	5	553,5	542,4	473,1	394,7	312,5	240,7	185,9	145,9	116,8	95,35	79,16	56,93	42,84				88,9	5
88,9	6	656,3	642,2	558,9	464,8	366,4	281,2	216,7	169,9	135,9	110,9	92,01	66,14	49,76				88,9	6
88,9	6,3	686,6	671,6	584,2	485,2	382,0	292,9	225,6	176,8	141,4	115,3	95,69	68,78	51,73				88,9	6,3
101,6	2	213,0	213,0	193,2	170,8	146,0	120,8	98,19	79,69	65,21	54,00	45,29	33,00	25,04	19,62			101,6	2
101,6	2,5	326,9	326,4	292,3	254,9	213,9	173,6	138,9	111,6	90,67	74,73	62,47	45,33	34,30	26,83			101,6	2,5
101,6	3	390,3	389,5	348,5	303,7	254,4	206,2	164,8	132,2	107,4	88,49	73,95	53,65	40,59	31,74			101,6	3
101,6	3,6	465,5	464,3	415,1	361,2	302,1	244,3	195,0	156,3	126,9	104,5	87,31	63,32	47,89	37,44			101,6	3,6
101,6	4	515,1	513,5	458,9	399,0	333,2	269,1	214,6	172,0	139,5	114,9	95,96	69,57	52,61	41,13			101,6	4
101,6	5	637,3	634,7	566,4	491,3	409,0	329,3	262,0	209,6	169,9	139,8	116,7	84,57	63,94	49,97			101,6	5
101,6	6	756,8	752,9	671,0	580,6	481,8	386,8	307,0	245,3	198,6	163,3	136,3	98,69	74,58	58,28			101,6	6
101,6	6,3	792,2	787,8	701,8	606,9	503,1	403,5	320,1	255,6	206,9	170,1	141,9	102,8	77,65	60,67			101,6	6,3
108	2	224,7	224,7	206,7	184,9	160,9	135,8	112,3	92,32	76,18	63,45	53,44	39,14	29,79	23,38	18,83		108	2
108	2,5	348,0	348,0	315,7	279,0	238,5	197,4	160,5	130,2	106,6	88,25	74,01	53,94	40,92	32,06	25,77		108	2,5
108	3	415,6	415,6	376,6	332,6	283,9	234,6	190,5	154,5	126,4	104,6	87,70	63,89	48,46	37,96	30,52		108	3
108	3,6	495,9	495,9	448,8	395,9	337,4	278,4	225,7	182,9	149,5	123,7	103,7	75,49	57,24	44,83	36,04		108	3,6
108	4	548,9	548,9	496,3	437,5	372,5	307,0	248,7	201,3	164,5	136,1	114,0	83,01	62,94	49,29	39,62		108	4
108	5	679,5	679,5	613,2	539,4	458,1	376,4	304,2	245,9	200,7	165,9	138,9	101,1	76,62	59,99	48,21		108	5
108	6	807,5	807,5	727,1	638,5	540,7	443,0	357,2	288,3	235,0	194,1	162,5	118,2	89,54	70,08	56,31		108	6
108	6,3	845,4	845,4	760,8	667,6	564,9	462,4	372,6	300,6	245,0	202,3	169,4	123,1	93,27	73,00	58,65		108	6,3
114,3	2	236,0	236,0	219,7	198,5	175,2	150,5	126,5	105,3	87,68	73,48	62,15	45,78	34,95	27,50	22,17	18,25	114,3	2
114,3	2,5	368,8	368,8	338,6	302,5	262,7	221,2	182,5	149,8	123,4	102,7	86,47	63,30	48,15	37,79	30,42	25,00	114,3	2,5
114,3	3	440,6	440,6	404,2	360,8	312,9	263,2	216,9	177,8	146,5	121,9	102,5	75,04	57,07	44,78	36,05	29,63	114,3	3
114,3	3,6	525,8	525,8	481,9	429,8	372,2	312,6	257,3	210,7	173,5	144,2	121,3	88,75	67,47	52,94	42,61	35,01	114,3	3,6
114,3	4	582,2	582,2	533,1	475,2	411,2	344,9	283,6	232,1	191,0	158,8	133,5	97,65	74,23	58,23	46,86	38,51	114,3	4
114,3	5	721,1	721,1	659,2	586,6	506,4	423,7	347,7	284,1	233,5	193,9	163,0	119,1	90,51	70,99	57,12	46,93	114,3	5
114,3	6	857,4	857,4	782,3	695,1	598,7	499,6	409,0	333,7	273,9	227,4	191,0	139,5	105,9	83,07	66,83	54,90	114,3	6
114,3	6,3	897,8	897,8	818,7	727,0	625,8	521,8	426,9	348,1	285,7	237,1	199,1	145,4	110,4	86,56	69,63	57,20	114,3	6,3

Table 11.2.7 Buckling resistance values for circular longitudinally welded hollow sections of steel grade S420MH (f_y = 420 N/mm^2) *continued*



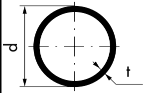
d = external diameter
t = wall thickness
L_{cr} = buckling length
$N_{b.Rd}$ = buckling resistance

The calculated resistance values are design values (see Chapter 2) based on recommended partial safety factor values **γ_{M0} = 1,0** and **γ_{M1} = 1,0 (for Class 4 circular hollow sections γ_{M1} = 1,1)** as given in Eurocode 3 (EN 1993). Partial safety factor values may differ in each country. National values must be checked from the National Annex of the relevant country.

d mm	t mm	$N_{b.Rd}$ (kN) L_{cr} (m) 0	0,5	1	1,5	2	2,5	3	3,5	4	4,5	5	6	7	8	9	10	d mm	t mm
127	2	258,1	258,1	245,2	225,0	203,1	179,5	155,4	132,6	112,6	95,64	81,69	60,94	46,87	37,05	29,97	24,73	127	2
127	2,5	332,8	332,8	315,0	288,3	259,3	228,1	196,4	166,8	141,1	119,5	101,9	75,81	58,22	45,97	37,17	30,65	127	2,5
127	3	490,8	490,8	459,7	417,4	371,0	321,5	272,5	228,4	191,2	160,8	136,4	100,8	77,08	60,72	49,00	40,35	127	3
127	4	649,2	649,2	607,1	550,6	488,7	422,6	357,5	299,1	250,0	210,1	178,1	131,5	100,5	79,17	63,88	52,60	127	4
127	5	804,9	804,9	751,7	680,9	603,3	520,6	439,6	367,2	306,6	257,4	218,0	160,9	122,9	96,78	78,07	64,27	127	5
127	6	957,9	957,9	893,3	808,3	715,0	615,7	518,9	432,7	360,8	302,7	256,2	188,9	144,3	113,6	91,59	75,39	127	6
127	6,3	1003	1003	935,3	846,0	747,9	643,7	542,1	451,8	376,6	315,9	267,3	197,0	150,5	118,4	95,52	78,62	127	6,3
133	2	268,4	268,4	256,9	237,1	215,8	192,8	169,0	145,8	124,9	106,9	91,76	68,90	53,20	42,16	34,16	28,22	133	2
133	2,5	346,5	346,5	330,6	304,4	276,1	245,6	214,1	183,9	156,9	133,8	114,6	85,83	66,16	52,37	42,41	35,01	133	2,5
133	3	514,6	514,6	485,9	443,9	398,2	349,1	299,6	253,7	214,0	181,0	154,1	114,5	87,81	69,30	56,01	46,17	133	3
133	4	680,8	680,8	642,1	586,0	525,0	459,4	393,5	332,6	280,2	236,8	201,5	149,5	114,7	90,47	73,09	60,24	133	4
133	5	844,5	844,5	795,3	725,2	648,8	566,7	484,5	408,9	344,0	290,4	246,9	183,1	140,4	110,7	89,43	73,69	133	5
133	6	1005	1005	945,7	861,5	769,6	671,0	572,5	482,4	405,4	341,9	290,5	215,3	164,9	130,0	105,0	86,53	133	6
133	6,3	1053	1053	990,3	901,8	805,3	701,7	598,4	504,0	423,3	357,0	303,3	224,7	172,1	135,7	109,6	90,27	133	6,3
139,7	3	541,1	541,1	515,1	473,5	428,5	380,0	330,1	282,7	240,5	204,8	175,1	130,9	100,8	79,71	64,52	53,25	139,7	3
139,7	4	716,2	716,2	681,0	625,5	565,4	500,5	434,1	371,1	315,4	268,2	229,3	171,2	131,7	104,2	84,30	69,56	139,7	4
139,7	5	888,7	888,7	844,1	774,6	699,3	618,1	535,2	456,8	387,7	329,4	281,4	209,9	161,4	127,6	103,3	85,18	139,7	5
139,7	6	1058	1058	1004	920,7	830,3	732,7	633,3	539,7	457,4	388,3	331,5	247,1	189,9	150,1	121,4	100,1	139,7	6
139,7	6,3	1109	1109	1052	964,0	869,0	766,5	662,1	564,0	477,9	405,6	346,1	257,9	198,2	156,6	126,7	104,5	139,7	6,3
139,7	8	1390	1390	1316	1204	1083	952,9	820,7	697,2	589,5	499,5	425,8	316,9	243,3	192,2	155,4	128,1	139,7	8
139,7	10	1711	1711	1616	1476	1325	1161	996,7	844,0	711,9	602,2	512,7	380,9	292,2	230,6	186,4	153,6	139,7	10
152,4	3	479,3	479,3	466,0	434,7	401,7	366,1	328,4	290,3	253,9	220,9	192,1	146,7	114,4	91,24	74,26	61,53	152,4	3
152,4	4	783,2	783,2	754,9	700,0	641,4	578,2	511,9	446,4	385,6	332,2	286,6	216,7	167,9	133,4	108,3	89,56	152,4	4
152,4	5	972,4	972,4	936,4	867,7	794,4	715,2	632,3	550,6	475,1	408,8	352,5	266,2	206,2	163,7	132,9	109,9	152,4	5
152,4	6	1159	1159	1115	1033	944,4	849,3	749,7	651,9	561,8	483,0	416,1	313,9	243,1	192,9	156,5	129,4	152,4	6
152,4	6,3	1214	1214	1168	1081	988,9	888,9	784,4	681,7	587,3	504,7	434,8	327,9	253,8	201,5	163,5	135,1	152,4	6,3
159	3	497,4	497,4	486,5	455,6	423,2	388,5	351,6	313,7	276,8	242,7	212,3	163,5	128,2	102,5	83,61	69,38	159	3
159	4	818,1	818,1	793,3	738,6	680,7	618,3	552,5	486,3	423,6	367,4	318,6	242,5	188,7	150,3	122,2	101,2	159	4
159	5	1016	1016	984,3	916,0	843,5	765,4	683,0	600,3	522,4	452,6	392,2	298,2	231,9	184,6	150,1	124,3	159	5
159	6	1211	1211	1173	1090	1003	909,5	810,5	711,5	618,3	535,2	463,5	352,0	273,6	217,8	177,0	146,5	159	6
159	6,3	1269	1269	1228	1142	1051	952,2	848,2	744,2	646,6	559,4	484,4	367,8	285,8	227,4	184,8	153,0	159	6,3

d mm	t mm	$N_{b.Rd}$ (kN), L_{cr} (m): 0	0,5	1	1,5	2	2,5	3	3,5	4	4,5	5	6	7	8	9	10	d mm	t mm
168,3	3	522,5	522,5	515,0	484,6	453,0	419,4	383,6	346,4	309,3	274,0	241,8	188,5	148,8	119,6	97,89	81,41	168,3	3
168,3	3,2	562,1	562,1	553,7	520,7	486,5	450,1	411,2	370,9	330,8	292,8	258,1	201,0	158,6	127,4	104,2	86,64	168,3	3,2
168,3	4	867,2	867,2	847,3	792,9	735,9	674,6	609,6	543,0	478,4	418,8	366,0	281,4	220,3	176,1	143,6	119,1	168,3	4
168,3	4,5	972,6	972,6	950,0	888,8	824,5	755,6	682,3	607,5	534,9	468,0	408,8	314,2	245,9	196,5	160,2	132,9	168,3	4,5
168,3	5	1077	1077	1052	983,9	912,5	835,8	754,4	671,2	590,6	516,6	451,1	346,4	271,1	216,6	176,6	146,4	168,3	5
168,3	6	1285	1285	1254	1172	1086	994,0	896,2	796,4	700,0	611,6	533,7	409,4	320,1	255,7	208,4	172,8	168,3	6
168,3	6,3	1347	1347	1314	1228	1138	1041	938,1	833,4	732,3	639,6	558,0	427,9	334,5	267,2	217,7	180,5	168,3	6,3
168,3	8	1692	1692	1648	1539	1425	1302	1171	1038	910,2	793,8	691,5	529,4	413,4	330,0	268,7	222,8	168,3	8
168,3	10	2089	2089	2032	1895	1752	1597	1434	1268	1109	965,5	839,8	641,6	500,5	399,2	324,9	269,2	168,3	10
193,7	4	1001	1001	995,0	940,9	885,4	826,8	764,4	699,0	632,3	567,2	506,0	400,8	319,7	258,6	212,5	177,3	193,7	4
193,7	5	1245	1245	1236	1169	1099	1026	948,0	866,1	782,8	701,6	625,4	494,8	394,4	318,9	262,0	218,5	193,7	5
193,7	6	1486	1486	1475	1394	1311	1222	1129	1030	930,4	833,1	742,0	586,4	467,0	377,4	310,0	258,4	193,7	6
193,7	6,3	1558	1558	1546	1461	1373	1281	1182	1079	974,0	871,9	776,5	613,4	488,4	394,7	324,1	270,2	193,7	6,3
193,7	8	1960	1960	1944	1836	1724	1606	1481	1349	1216	1087	967,0	762,4	606,3	489,5	401,8	334,8	193,7	8
193,7	10	2424	2424	2401	2266	2126	1978	1821	1657	1491	1330	1181	929,1	737,8	595,2	488,1	406,6	193,7	10
193,7	12,5	2989	2989	2956	2787	2612	2427	2230	2024	1817	1618	1434	1125	891,6	718,4	588,8	490,2	193,7	12,5
219,1	4	909,9	909,9	909,9	880,5	839,6	797,4	753,1	706,5	657,8	607,8	557,8	463,2	381,9	315,8	263,4	222,1	219,1	4
219,1	4,5	1274	1274	1274	1222	1160	1096	1029	957,4	883,5	808,5	734,9	600,0	488,5	400,5	332,1	278,8	219,1	4,5
219,1	5	1412	1412	1412	1354	1285	1214	1139	1060	977,8	894,5	812,8	663,2	539,7	442,4	366,8	307,9	219,1	5
219,1	6	1687	1687	1687	1616	1534	1448	1358	1263	1164	1064	966,5	787,7	640,5	524,7	434,9	364,9	219,1	6
219,1	6,3	1769	1769	1769	1694	1608	1518	1423	1323	1220	1115	1012	824,5	670,2	549,0	455,0	381,8	219,1	6,3
219,1	8	2228	2228	2228	2131	2022	1908	1787	1660	1528	1395	1265	1029	835,0	683,3	565,9	474,6	219,1	8
219,1	10	2759	2759	2759	2635	2498	2355	2204	2045	1880	1714	1552	1259	1020	834,1	690,3	578,7	219,1	10
219,1	12,5	3408	3408	3408	3249	3077	2898	2709	2510	2303	2096	1894	1532	1240	1012	836,7	701,0	219,1	12,5
244,5	6	1888	1888	1888	1837	1756	1672	1585	1493	1397	1298	1198	1006	836,0	695,5	582,5	492,4	244,5	6
244,5	8	2496	2496	2496	2426	2318	2206	2090	1967	1839	1707	1574	1318	1094	908,6	760,4	642,5	244,5	8
244,5	10	3094	3094	3094	3004	2868	2729	2583	2429	2269	2103	1937	1619	1341	1113	930,5	785,7	244,5	10
244,5	12,5	3826	3826	3826	3710	3540	3366	3183	2990	2789	2582	2375	1979	1636	1356	1132	955,6	244,5	12,5
273	4	1097	1097	1097	1097	1055	1016	976,2	935,2	892,4	847,9	801,8	707,2	614,3	529,0	454,5	391,4	273	4
273	5	1418	1418	1418	1409	1359	1307	1255	1200	1143	1084	1023	897,5	775,8	665,3	569,8	489,5	273	5
273	6	2114	2114	2114	2086	2005	1922	1838	1749	1656	1560	1461	1262	1075	910,8	773,0	659,7	273	6
273	6,3	2217	2217	2217	2187	2102	2016	1927	1834	1736	1635	1531	1322	1126	953,9	809,4	690,7	273	6,3
273	8	2797	2797	2797	2758	2650	2540	2427	2308	2185	2056	1924	1659	1410	1194	1012	863,1	273	8
273	10	3470	3470	3470	3418	3283	3146	3004	2856	2701	2540	2374	2044	1735	1466	1242	1059	273	10
273	12,5	4297	4297	4297	4227	4059	3887	3709	3524	3330	3128	2921	2508	2125	1793	1517	1292	273	12,5
323,9	4	1262	1262	1262	1262	1243	1207	1170	1132	1094	1054	1013	926,2	836,4	746,8	661,7	584,0	323,9	4
323,9	5	1641	1641	1641	1641	1613	1564	1515	1464	1413	1359	1304	1187	1067	948,8	837,1	736,2	323,9	5
323,9	6	2022	2022	2022	2022	1982	1921	1859	1796	1731	1663	1594	1447	1296	1148	1010	886,3	323,9	6
323,9	6,3	2136	2136	2136	2136	2093	2029	1963	1896	1826	1755	1680	1525	1365	1208	1062	930,9	323,9	6,3
323,9	8	3335	3335	3335	3335	3242	3133	3023	2910	2793	2671	2545	2282	2017	1763	1534	1334	323,9	8
323,9	10	4142	4142	4142	4142	4023	3888	3750	3608	3462	3309	3151	2823	2491	2175	1890	1642	323,9	10
323,9	12,5	5136	5136	5136	5136	4984	4814	4642	4465	4281	4090	3892	3481	3067	2674	2321	2015	323,9	12,5

Table 11.2.8 Buckling resistance values for circular spirally welded hollow sections of steel grade S420MH (f_y = 420 N/mm^2)
(Technical delivery conditions to be agreed when ordering)



d = external diameter
t = wall thickness
L_{cr} = buckling length
$N_{b.Rd}$ = buckling resistance

The calculated resistance values are design values (see Chapter 2) based on recommended partial safety factor values **γ_{M0} = 1,0** and **γ_{M1} = 1,0 (for Class 4 circular hollow sections γ_{M1} = 1,1)** as given in Eurocode 3 (EN 1993). Partial safety factor values may differ in each country. National values must be checked from the National Annex of the relevant country.

d	t	$N_{b.Rd}$ (kN)																	d	t
mm	mm	L_{cr} (m)																	mm	mm
		0	1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16		
406,4	6,3	2596	2596	2596	2491	2366	2237	2100	1957	1807	1655	1506	1363	1231	1111	1004	908,0	823,5	406,4	6,3
406,4	8	3408	3408	3408	3259	3091	2917	2732	2538	2336	2132	1933	1744	1571	1415	1275	1152	1043	406,4	8
406,4	10	5230	5230	5222	4952	4676	4387	4079	3755	3423	3093	2778	2486	2223	1990	1785	1607	1451	406,4	10
406,4	12,5	6497	6497	6482	6145	5800	5437	5052	4647	4231	3819	3427	3065	2739	2450	2197	1977	1784	406,4	12,5
457	6,3	2862	2862	2862	2796	2676	2554	2427	2293	2153	2008	1861	1715	1574	1441	1317	1203	1101	457	6,3
457	8	3774	3774	3774	3675	3513	3347	3174	2992	2802	2605	2406	2211	2023	1846	1683	1535	1401	457	8
457	10	5898	5898	5898	5688	5415	5134	4837	4525	4199	3867	3536	3217	2917	2642	2394	2171	1973	457	10
457	12	7046	7046	7046	6790	6464	6125	5769	5394	5003	4603	4207	3825	3467	3139	2842	2577	2341	457	12
457	12,5	7331	7331	7331	7064	6724	6371	6001	5610	5202	4786	4373	3975	3603	3261	2953	2677	2432	457	12,5
508	6,3	3119	3119	3119	3090	2974	2858	2737	2613	2482	2346	2206	2064	1922	1783	1650	1524	1407	508	6,3
508	8	4130	4130	4130	4080	3923	3764	3600	3429	3250	3064	2872	2679	2487	2300	2122	1955	1801	508	8
508	10	5326	5326	5326	5247	5040	4830	4613	4386	4149	3902	3649	3394	3143	2900	2669	2455	2257	508	10
508	12,5	8172	8172	8172	7991	7653	7307	6947	6569	6174	5765	5348	4935	4533	4153	3799	3474	3179	508	12,5
559	6,3	3364	3364	3364	3364	3259	3147	3033	2915	2793	2665	2534	2398	2260	2123	1987	1855	1728	559	6,3
559	8	4475	4475	4475	4475	4318	4164	4007	3845	3677	3501	3319	3133	2944	2756	2572	2395	2226	559	8
559	10	5788	5788	5788	5788	5567	5364	5155	4940	4715	4481	4239	3992	3742	3494	3253	3022	2803	559	10
559	12,5	9014	9014	9014	8917	8580	8238	7886	7519	7136	6738	6327	5911	5496	5092	4706	4342	4004	559	12,5
610	8	4807	4807	4807	4807	4699	4550	4399	4244	4084	3918	3747	3569	3388	3204	3020	2839	2663	610	8
610	10	6238	6238	6238	6238	6080	5881	5680	5473	5259	5037	4807	4570	4328	4083	3840	3601	3370	610	10
610	12,5	9855	9855	9855	9844	9507	9167	8821	8463	8090	7703	7301	6890	6473	6058	5652	5261	4889	610	12,5
610	14,2	11163	11163	11163	11147	10764	10379	9985	9578	9155	8714	8258	7790	7317	6846	6385	5941	5520	610	14,2
660	8	5122	5122	5122	5122	5061	4915	4769	4620	4467	4310	4147	3978	3805	3628	3449	3269	3091	660	8
660	10	6667	6667	6667	6667	6568	6374	6179	5979	5774	5562	5343	5117	4884	4646	4407	4168	3932	660	10
660	12,5	8607	8607	8607	8607	8457	8201	7943	7679	7407	7125	6834	6533	6224	5910	5594	5279	4971	660	12,5
660	14,2	12100	12100	12100	12100	11796	11412	11023	10623	10209	9780	9335	8876	8407	7934	7462	7000	6552	660	14,2
711	8	5432	5432	5432	5432	5432	5275	5133	4989	4842	4692	4537	4377	4212	4043	3871	3696	3521	711	8
711	10	7093	7093	7093	7093	7053	6863	6672	6479	6282	6079	5869	5653	5431	5203	4970	4736	4501	711	10
711	12,5	9180	9180	9180	9180	9106	8854	8602	8345	8083	7812	7533	7245	6948	6644	6336	6025	5715	711	12,5
711	14,2	13056	13056	13056	13056	12849	12466	12079	11685	11279	10859	10425	9977	9516	9046	8573	8101	7637	711	14,2

d mm	t mm	$N_{b.Rd}$ (kN), L_{cr} (m): 0	1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	d mm	t mm
762	8	5731	5731	5731	5731	5731	5621	5483	5344	5202	5058	4910	4758	4601	4440	4276	4108	3937	762	8
762	10	7507	7507	7507	7507	7507	7338	7152	6964	6773	6578	6377	6170	5958	5739	5516	5288	5058	762	10
762	12,5	9741	9741	9741	9741	9741	9493	9245	8995	8740	8479	8211	7935	7650	7357	7059	6755	6449	762	12,5
762	14,2	11265	11265	11265	11265	11265	10960	10670	10377	10078	9772	9457	9132	8797	8453	8102	7746	7388	762	14,2
813	8	6019	6019	6019	6019	6019	5956	5821	5685	5549	5410	5268	5122	4973	4820	4663	4502	4338	813	8
813	10	7910	7910	7910	7910	7910	7800	7618	7435	7249	7060	6867	6669	6465	6256	6041	5822	5600	813	10
813	12,5	10290	10290	10290	10290	10290	10118	9874	9630	9382	9129	8870	8603	8330	8049	7760	7466	7168	813	12,5
813	14,2	11914	11914	11914	11914	11914	11697	11411	11124	10833	10536	10231	9918	9595	9264	8925	8579	8229	813	14,2
813	16	13636	13636	13636	13636	13636	13370	13039	12707	12369	12024	11670	11307	10932	10548	10154	9752	9346	813	16
914	10	8674	8674	8674	8674	8674	8674	8502	8327	8151	7972	7791	7607	7418	7226	7028	6826	6619	914	10
914	12,5	11341	11341	11341	11341	11341	11341	11079	10843	10606	10366	10122	9873	9618	9357	9089	8815	8536	914	12,5
914	14,2	13163	13163	13163	13163	13163	13163	12836	12559	12280	11997	11709	11415	11114	10806	10489	10166	9835	914	14,2
914	16	15096	15096	15096	15096	15096	15021	14699	14377	14053	13724	13389	13047	12697	12337	11969	11592	11207	914	16
1016	10	9403	9403	9403	9403	9403	9403	9346	9177	9008	8839	8668	8494	8318	8138	7955	7768	7577	1016	10
1016	12,5	12357	12357	12357	12357	12357	12357	12242	12014	11786	11556	11323	11088	10849	10604	10354	10099	9838	1016	12,5
1016	14,2	14376	14376	14376	14376	14376	14376	14220	13951	13681	13410	13135	12856	12572	12283	11986	11684	11374	1016	14,2
1016	16	16521	16521	16521	16521	16521	16521	16319	16005	15691	15375	15055	14730	14398	14060	13714	13360	12999	1016	16
1219	10	10728	10728	10728	10728	10728	10728	10728	10728	10569	10413	10257	10100	9942	9782	9621	9457	9291	1219	10
1219	12,5	14245	14245	14245	14245	14245	14245	14245	14245	13978	13764	13549	13333	13115	12895	12672	12446	12216	1219	12,5
1219	14,2	16654	16654	16654	16654	16654	16654	16654	16563	16308	16054	15798	15541	15282	15020	14754	14484	14209	1219	14,2
1219	16	19215	19215	19215	19215	19215	19215	19215	19081	18783	18485	18186	17885	17581	17274	16962	16644	16322	1219	16

Annex 11.3 Calculation tables for truss joints

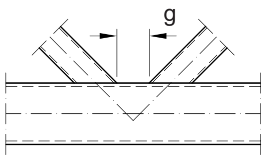
This Annex includes formulae for the most common joint types of uniplanar lattice structures, based mainly on Eurocode Part EN 1993-1-8 [1,2,3]. The Tables also include values from [4,5]. Additional joint types may be found from the given References.

Table	Joint type	Chord	Brace member
11.3.1	T, Y and X joint	Square Rectangular	Circular Square Rectangular
11.3.2	Gap N, K and KT joint	Square Rectangular	Circular Square Rectangular
11.3.3	Overlap N, K and KT joint	Square Rectangular	Circular Square Rectangular
11.3.4	T, Y and X joint	Circular	Circular
11.3.5	Gap N, K and KT joint	Circular	Circular
11.3.6	Overlap N, K and KT joint	Circular	Circular
11.3.7	Gap N and K joint and T, Y and X joint	I-section	Circular Square Rectangular
11.3.8	Overlap N and K joint	I-section	Circular Square Rectangular
11.3.9	T joint bending resistance	I-section	Square Rectangular
11.3.10	T joint bending resistance	Square Rectangular	I-section
11.3.11	T and X joint bending resistance	Square Rectangular	Square Rectangular
11.3.12	T, Y and X joint bending resistance	Circular	Circular
11.3.13	T and X joint bending resistance	Circular	Square Rectangular
11.3.14	Knee joint bending resistance	Square Rectangular	Circular Square Rectangular
11.3.15	Plate joint	Square Rectangular	Plate
11.3.16	Plate joint	Circular	Plate
11.3.17	T, Y and X joint with reinforced chord face	Square Rectangular	Circular Square Rectangular
11.3.18	T, Y and X joint with reinforced chord web	Square Rectangular	Circular Square Rectangular
11.3.19	Gap N, K and KT joint with reinforced chord face	Square Rectangular	Circular Square Rectangular
11.3.20	Gap N, K and KT joint with reinforced chord web	Square Rectangular	Circular Square Rectangular
11.3.21	Overlap N, K and KT joint, reinforced with intermediate plate	Square Rectangular	Circular Square Rectangular

Symbols

A_i	is the cross-section area of the brace member
A_{vp}	is the shear area of the reinforcing plate at chord web
A_{v0}	is the shear area of the chord
A_0	is the cross-section area of the chord
E	is the Young's modulus of elasticity for steel
$M_{ip.i.Ed}$	is the design value of the in-plane bending moment in brace member
$M_{ip.i.Rd}$	is the design value of the joint's bending resistance for the in-plane bending moment in brace member
$M_{op.i.Ed}$	is the design value of the out-of-plane bending moment in brace member
$M_{op.i.Rd}$	is the design value of the joint's bending resistance for the out-of-plane bending moment in brace member
$M_{pl.Rd}$	is the design plastic bending resistance of the cross-section
$M_{0.Ed}$	is the design bending moment in chord
N_{Ed}	is the design normal force in the cross-section
$N_{i.Ed}$	is the design normal force in the brace member
$N_{i.Rd}$	is the design value of the joint's normal force resistance
$N_{p.Ed}$	is the design value of the smaller of the chord normal force (left / right) absolute values at the joint (this value is used for circular chord)
$N_{pl.Rd}$	is the design plastic normal force resistance of the cross-section
$N_{0.Ed}$	is the design value of the bigger of the chord normal force (left / right) absolute values at the joint (this value is used for square and rectangular chord)
$N_{0.gap.Rd}$	is the design normal force resistance of the chord at joint's gap region (this value is used for square and rectangular chord)
V_{Ed}	is the design shear force in the cross-section
$V_{pl.Rd}$	is the design plastic shear resistance of the cross-section
$W_{el.i}$	is the elastic section modulus of the brace member
$W_{el.0}$	is the elastic section modulus of the chord
$W_{pl.i}$	is the plastic section modulus of the brace member
$W_{pl.0}$	is the plastic section modulus of the chord
b_i	is the width of the brace member
b_0	is the width of the chord
b_{eff}	is the effective width for a brace member to chord connection
$b_{e.ov}$	is the effective width for an overlapping brace (ie. the brace located on top) to overlapped brace (ie. the brace located underneath) connection
$b_{e.p}$	is the effective width when calculating punching shear of the chord face
b_p	is the width of the reinforcing plate
b_w	is the effective width of the chord web with an I-section chord
d_i	is the diameter of the circular brace member
d_0	is the diameter of the circular chord
e	is the eccentricity of the joint
f_b	is the buckling strength of the chord web
f_{yi}	is the nominal yield strength of the brace member
f_{yk}	is the chord web reduced yield strength for some joints
f_{y0}	is the nominal yield strength of the chord

g	is the gap of the joint



h_i	is the depth of the brace member
h_w	is the depth of the I-section web
h_0	is the depth of the chord (used for hollow section and I-section)
i	is an integer subscript to designate a brace member (1,2,3)
k_g	is the reduction factor for the resistance of gap and overlap joints with circular chord
k_m	is the reduction factor for the resistance of some joints with plate or I-section
k_n	is the reduction factor for the resistance of a joint where all members are hollow sections, and chord is square or rectangular hollow section
k_p	is the reduction factor for the resistance of a joint with circular chord
m	is the amount of the brace members at the joint
n	is the ratio of the compression stress to design yield stress (due to $N_{0.Ed}$ and $M_{0.Ed}$) with square and rectangular chords
n_p	is the ratio of the compression stress to design yield stress (due to $N_{p.Ed}$ and $M_{0.Ed}$) with circular chords
p_{eff}	is the effective width for a brace member to chord connection with I-section chord
r	is the root radius between flange and web of the rolled I-section (with welded I-sections radius *r* shall be replaced with $\sqrt{2}a$, where *a* is the throat thickness of the weld between flange and web)
t_f	is the flange thickness of the I-section
t_i	is the wall thickness of the brace member
t_0	is the wall thickness of the chord
t_p	is the thickness of the reinforcing plate
t_w	is the web thickness of the I-section
α	is a parameter for calculating the chord shear area at the joint
β	is the ratio of the brace member diameter or width to the chord diameter or width. The brace member size is taken as an average if the joint has several brace members:

T, Y and X joint:

$$\beta = \frac{d_1}{d_0} \; ; \; \frac{d_1}{b_0} \text{ tai } \frac{b_1}{b_0}$$

N, K and KT joint:

$$\beta = \frac{\sum_{i=1}^{m} b_i + \sum_{i=1}^{m} h_i}{2m \cdot b_0}$$

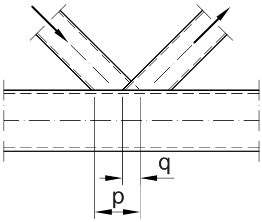
where *m* is the amount of the brace members at the joint:

N and K joint: m = 2

KT joint: m = 3

β_p	is the ratio of the the brace member width to the width of the reinforcing plate (this parameter is used for the joints having the reinforcing plate) $\beta_p = b_i / b_p$
γ	is the ratio of the chord width to twice its wall thickness $\gamma = 0{,}5\, d_0 / t_0$ or $= 0{,}5\, b_0 / t_0$ or $= 0{,}5\, b_0 / t_f$
γ_{M0}	is the partial safety factor for the resistance of the member
γ_{M5}	is the partial safety factor for the resistance of the lattice structure joint
η	is the ratio of the brace member depth to chord width $\eta = h_i / b_0$
η_p	is the ratio of the brace member depth to reinforcing plate width (this parameter is used for the joints having the reinforcing plate) $\eta_p = h_i / b_p$
κ	is a parameter for the resistance of the knee joint, when the joint angle is θ
κ_{90}	is a parameter for the resistance of the knee joint, when the joint angle is $\theta = 90°$
$\bar{\lambda}$	is the non-dimensional slenderness of the chord web for flexural buckling
λ_{ov}	is the overlap ratio of the overlap joint

$$\lambda_{ov} = \frac{q}{p} \cdot 100\ \% = \frac{q \cdot \sin\theta_i}{h_i} \cdot 100\ \%$$



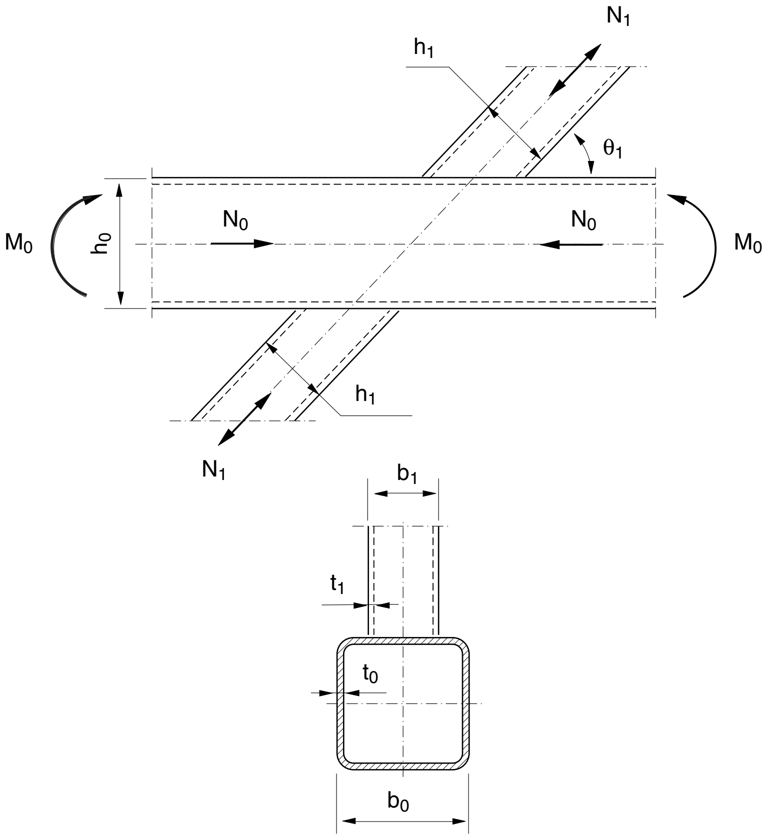
$\lambda_{ov.lim}$	is the maximum limit for the overlap ratio of the overlap joint
θ_i	is the smaller angle between the brace member and the chord

In all tables of this Annex, the following limitations apply
(unless otherwise stated in any particular Table):

- $t_0 \geq 2{,}5$ mm
- $t_i \geq 2{,}5$ mm
- $\theta_i \geq 30°$ (also for the angle between the brace members)
- $-0{,}55 \geq e/h_0 \leq 0{,}25$ or $-0{,}55 \leq e/d_0 \leq 0{,}25$
- $L_i \geq 6h_i$ tai $6d_i$
- the cross-section of the chords must be in Class 1 or 2 (based on pure axial compression)
- the cross-section of the compressed brace member must be in Class 1 or 2 (based on pure axial compression)
- the brace member ends must not be flattened

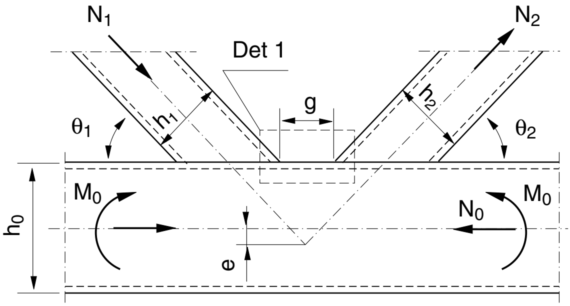
Depending on the joint type, the tables may include additional requirements.

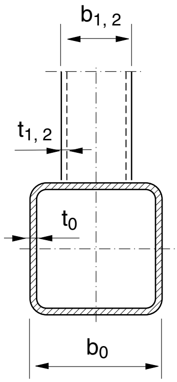
Table 11.3.1 Resistance of T, Y and X joints [1...5]:
- chords are square or rectangular hollow sections
- brace members are square, rectangular or circular hollow sections



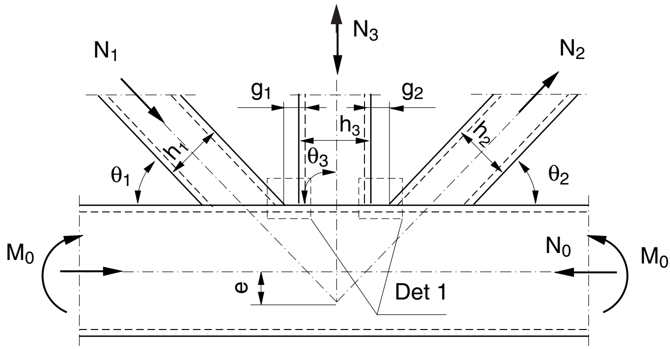
Resistance [i = 1]	Parameters	Validity range
$\beta \leq 0{,}85$, Chord face failure		In general: $30^\circ \leq \theta_i \leq 90^\circ$ $t_0 \geq 2{,}5$ mm $t_i \geq 2{,}5$ mm $L_i \geq 6h_i$ or $6d_i$ Square and rectangular brace members: - in general: $b_i / b_0 \geq 0{,}25$ $b_i / t_i \leq 35$ $h_i / t_i \leq 35$ $0{,}5 \leq h_i / b_i \leq 2{,}0$ - compression brace member: Class 1 or 2 (based on pure axial compression) Circular brace members: - in general: $0{,}4 \leq d_i / b_0 \leq 0{,}8$ - tension brace member: $d_i / t_i \leq 50$ - compression brace member: Class 1 (based on pure axial compression) Chords: $b_0 / t_0 \leq 35$ $h_0 / t_0 \leq 35$ $0{,}5 \leq h_0 / b_0 \leq 2{,}0$ Class 1 or 2 (based on pure axial compression)
$N_{i.Rd} = \dfrac{k_n \cdot f_{y0} \cdot t_0^2}{(1-\beta)\sin\theta_i} \cdot \left(\dfrac{2\eta}{\sin\theta_i} + 4\sqrt{1-\beta}\right) / \gamma_{M5}$	$\beta = b_i / b_0$ $\eta = h_i / b_0$ Tension chord: $k_n = 1{,}0$ Compression chord: $k_n = 1{,}3 - \dfrac{0{,}4\lvert n\rvert}{\beta} \leq 1{,}0$ $n = \dfrac{N_{0.Ed}}{A_0 \cdot f_{y0}/\gamma_{M5}} + \dfrac{M_{0.Ed}}{W_{el.0} \cdot f_{y0}/\gamma_{M5}}$	
$0{,}85 < \beta < 1{,}0$		
Use linear interpolation between the following values: - chord face failure when $\beta = 0{,}85$ - the critical of following values: chord side wall buckling/yielding when $\beta = 1{,}0$, or Table 11.3.2: chord shear		
$\beta = 1{,}0$, Chord side wall buckling or yielding [a)]		
$N_{i.Rd} = \dfrac{k_n \cdot f_b \cdot t_0}{\sin\theta_i} \cdot \left(\dfrac{2h_i}{\sin\theta_i} + 10t_0\right) / \gamma_{M5}$	Tension chord: $f_b = f_{y0}$ Compression chord: $f_b = \chi \cdot f_{y0}$ (T and Y joint) $f_b = 0{,}8\chi \cdot f_{y0} \cdot \sin\theta_i$ (X joint) χ = reduction factor for flexural buckling using buckling curve c and non-dimensional slenderness determined from: $\bar{\lambda} = 3{,}46 \cdot \dfrac{\left(\dfrac{h_0}{t_0} - 2\right) \cdot \sqrt{\dfrac{1}{\sin\theta_i}}}{\pi \cdot \sqrt{\dfrac{E}{f_{y0}}}}$	
$0{,}85 \leq \beta \leq (1 - 1/\gamma)$, Chord face punching shear		
$N_{i.Rd} = \dfrac{f_{y0} \cdot t_0}{\sqrt{3}\sin\theta_i} \cdot \left(\dfrac{2h_i}{\sin\theta_i} + 2b_{e.p}\right) / \gamma_{M5}$	$b_{e.p} = \dfrac{10}{b_0/t_0} b_i \leq b_i$ $\gamma = \dfrac{b_0}{2t_0}$	
$\beta \geq 0{,}85$, Brace failure		
$N_{i.Rd} = f_{yi} \cdot t_i \cdot (2h_i - 4t_i + 2b_{eff}) / \gamma_{M5}$	$b_{eff} = \dfrac{10}{b_0/t_0} \cdot \dfrac{f_{y0} \cdot t_0}{f_{yi} \cdot t_i} \cdot b_i \leq b_i$	
If circular brace members, multiply above resistance values by $\pi/4$ and replace b_i and h_i with the diameter d_i .		
a) For X joint with $\cos\theta_i > h_i / h_0$ use the smaller of following values: this value or Table 11.3.2: gap N and K joint: chord shear.		
Reduction factor for above resistances:	S235 - S355: 1,0 S420 - S460: 0,9	

Table 11.3.2 Resistance of gap N, K and KT joints [1...5]:
- chords are square or rectangular hollow sections
- brace members are square, rectangular or circular hollow sections





N_1 always compression
N_2 always tension
N_0 the bigger of chord normal force (left/right) absolute values



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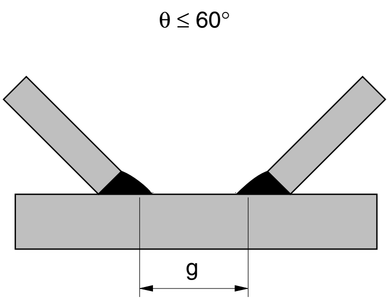
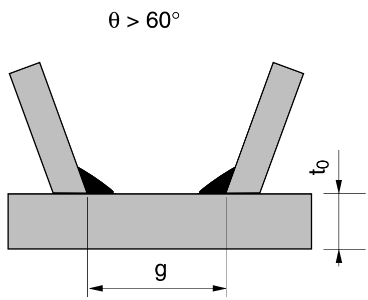
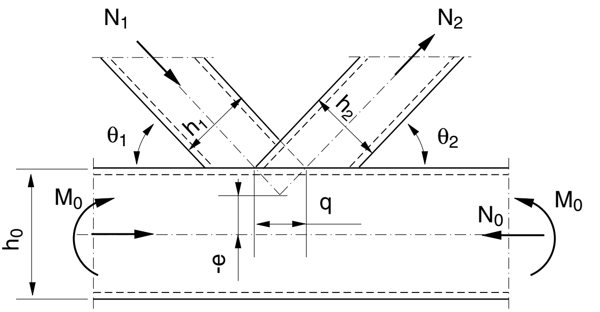
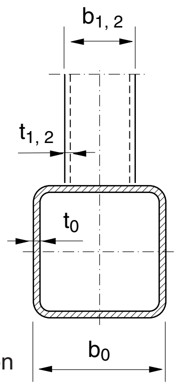




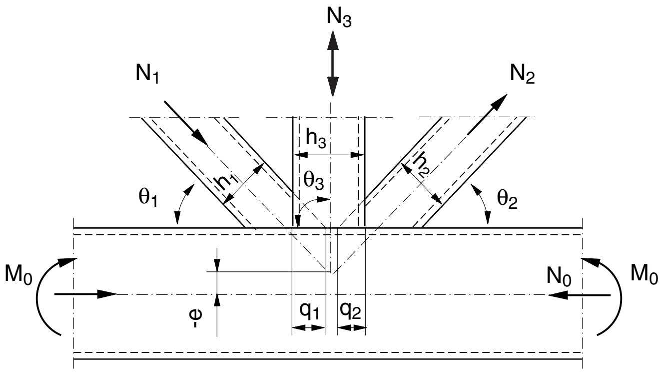
Resistance [i = 1 or 2]	Parameters	Validity range
N and K joint: Chord face failure		In general: $30^\circ \le \theta_i \le 90^\circ$ $t_0 \ge 2{,}5$ mm $t_i \ge 2{,}5$ mm $L_i \ge 6h_i$ or $6d_i$ $-0{,}55 \le e/h_0 \le 0{,}25$ Square and rectangular brace members: - in general: $b_i/b_0 \ge 0{,}35$ $b_i/b_0 \ge 0{,}1+0{,}01b_0/t_0$ $0{,}5 \le h_i/b_i \le 2{,}0$ $b_i/t_i \le 35$ $h_i/t_i \le 35$ - compression brace member: Class 1 or 2 (based on pure axial compression) Circular brace members: - in general: $0{,}4 \le d_i/b_0 \le 0{,}8$ - tension brace member: $d_i/t_i \le 50$ - compression brace member: Class 1 (based on pure axial compression) Chords: $b_0/t_0 \le 35$ $h_0/t_0 \le 35$ $0{,}5 \le h_0/b_0 \le 2{,}0$ Class 1 or 2 (based on pure axial compression) Gap: $g \ge t_1 + t_2$ $g/b_0 \ge 0{,}5(1-\beta)$ $g/b_0 \le 1{,}5(1-\beta)$ a) If KT joint, check above conditions for both gaps separately.
$N_{i.Rd} = \dfrac{8{,}9 \cdot k_n \cdot f_{y0} \cdot t_0^2 \cdot \sqrt{\gamma}}{\sin\theta_i} \cdot \left(\dfrac{\sum_{i=1}^{m} b_i + \sum_{i=1}^{m} h_i}{2m \cdot b_0} \right) / \gamma_{M5}$	$\beta = \dfrac{\sum_{i=1}^{m} b_i + \sum_{i=1}^{m} h_i}{2m \cdot b_0}$ m is the amount of brace members $\gamma = 0{,}5\, b_0/t_0$ Tension chord: $k_n = 1{,}0$ Compression chord: $k_n = 1{,}3 - \dfrac{0{,}4\lvert n\rvert}{\beta} \le 1{,}0$ $n = \dfrac{N_{0.Ed}}{A_0 \cdot f_{y0}/\gamma_{M5}} + \dfrac{M_{0.Ed}}{W_{el.0} \cdot f_{y0}/\gamma_{M5}}$	
N and K joint: Chord shear		
$N_{i.Rd} = \dfrac{f_{y0} \cdot A_{v0}}{\sqrt{3}\sin\theta_i} / \gamma_{M5}$ $N_{0.gap.Rd} = \dfrac{\left((A_0 - A_{v0}) f_{y0} + A_{v0} f_{y0} \sqrt{1 - \left(\dfrac{V_{Ed}}{V_{pl.Rd}} \right)^2} \right)}{\gamma_{M5}}$	$A_{v0} = (2h_0 + \alpha b_0) t_0$ $\alpha = \sqrt{\dfrac{1}{1 + \dfrac{4g^2}{3t_0^2}}}$ $\alpha = 0$ for circular brace members V_{Ed} is chord shear at gap area $V_{pl.Rd} = \dfrac{f_{y0} \cdot A_{v0}}{\sqrt{3} \cdot \gamma_{M5}}$	
N and K joint: $\beta \le (1 - 1/\gamma)$, Chord face punching shear		
$N_{i.Rd} = \dfrac{f_{y0} \cdot t_0}{\sqrt{3}\sin\theta_i} \cdot \left(\dfrac{2h_i}{\sin\theta_i} + b_i + b_{e.p} \right) / \gamma_{M5}$	$b_{e.p} = \dfrac{10}{b_0/t_0} b_i \le b_i$	
N and K joint: Brace failure		
$N_{i.Rd} = f_{yi}\, t_i\, (2h_i - 4t_i + b_i + b_{eff}) / \gamma_{M5}$	$b_{eff} = \dfrac{10}{b_0/t_0} \cdot \dfrac{f_{y0} \cdot t_0}{f_{yi} \cdot t_i} \cdot b_i \le b_i$	
If circular brace members, multiply above resistance values by $\pi/4$ and replace b_i and h_i with the diameter d_i.		
KT joint: - For each brace member, check following condition: $N_{i.Rd} \ge N_{i.Ed}$ (i = 1,2,3) - Resistances for the brace members are calculated as for N joint using adjacent brace members in pairs as follows: A) brace members 1 & 3 B) brace members 2 & 3 If brace member 3 is in tension, its resistance is calculated from Case A. If brace member 3 is in compression, its resistance is calculated from Case B.		
a) If $g/b_0 > 1{,}5(1-\beta)$ and $g \ge t_1 + t_2$ treat the joint as two separate T or Y joint.		
Reduction factor for above resistances:	S235 - S355: 1,0 S420 - S460: 0,9	

Table 11.3.3 Resistance of overlap N, K and KT joints [1...5]:
- chords are square or rectangular hollow sections
- brace members are square, rectangular or circular hollow sections





N_1 , N_2 one brace in compression, one brace in tension
N_1 overlapping brace member (located on top)
N_2 overlapped brace member (located underneath)
N_0 the bigger of chord normal force (left/right) absolute values



N_1 , N_2 one brace in compression, one brace in tension
N_1 , N_2 overlapping brace members (located on top)
N_3 overlapped brace member (located underneath)
N_0 the bigger of chord normal force (left/right) absolute values

<table>
<tr><td>Resistance [i = 1 or 2]</td><td>Parameters</td><td>Validity range</td></tr>
<tr><td colspan="2">25 % ≤ λ_{ov} < 50 % , Brace failure</td><td rowspan="10">In general:
$30^\circ \le \theta_i \le 90^\circ$
$t_0 \ge 2{,}5$ mm
$t_i \ge 2{,}5$ mm
$L_i \ge 6h_i$ or $6d_i$
$-0{,}55 \le e/h_0 \le 0{,}25$

Square and rectangular brace members:
- in general:
$b_i/b_0 \ge 0{,}25$
$0{,}5 \le h_i/b_i \le 2{,}0$
- tension brace member:
$b_i/t_i \le 35$
$h_i/t_i \le 35$
- compression brace member:
Class 1
(based on pure axial compression)
- N and K joint:
$0{,}75 \le b_1/b_2 \le 1{,}0$
$(t_1 f_{y1}) / (t_2 f_{y2}) \le 1{,}0$
- KT joint:
$0{,}75 \le b_1/b_3 \le 1{,}0$
$(t_1 f_{y1}) / (t_3 f_{y3}) \le 1{,}0$
$0{,}75 \le b_2/b_3 \le 1{,}0$
$(t_2 f_{y2}) / (t_3 f_{y3}) \le 1{,}0$

Circular brace members:
- in general:
$0{,}4 \le d_i/b_0 \le 0{,}8$
- tension brace member:
$d_i/t_i \le 50$
- compression brace member:
Class 1
(based on pure axial compression)
- N and K joint:
$0{,}75 \le d_1/d_2 \le 1{,}0$
$(t_1 f_{y1}) / (t_2 f_{y2}) \le 1{,}0$
- KT joint:
$0{,}75 \le d_1/d_3 \le 1{,}0$
$(t_1 f_{y1}) / (t_3 f_{y3}) \le 1{,}0$
$0{,}75 \le d_2/d_3 \le 1{,}0$
$(t_2 f_{y2}) / (t_3 f_{y3}) \le 1{,}0$

Chords:
$b_0/t_0 \le 35$
$h_0/t_0 \le 35$
$0{,}5 \le h_0/b_0 \le 2{,}0$
Class 1 or 2
(based on pure axial compression)

Overlap:
$25\ \% \le \lambda_{ov} \le \lambda_{ov.lim}$ a)</td></tr>
<tr><td>N and K joint:
$N_{1.Rd} = f_{y1} t_1 \left(b_{eff} + b_{e.ov} + \frac{\lambda_{ov}}{50} \cdot 2h_1 - 4t_1\right) / \gamma_{M5}$

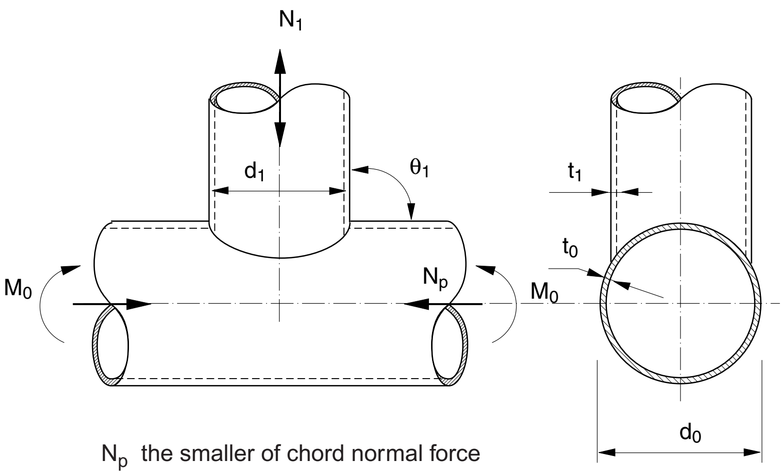
KT joint:
$N_{i.Rd} = f_{yi} t_i \left(b_{eff} + b_{e.ov} + \frac{\lambda_{ov}}{50} \cdot 2h_i - 4t_i\right) / \gamma_{M5}$
(i = 1,2)</td><td>N and K joint:
$\lambda_{ov} = \frac{q \cdot \sin\theta_1}{h_1} \cdot 100\ \%$
$b_{eff} = \frac{10}{b_0/t_0} \cdot \frac{f_{y0} \cdot t_0}{f_{y1} \cdot t_1} \cdot b_1 \le b_1$
$b_{e.ov} = \frac{10}{b_2/t_2} \cdot \frac{f_{y2} \cdot t_2}{f_{y1} \cdot t_1} \cdot b_1 \le b_1$

KT joint:
$\lambda_{ov} = \frac{q \cdot \sin\theta_i}{h_i}$
$b_{eff} = \frac{10}{b_0/t_0} \cdot \frac{f_{y0} \cdot t_0}{f_{yi} \cdot t_i} \cdot b_i \le b_i$
$b_{e.ov} = \frac{10}{b_3/t_3} \cdot \frac{f_{y3} \cdot t_3}{f_{yi} \cdot t_i} \cdot b_i \le b_i$
(i = 1,2)</td></tr>
<tr><td colspan="2">50 % ≤ λ_{ov} < 80 % , Brace failure</td></tr>
<tr><td>N and K joint:
$N_{1.Rd} = f_{y1} t_1 \cdot (b_{eff} + b_{e.ov} + 2h_1 - 4t_1) / \gamma_{M5}$

KT joint:
$N_{i.Rd} = f_{yi} t_i \cdot (b_{eff} + b_{e.ov} + 2h_i - 4t_i) / \gamma_{M5}$
(i = 1,2)</td><td></td></tr>
<tr><td colspan="2">$\lambda_{ov} \ge 80$ % , Brace failure</td></tr>
<tr><td>N and K joint:
$N_{1.Rd} = f_{y1} t_1 \cdot (b_1 + b_{e.ov} + 2h_1 - 4t_1) / \gamma_{M5}$

KT joint:
$N_{i.Rd} = f_{yi} t_i \cdot (b_i + b_{e.ov} + 2h_i - 4t_i) / \gamma_{M5}$
(i = 1,2)</td><td></td></tr>
<tr><td colspan="2">If circular brace members, multiply above resistance values by $\pi/4$ and replace b_i and h_i with the diameter d_i .</td></tr>
<tr><td colspan="2">Brace 1 (= overlapping brace): resistance is determined acc. to above formulas.
Brace 2 (= overlapped brace): resistance in N/K/KT joints is determined by using joint efficiency (ie. the design resistance of the joint divided by the plastic resistance of the brace member) which should be taken as equal to joint efficiency of the overlapping brace member.</td></tr>
<tr><td colspan="2">a) If $\lambda_{ov} > \lambda_{ov.lim}$ <u>or</u> if the braces are rectangular sections with $h_1 < b_1$ and/or $h_2 < b_2$, the connection between braces and chord has to be checked for shear as given in clause 7.4.7:
$\lambda_{ov.lim}$ = 60 % if the hidden seam of the overlapped brace is not welded to the chord
$\lambda_{ov.lim}$ = 80 % if the hidden seam of the overlapped brace is welded to the chord.</td></tr>
<tr><td>Reduction factor for above resistances:</td><td>S235 - S355: 1,0
S420 - S460: 0,9</td></tr>
</table>

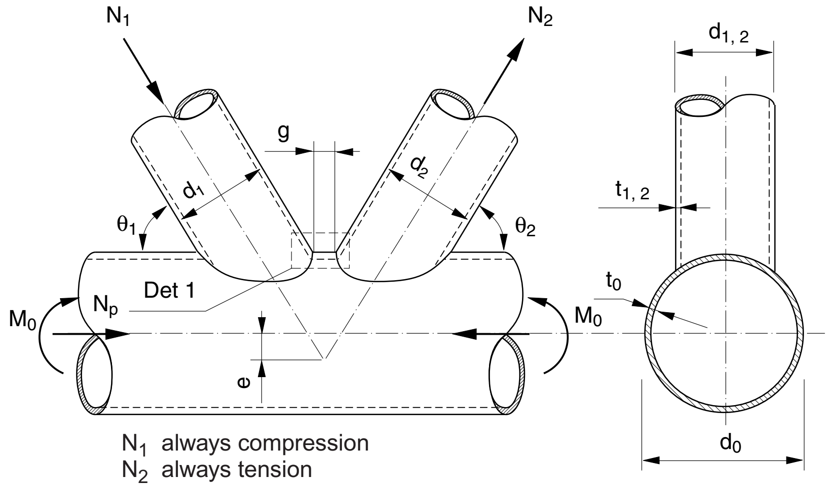
Table 11.3.4 Resistance of T, Y and X joints [1...5]:
- chords are circular hollow sections
- brace members are circular hollow sections



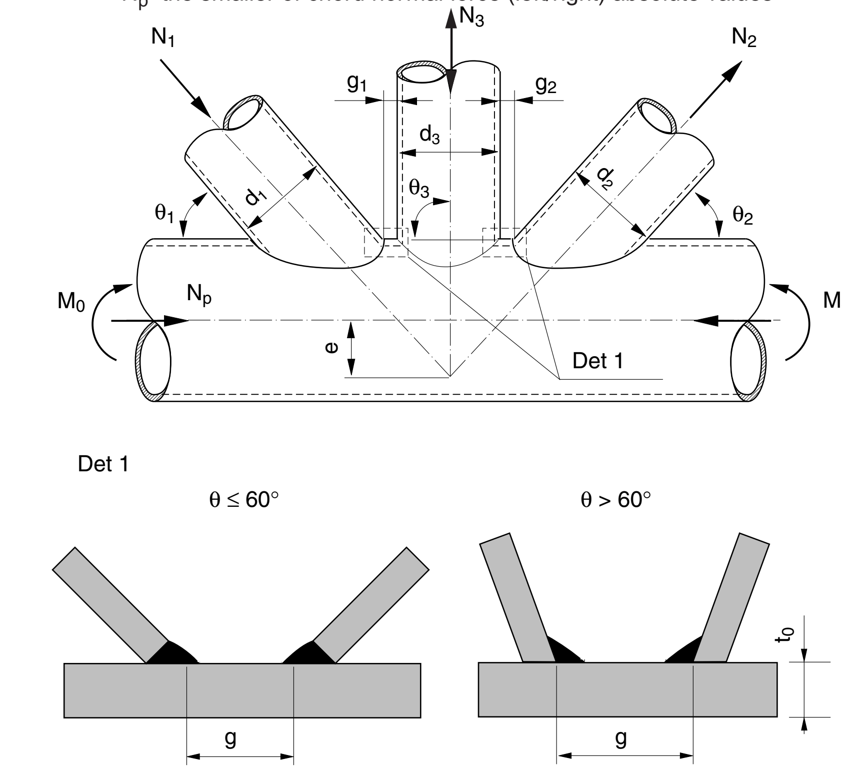
N_p the smaller of chord normal force (left/right) absolute values

Resistance [i = 1]	Parameters	Validity range
T and Y joint: Chord face failure		In general: $30^\circ \le \theta_i \le 90^\circ$ $t_0 \ge 2{,}5$ mm $t_i \ge 2{,}5$ mm $L_i \ge 6 d_i$ Brace members: - in general: $0{,}2 \le d_i / d_0 \le 1{,}0$ - tension brace member: $d_i / t_i \le 50$ - compression brace member: Class 1 or 2 (based on pure axial compression) Chords: - T and Y joint: $10 \le d_0 / t_0 \le 50$ - X joint: $10 \le d_0 / t_0 \le 40$ - compression chord: Class 1 or 2 (based on pure axial compression)
$N_{i.Rd} = \frac{\gamma^{0,2} \cdot k_p \cdot f_{y0} \cdot t_0^2}{\sin\theta_i}(2,8 + 14,2\beta^2)/\gamma_{M5}$	$\beta = d_i / d_0$ $\gamma = 0,5\, d_0 / t_0$ Tension chord: $k_p = 1,0$ Compression chord: $k_p = 1,0 - 0,3 \lvert n_p \rvert - 0,3 n_p^2 \le 1,0$ $n_p = \frac{N_{p.Ed}}{A_0 \cdot f_{y0}/\gamma_{M5}} + \frac{M_{0.Ed}}{W_{el.0} \cdot f_{y0}/\gamma_{M5}}$	
X joint: Chord face failure		
$N_{i.Rd} = \frac{k_p \cdot f_{y0} \cdot t_0^2}{\sin\theta_i} \cdot \frac{5,2}{(1 - 0,81\beta)}/\gamma_{M5}$	$\beta = d_i / d_0$ $\gamma = 0,5\, d_0 / t_0$ Tension chord: $k_p = 1,0$ Compression chord: $k_p = 1,0 - 0,3 \lvert n_p \rvert - 0,3 n_p^2 \le 1,0$ $n_p = \frac{N_{p.Ed}}{A_0 \cdot f_{y0}/\gamma_{M5}} + \frac{M_{0.Ed}}{W_{el.0} \cdot f_{y0}/\gamma_{M5}}$	
T, Y and X joint: $d_i \le d_0 - 2t_0$, Chord face punching shear		
$N_{i.Rd} = \frac{f_{y0}}{\sqrt{3}} \cdot t_0 \cdot \pi d_i \cdot \frac{1 + \sin\theta_i}{2 \sin^2\theta_i}/\gamma_{M5}$		
Reduction factor for above resistances:	S235 - S355: 1,0 S420 - S460: 0,9	

Table 11.3.5 Resistance of gap N, K and KT joints [1...5]:
- chords are circular hollow sections
- brace members are circular hollow sections

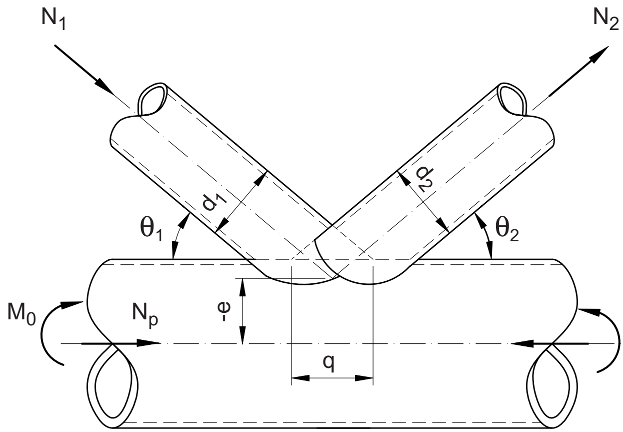


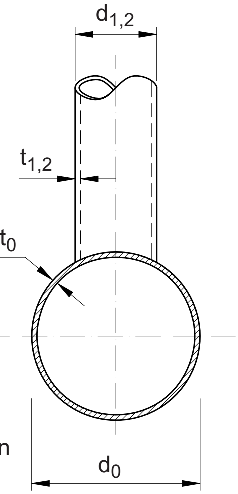
N_1 always compression
N_2 always tension
N_p the smaller of chord normal force (left/right) absolute values



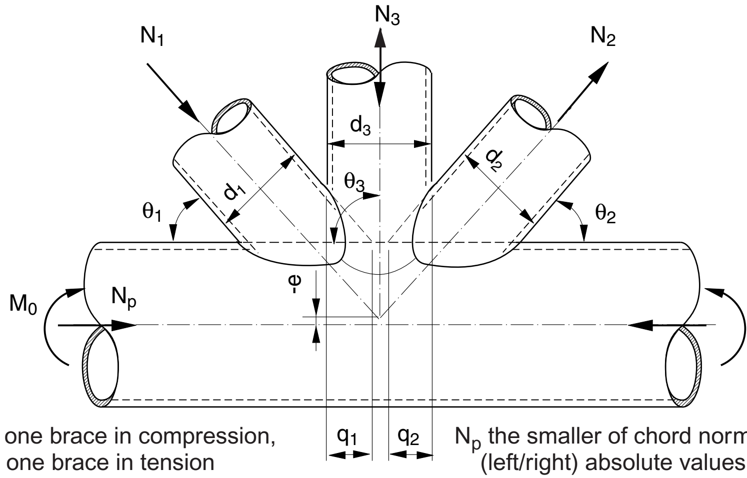
Resistance [i = 1 or 2]	Parameters	Validity range
N and K joint: Chord face failure		In general: $30^\circ \le \theta_i \le 90^\circ$ $t_0 \ge 2{,}5$ mm $t_i \ge 2{,}5$ mm $L_i \ge 6 d_i$ $-0{,}55 \le e / d_0 \le 0{,}25$
Brace 1 (= compression brace): $N_{1.Rd} = \frac{k_g \cdot k_p \cdot f_{y0} \cdot t_0^2}{\sin\theta_1} \cdot \left(1{,}8 + 10{,}2 \frac{d_1}{d_0}\right) / \gamma_{M5}$ Brace 2 (= tension brace): $N_{2.Rd} = \frac{\sin\theta_1}{\sin\theta_2} \cdot N_{1.Rd}$	$k_g = \gamma^{0,2} \cdot \left(1 + \frac{0{,}024 \cdot \gamma^{1,2}}{1 + e^{\left(\frac{g}{2t_0} - 1{,}33\right)}}\right)$ $\gamma = 0{,}5\, d_0 / t_0$ Tension chord: $k_p = 1{,}0$ Compression chord: $k_p = 1{,}0 - 0{,}3 \lvert n_p \rvert - 0{,}3 n_p^2 \le 1{,}0$ $n_p = \frac{N_{p.Ed}}{A_0 \cdot f_{y0} / \gamma_{M5}} + \frac{M_{0.Ed}}{W_{el.0} \cdot f_{y0} / \gamma_{M5}}$	Brace members: - in general: $0{,}2 \le d_i / d_0 \le 1{,}0$ - tension brace member: $d_i / t_i \le 50$ - compression brace member: Class 1 or 2 (based on pure axial compression)
N and K joint: $d_i \le d_0 - 2t_0$, Chord face punching shear		Chords: - in general: $10 \le d_0 / t_0 \le 50$ - compression chord: Class 1 or 2 (based on pure axial compression)
$N_{i.Rd} = \frac{f_{y0}}{\sqrt{3}} \cdot t_0 \cdot \pi\, d_i \cdot \frac{1 + \sin\theta_i}{2 \sin^2\theta_i} / \gamma_{M5}$		
KT joint: - For each brace member, check following condition: $N_{i.Rd} \ge N_{i.Ed}$ (i = 1,2,3) - Resistances for the brace members are calculated as for N joint using adjacent brace members in pairs as follows: A) brace members 1 & 3 B) brace members 2 & 3 If brace member 3 is in tension, its resistance is calculated from Case A. If brace member 3 is in compression, its resistance is calculated from Case B.		Gap: $g \ge t_1 + t_2$
Reduction factor for above resistances:	S235 - S355: 1,0 S420 - S460: 0,9	

Table 11.3.6 Resistance of overlap N, K and KT joints [1...5]:
- chords are circular hollow sections
- brace members are circular hollow sections





N_1 , N_2 one brace in compression, one brace in tension
N_1 overlapped brace member (located underneath)
N_2 overlapping brace member (located on top)
N_p the smaller of chord normal force (left/right) absolute values



N_1 , N_2 one brace in compression, one brace in tension
N_p the smaller of chord normal force (left/right) absolute values
N_1 , N_2 overlapping brace member (located on top)
N_3 overlapped brace member (located underneath)

<table>
<tr><th>Resistance [i = 1 or 2]</th><th>Parameters</th><th>Validity range</th></tr>
<tr><td colspan="2">Chord face failure</td><td rowspan="4">In general:
$30^\circ \le \theta_i \le 90^\circ$
$t_0 \ge 2,5$ mm
$t_i \ge 2,5$ mm
$L_i \ge 6 d_i$
$-0,55 \le e/d_0 \le 0,25$

Brace members:
- in general:
$0,2 \le d_i/d_0 \le 1,0$
- tension brace member:
$d_i/t_i \le 50$
- compression brace member:
Class 1 or 2
(based on pure axial compression)
- N and K joint:
$0,75 \le d_1/d_2 \le 1,0$
$(t_1 f_{y1}) / (t_2 f_{y2}) \le 1,0$
- KT joint:
$0,75 \le d_1/d_3 \le 1,0$
$(t_1 f_{y1}) / (t_3 f_{y3}) \le 1,0$
$0,75 \le d_2/d_3 \le 1,0$
$(t_2 f_{y2}) / (t_3 f_{y3}) \le 1,0$

Chords:
- in general:
$10 \le d_0/t_0 \le 50$
- compression chord:
Class 1 or 2
(based on pure axial compression)

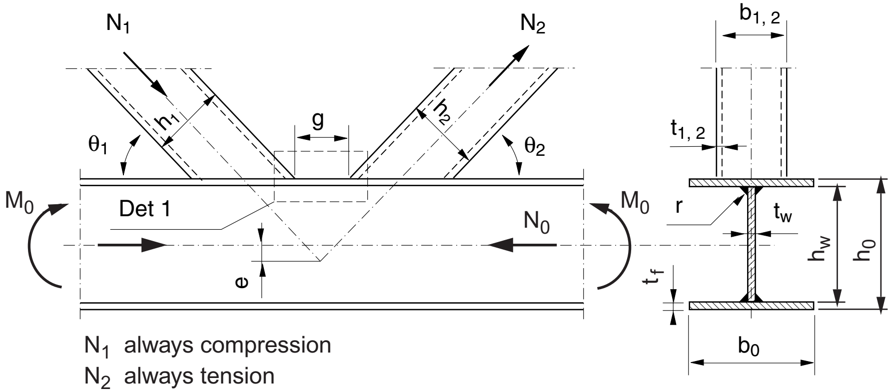
Overlap:
$25\ \% \le \lambda_{ov} \le \lambda_{ov.lim}$ a)</td></tr>
<tr><td>Brace 1 (= overlapped brace):
$N_{1.Rd} = \frac{k_g \cdot k_p \cdot f_{y0} \cdot t_0^2}{\sin\theta_1} \cdot \left(1,8 + 10,2 \frac{d_1}{d_0}\right) / \gamma_{M5}$

Brace 2 (= overlapping brace):
$N_{2.Rd} = \frac{\sin\theta_1}{\sin\theta_2} \cdot N_{1.Rd}$</td><td>$\lambda_{ov} = \frac{q \cdot \sin\theta_2}{d_2} \cdot 100\ \%$
$k_g = \gamma^{0,2} \cdot \left(1 + \frac{0,024 \cdot \gamma^{1,2}}{1 + e^{\left(\frac{-q}{2t_0} - 1,33\right)}}\right)$
$\gamma = 0,5\, d_0/t_0$

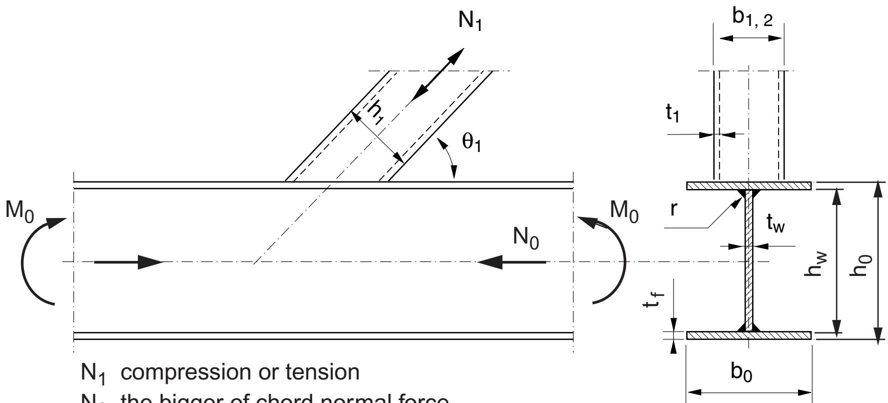
Tension chord:
$k_p = 1,0$

Compression chord:
$k_p = 1,0 - 0,3\lvert n_p \rvert - 0,3 n_p^2 \le 1,0$
$n_p = \frac{N_{p.Ed}}{A_0 \cdot f_{y0}/\gamma_{M5}} + \frac{M_{0.Ed}}{W_{el.0} \cdot f_{y0}/\gamma_{M5}}$</td></tr>
<tr><td colspan="2">KT joint:
- For each brace member, check following condition:
$N_{i.Rd} \ge N_{i.Ed}$ (i = 1,2,3)
- Resistances for the brace members are calculated as for N joint using adjacent brace members in pairs as follows:
A) N-joint, brace members 1 & 3
B) N-joint, brace members 2 & 3
Resistance for brace member 3 (= overlapped brace) is chosen as the smaller of the values calculated from Case A and Case B.</td></tr>
<tr><td colspan="2">a) If $\lambda_{ov} > \lambda_{ov.lim}$ <u>or</u> if the braces are rectangular sections with $h_1 < b_1$ and/or $h_2 < b_2$, the connection between braces and chord has to be checked for shear as given in clause 7.4.7:
$\lambda_{ov.lim} = 60\ \%$ if the hidden seam of the overlapped brace is not welded to the chord
$\lambda_{ov.lim} = 80\ \%$ if the hidden seam of the overlapped brace is welded to the chord.</td></tr>
<tr><td>Reduction factor for above resistances:</td><td colspan="2">S235 - S355: 1,0
S420 - S460: 0,9</td></tr>
</table>

Table 11.3.7 Resistance of T, Y and X joints and gap N and K joints [1...5]:
- chords are I-sections
- brace members are square, rectangular or circular hollow sections

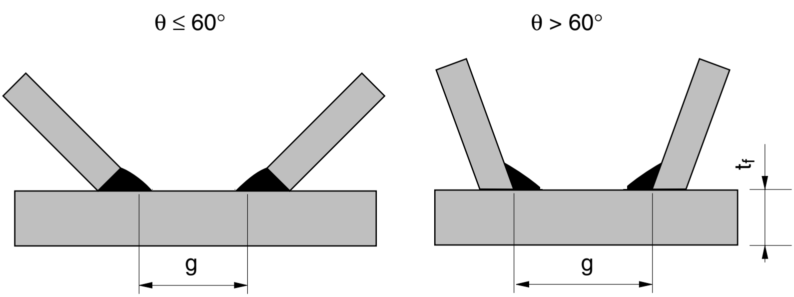


N_1 always compression
N_2 always tension
N_0 the bigger of chord normal force (left/right) absolute values



N_1 compression or tension
N_0 the bigger of chord normal force (left/right) absolute values

Det 1



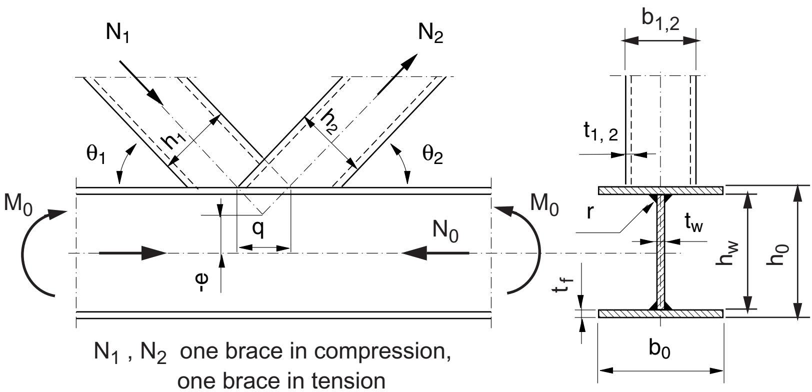
<table>
<tr><th>Resistance [i = 1 or 2]</th><th>Parameters</th><th>Validity range</th></tr>
<tr><td colspan="2">T, Y and X joint: Chord web yielding</td><td rowspan="12">In general:
$30^\circ \le \theta_i \le 90^\circ$
$t_i \ge 2{,}5$ mm
$L_i \ge 6h_i$ or $6d_i$
$-0{,}55 \le e/h_0 \le 0{,}25$

T, Y, N and K joint:
- brace members
in general:
$h_i/b_i = 1{,}0$
$b_i/t_i \le 35$
$h_i/t_i \le 35$
$d_i/t_i \le 50$
- compression
brace member:
Class 1
(based on pure
axial compression)
- chords:
web:
$h_w \le 400$ mm
web and flanges:
Class 1 or 2
(based on pure
axial compression)

X joint:
- brace members
in general:
$0{,}5 \le h_i/b_i \le 2{,}0$
$b_i/t_i \le 35$
$h_i/t_i \le 35$
$d_i/t_i \le 50$
- compression
brace member:
Class 1
(based on pure
axial compression)
- chords:
web:
$h_w \le 400$ mm
Class 1
flanges:
Class 1 or 2
(based on pure
axial compression)

Gap:
$g \ge t_1 + t_2$</td></tr>
<tr><td>$N_{i.Rd} = \dfrac{f_{y0} \cdot t_w \cdot b_w}{\sin\theta_i} / \gamma_{M5}$</td><td>$b_w = (h_i / \sin\theta_i) + 5(t_f + r)$
but: $b_w \le 2t_i + 10(t_f + r)$</td></tr>
<tr><td colspan="2">T, Y and X joint: Brace failure</td></tr>
<tr><td>$N_{i.Rd} = 2 \cdot f_{yi} \cdot t_i \cdot p_{eff} / \gamma_{M5}$</td><td>$p_{eff} = t_w + 2r + 7t_f \cdot f_{y0} / f_{yi}$
but: $p_{eff} \le b_i + h_i - 2t_i$</td></tr>
<tr><td colspan="2">N and K joint: Chord web yielding</td></tr>
<tr><td>$N_{i.Rd} = \dfrac{f_{y0} \cdot t_w \cdot b_w}{\sin\theta_i} / \gamma_{M5}$</td><td>$b_w = (h_i / \sin\theta_i) + 5(t_f + r)$
but: $b_w \le 2t_i + 10(t_f + r)$</td></tr>
<tr><td colspan="2">N and K joint: Brace failure</td></tr>
<tr><td>$N_{i.Rd} = 2 \cdot f_{yi} \cdot t_i \cdot p_{eff} / \gamma_{M5}$</td><td>$p_{eff} = t_w + 2r + 7t_f \cdot f_{y0} / f_{yi}$
but: $p_{eff} \le b_i + h_i - 2t_i$
Brace failure need not be checked if following conditions are fulfilled:
$\beta \le 1{,}0 - 0{,}03\gamma$
$g/t_f \le 20 - 28\beta$
$0{,}75 \le b_1/b_2 \le 1{,}33$
$0{,}75 \le d_1/d_2 \le 1{,}33$
where:
$\beta = \dfrac{\sum_{i=1}^{m} b_i + \sum_{i=1}^{m} h_i}{2m \cdot b_0}$
m is the amount of brace members
$\gamma = 0{,}5\, b_0/t_f$</td></tr>
<tr><td colspan="2">N and K joint: Chord shear</td></tr>
<tr><td>$N_{i.Rd} = \dfrac{f_{y0} \cdot A_{v0}}{\sqrt{3}\sin\theta_i} / \gamma_{M5}$
$N_{0.gap.Rd} =$
$\dfrac{\left((A_0 - A_{v0})f_{y0} + A_{v0}f_{y0}\sqrt{1 - \left(\dfrac{V_{Ed}}{V_{pl.Rd}}\right)^2}\right)}{\gamma_{M5}}$</td><td>$A_{v0} = A_0 - (2 - \alpha)b_0 t_f + (t_w + 2r)t_f$
$\alpha = \sqrt{\dfrac{1}{1 + \dfrac{4g^2}{3t_f^2}}}$
$\alpha = 0$ for circular brace members
V_{Ed} is chord shear at gap area
$V_{pl.Rd} = \dfrac{f_{y0} \cdot A_{v0}}{\sqrt{3} \cdot \gamma_{M5}}$</td></tr>
<tr><td colspan="2">If circular brace members, replace b_i and h_i with the diameter d_i, and for brace failure multiply above resistance value by $\pi/4$.</td></tr>
<tr><td>Reduction factor for above resistances:</td><td colspan="2">S235 - S355: 1,0
S420 - S460: 0,9</td></tr>
</table>

Table 11.3.8 Resistance of overlap N and K joints [1...5]:
- chords are I-sections
- brace members are square, rectangular or circular hollow sections



N_1 , N_2 one brace in compression,
one brace in tension

N_1 overlapping brace member (located on top)

N_2 overlapped brace member (located underneath)

N_0 the bigger of chord normal force (left/right) absolute values

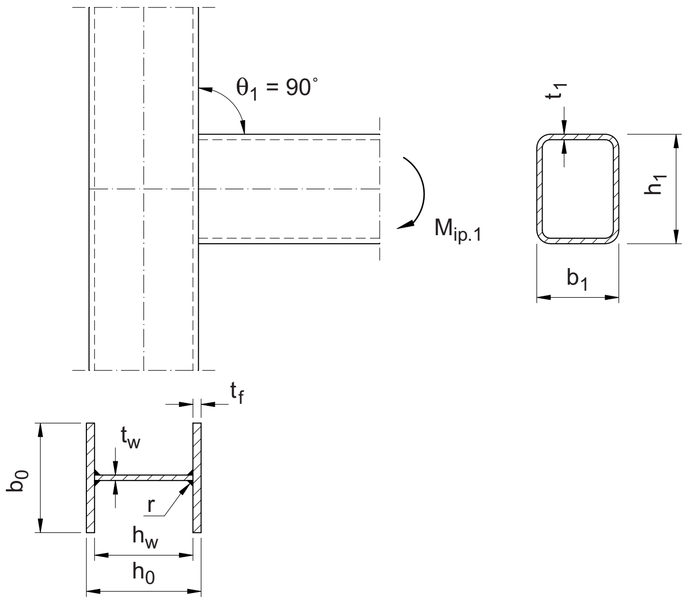
<table>
<tr><td>Resistance [i = 1]</td><td>Parameters</td><td>Validity range</td></tr>
<tr><td colspan="2">25 % $\le \lambda_{ov} <$ 50 % , Brace failure</td><td rowspan="9">In general:
$30^\circ \le \theta_i \le 90^\circ$
$t_i \ge 2{,}5$ mm
$L_i \ge 6h_i$ or $6d_i$
$-0{,}55 \le e/h_0 \le 0{,}25$

Brace members:
- in general:
$0{,}5 \le h_i/b_i \le 2{,}0$
$b_i/t_i \le 35$
$h_i/t_i \le 35$
$d_i/t_i \le 50$
$0{,}75 \le b_1/b_2 \le 1{,}0$
$0{,}75 \le d_1/d_2 \le 1{,}0$
$(t_1 f_{y1}) / (t_2 f_{y2}) \le 1{,}0$
- compression brace member:
Class 1
(based on pure axial compression)

Chords:
- web:
$h_w \le 400$ mm
- web and flanges:
Class 1 or 2
(based on pure axial compression)

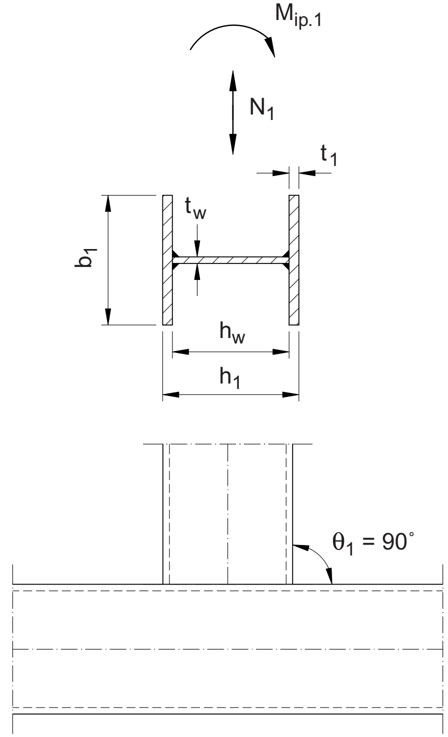
Overlap:
$25\ \% \le \lambda_{ov} \le \lambda_{ov.lim}$ a)</td></tr>
<tr><td>$N_{1.Rd} = f_{y1} t_1 \left(p_{eff} + b_{e.ov} + \frac{\lambda_{ov}}{50} \cdot 2h_1 - 4t_1\right) / \gamma_{M5}$</td><td>$\lambda_{ov} = \frac{q \cdot \sin\theta_1}{h_1} \cdot 100\ \%$
$p_{eff} = t_w + 2r + 7t_f \cdot f_{y0}/f_{y1}$
but: $p_{eff} \le b_1 + h_1 - 2t_1$
$b_{e.ov} = \frac{10}{b_2/t_2} \cdot \frac{f_{y2} \cdot t_2}{f_{y1} \cdot t_1} \cdot b_1 \le b_1$</td></tr>
<tr><td colspan="2">50 % $\le \lambda_{ov} <$ 80 % , Brace failure</td></tr>
<tr><td>$N_{1.Rd} = f_{y1} t_1 \cdot (p_{eff} + b_{e.ov} + 2h_1 - 4t_1) / \gamma_{M5}$</td><td>$\lambda_{ov} = \frac{q \cdot \sin\theta_1}{h_1} \cdot 100\ \%$
$p_{eff} = t_w + 2r + 7t_f \cdot f_{y0}/f_{y1}$
but: $p_{eff} \le b_1 + h_1 - 2t_1$
$b_{e.ov} = \frac{10}{b_2/t_2} \cdot \frac{f_{y2} \cdot t_2}{f_{y1} \cdot t_1} \cdot b_1 \le b_1$</td></tr>
<tr><td colspan="2">$\lambda_{ov} \ge$ 80 % , Brace failure</td></tr>
<tr><td>$N_{1.Rd} = f_{y1} t_1 \cdot (b_1 + b_{e.ov} + 2h_1 - 4t_1) / \gamma_{M5}$</td><td>$\lambda_{ov} = \frac{q \cdot \sin\theta_1}{h_1} \cdot 100\ \%$
$b_{e.ov} = \frac{10}{b_2/t_2} \cdot \frac{f_{y2} \cdot t_2}{f_{y1} \cdot t_1} \cdot b_1 \le b_1$</td></tr>
<tr><td colspan="2">If circular brace members, multiply above resistance values by $\pi/4$ and replace b_i and h_i with the diameter d_i .</td></tr>
<tr><td colspan="2">**Brace 1 (= overlapping brace):** resistance is determined acc. to above formulas.
Brace 2 (= overlapped brace): resistance in N/K joints is determined by using joint efficiency (ie. the design resistance of the joint divided by the plastic resistance of the brace member) which should be taken as equal to joint efficiency of the overlapping brace member.</td></tr>
<tr><td colspan="2">a) If $\lambda_{ov} > \lambda_{ov.lim}$ <u>or</u> if the braces are rectangular sections with $h_1 < b_1$ and/or $h_2 < b_2$, the connection between braces and chord has to be checked for shear as given in clause 7.4.7:
$\lambda_{ov.lim}$ = 60 % if the hidden seam of the overlapped brace is not welded to the chord
$\lambda_{ov.lim}$ = 80 % if the hidden seam of the overlapped brace is welded to the chord.</td></tr>
<tr><td>Reduction factor for above resistances:</td><td colspan="2">S235 - S355: 1,0
S420 - S460: 0,9</td></tr>
</table>

Table 11.3.9 Resistance of T joint for bending moment [1...5]:
- chords are I-sections
- brace members are square or rectangular hollow sections



Resistance [i = 1]	Parameters	Validity range
Brace failure		In general: $t_i \geq 2,5$ mm $L_i \geq 6 h_i$
$M_{ip.1.Rd} = f_{y1} \cdot t_1 \cdot p_{eff} \cdot (h_1 - t_1)/\gamma_{M5}$	$p_{eff} = t_w + 2r + 7 t_f f_{y0} / f_{yi}$ but: $p_{eff} \leq b_i + h_i - 2t_i$	
Chord web yielding		Brace members: $0,5 \leq h_i / b_i \leq 2,0$ $b_i / t_i \leq 35$ $h_i / t_i \leq 35$ $d_i / t_i \leq 50$
$M_{ip.1.Rd} = 0,5 \cdot f_{y0} \cdot t_w \cdot b_w \cdot (h_1 - t_1)/\gamma_{M5}$	$b_w = \dfrac{h_i}{\sin\theta_i} + 5(t_f + r)$ but: $b_w \leq 2t_i + 10(t_f + r)$	
		Chords: - web: $h_w \leq 400$ mm - web and flanges: Class 1 or 2 (based on pure axial compression)
Reduction factor for above resistances:	S235 - S355: 1,0 S420 - S460: 0,9	

Table 11.3.10 Resistance of T joint for bending moment [1...5]:
- chords are square or rectangular hollow sections
- brace members are I-sections



<table>
<tr><th>Resistance [i = 1]</th><th>Parameters</th><th>Validity range</th></tr>
<tr><td colspan="2">Resistance, if $\eta \geq 2\sqrt{1-\beta}$ a)</td><td rowspan="10">In general:
$t_0 \geq 2{,}5$ mm
$L_i \geq 6 h_i$

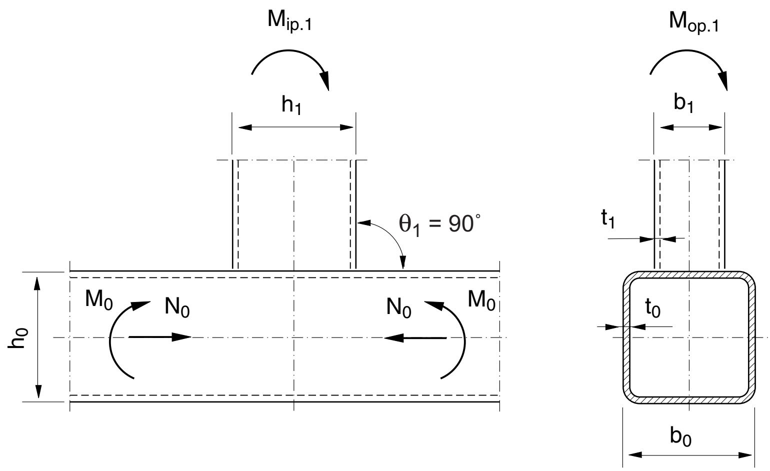
Brace members:
$h_w \leq 400$ mm
$0{,}5 \leq h_i / b_i \leq 2{,}0$
$0{,}5 \leq b_i / b_o \leq 1{,}0$
- web and flanges:
Class 1 or 2
(based on pure
axial compression)

Chords:
$b_0 / t_0 \leq 30$
$h_0 / t_0 \leq 35$
$0{,}5 \leq h_0 / b_0 \leq 2{,}0$
Class 1 or 2
(based on pure
axial compression)</td></tr>
<tr><td colspan="2">$\beta \leq 0{,}85$, Chord face failure</td></tr>
<tr><td>$N_{1.Rd} = 2 \cdot k_m \cdot f_{y0} \cdot t_0^2 \cdot \frac{2 + 2{,}8\beta}{\sqrt{1 - 0{,}9\beta}} / \gamma_{M5}$</td><td>$\beta = b_i / b_0$
$\eta = h_i / b_0$

Tension chord:
$k_m = 1{,}0$

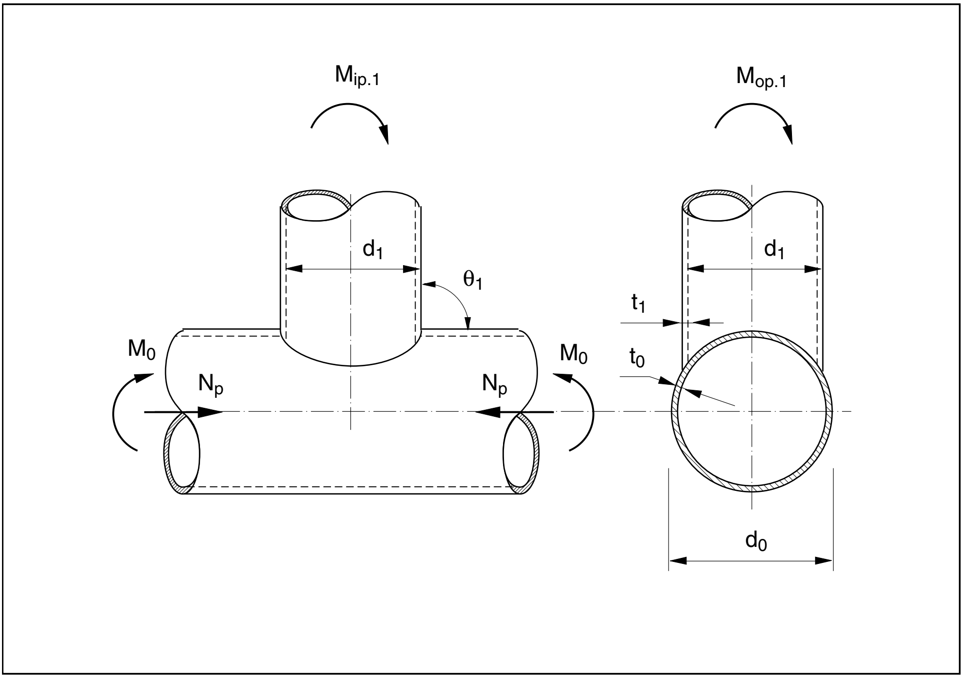
Compression chord:
$k_m = 1{,}3 - 1{,}3\lvert n \rvert \leq 1{,}0$
$n = \frac{N_{0.Ed}}{A_0 \cdot f_{y0} / \gamma_{M5}} + \frac{M_{0.Ed}}{W_{el.0} \cdot f_{y0} / \gamma_{M5}}$</td></tr>
<tr><td colspan="2">$b_1 \leq b_0 - 2t_0$, Chord face punching shear</td></tr>
<tr><td>$N_{1.Rd} = 2 \cdot \frac{f_{y0} \cdot t_0}{\sqrt{3}} \cdot (2t_1 + 2b_{e.p}) / \gamma_{M5}$</td><td>$b_{e.p} = \frac{10}{b_0 / t_0} b_i \leq b_i$</td></tr>
<tr><td colspan="2">$b_1 \geq b_0 - 2t_0$, Chord web yielding</td></tr>
<tr><td>$N_{1.Rd} = 2 \cdot k_m \cdot f_{y0} \cdot t_0 \cdot (2t_1 + 10t_0) / \gamma_{M5}$</td><td></td></tr>
<tr><td colspan="2">In-plane bending moment</td></tr>
<tr><td colspan="2">$M_{ip.1.Rd} = 0{,}5 \cdot N_{1.Rd} \cdot (h_1 - t_1) / \gamma_{M5}$</td></tr>
<tr><td colspan="2">a) If $0 \leq \eta < 2\sqrt{1-\beta}$, use linear interpolation between the following values:
- for $\eta = 2\sqrt{1-\beta}$, resistance is determined acc. to above formulas
- for $\eta = 0$, multiply above resistance values by 0,5</td></tr>
<tr><td>Reduction factor for above resistances:</td><td colspan="2">S235 - S355: 1,0
S420 - S460: 0,9</td></tr>
</table>

Table 11.3.11 Resistance of T and X joints for bending moment [1...5]:
- chords are square or rectangular hollow sections
- brace members are square or rectangular hollow sections



Resistance [i = 1]	Parameters	Validity range
In-plane bending moment, $\beta \le 0,85$, Chord face failure		In general: $t_0 \ge 2,5$ mm $t_i \ge 2,5$ mm $L_i \ge 6h_i$ Brace members: - in general: $b_i / b_0 \ge 0,25$ $b_i / t_i \le 35$ $h_i / t_i \le 35$ $0,5 \le h_i / b_i \le 2,0$ - compression brace member: Class 1 or 2 (based on pure axial compression) Chords: $b_0 / t_0 \le 35$ $h_0 / t_0 \le 35$ $0,5 \le h_0 / b_0 \le 2,0$ Class 1 or 2 (based on pure axial compression)
$M_{ip.1.Rd} =$ $k_n \cdot f_{y0} \cdot t_0^2 \cdot h_1 \cdot \left(\frac{1}{2\eta} + \frac{2}{\sqrt{1-\beta}} + \frac{\eta}{1-\beta}\right) / \gamma_{M5}$	$\beta = b_i / b_0$ $\eta = h_i / b_0$ Tension chord: $k_n = 1,0$ Compression chord: $k_n = 1,3 - \frac{0,4\lvert n\rvert}{\beta} \le 1,0$ $n = \frac{N_{0.Ed}}{A_0 \cdot f_{y0}/\gamma_{M5}} + \frac{M_{0.Ed}}{W_{el.0} \cdot f_{y0}/\gamma_{M5}}$	
In-plane bending moment, $0,85 < \beta \le 1,0$, Chord side wall failure		
$M_{ip.1.Rd} = 0,5 \cdot f_{yk} \cdot t_0 \cdot (h_1 + 5t_0)^2 / \gamma_{M5}$	$f_{yk} = f_{y0}$ (for T joint) $f_{yk} = 0,8\, f_{y0}$ (for X joint)	
In-plane bending moment, $0,85 < \beta \le 1,0$, Brace failure		
$M_{ip.1.Rd} =$ $f_{y1} \cdot [W_{pl.1} - (1 - b_{eff}/b_1) b_1 (h_1 - t_1) t_1] / \gamma_{M5}$	$b_{eff} = \frac{10}{b_0/t_0} \cdot \frac{f_{y0} \cdot t_0}{f_{yi} \cdot t_i} \cdot b_i \le b_i$	
Out-of-plane bending moment, $\beta \le 0,85$, Chord face failure		
$M_{op.1.Rd} =$ $k_n \cdot f_{y0} \cdot t_0^2 \cdot \left(\frac{h_1(1+\beta)}{2(1-\beta)} + \sqrt{\frac{2b_0 b_1 (1+\beta)}{1-\beta}}\right) / \gamma_{M5}$	$\beta = b_i / b_0$	
Out-of-plane bending moment, $0,85 < \beta \le 1,0$, Chord side wall failure		
$M_{op.1.Rd} = f_{yk} \cdot t_0 \cdot (b_0 - t_0)(h_1 + 5t_0) / \gamma_{M5}$	$f_{yk} = f_{y0}$ (for T joint) $f_{yk} = 0,8\, f_{y0}$ (for X joint)	
Out-of-plane bending moment, $0,85 < \beta \le 1,0$, Brace failure		
$M_{op.1.Rd} =$ $f_{y1} \cdot [W_{pl.1} - 0,5(1 - b_{eff}/b_1)^2 b_1^2 t_1] / \gamma_{M5}$	$b_{eff} = \frac{10}{b_0/t_0} \cdot \frac{f_{y0} \cdot t_0}{f_{yi} \cdot t_i} \cdot b_i \le b_i$	
Out-of-plane bending moment, T joint, Chord distortional failure		
$M_{op.1.Rd} =$ $2 \cdot f_{y0} \cdot t_0 \cdot (h_1 t_0 + \sqrt{b_0 h_0 t_0 (b_0 + h_0)}) / \gamma_{M5}$		
Reduction factor for above resistances:	S235 - S355: 1,0 S420 - S460: 0,9	

Table 11.3.12 Resistance of T, Y and X joints for bending moment [1...5]:
- chords are circular hollow sections
- brace members are circular hollow sections



<table>
<tr><th>Resistance [i = 1]</th><th>Parameters</th><th>Validity range</th></tr>
<tr><td colspan="2">In-plane bending moment,
Chord face failure</td><td rowspan="8">In general:
$30^\circ \le \theta_i \le 90^\circ$
$t_0 \ge 2{,}5$ mm
$t_i \ge 2{,}5$ mm
$L_i \ge 6 d_i$

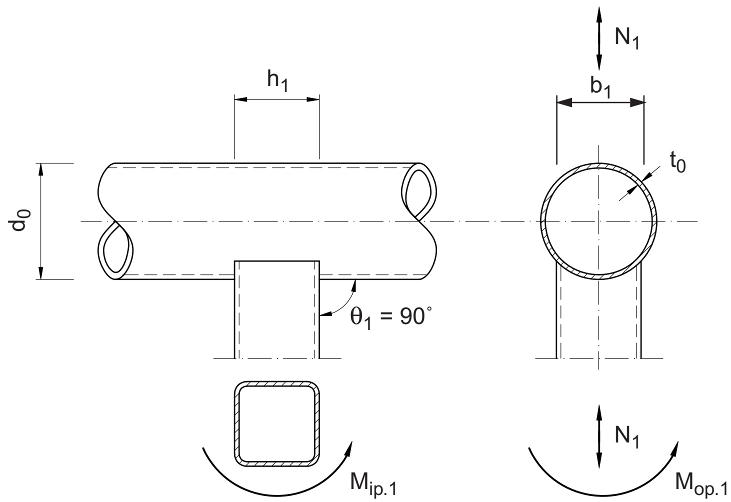
Brace members:
- in general:
$0{,}2 \le d_i / d_0 \le 1{,}0$
- tension brace member:
$d_i / t_i \le 50$
- compression brace member:
Class 1 or 2
(based on pure axial compression)

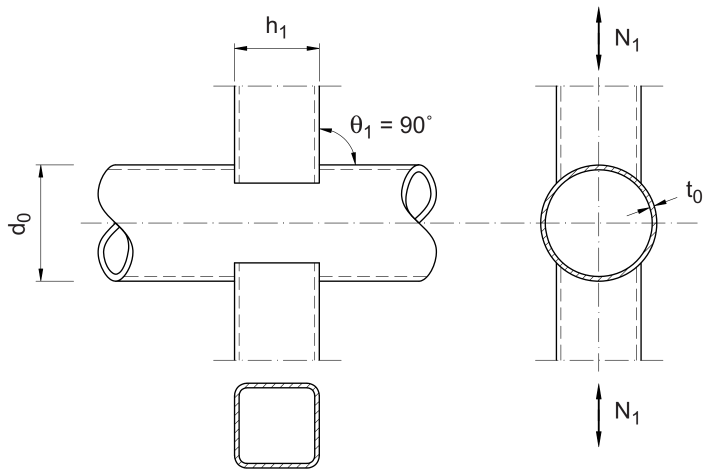
Chords:
- T and Y joint:
$10 \le d_0 / t_0 \le 50$
- X joint:
$10 \le d_0 / t_0 \le 40$
- compression chord:
Class 1 or 2
(based on pure axial compression)</td></tr>
<tr><td>$M_{ip.1.Rd} = \frac{4,85 \cdot \sqrt{\gamma} \cdot k_p \cdot f_{y0} \cdot t_0^2 \cdot d_1}{\sin\theta_1} \cdot \beta / \gamma_{M5}$</td><td>$\beta = d_i / d_0$
$\gamma = 0{,}5\, d_0 / t_0$

Tension chord:
$k_p = 1{,}0$

Compression chord:
$k_p = 1,0 - 0,3\lvert n_p\rvert - 0,3 n_p^2 \le 1,0$
$n_p = \frac{N_{p.Ed}}{A_0 \cdot f_{y0}/\gamma_{M5}} + \frac{M_{0.Ed}}{W_{el.0} \cdot f_{y0}/\gamma_{M5}}$</td></tr>
<tr><td colspan="2">In-plane bending moment,
$d_1 \le d_0 - 2t_0$, Chord face punching shear</td></tr>
<tr><td>$M_{ip.1.Rd} = \frac{f_{y0} \cdot t_0 \cdot d_1^2}{\sqrt{3}} \cdot \frac{1 + 3\sin\theta_1}{4\sin^2\theta_1} / \gamma_{M5}$</td><td></td></tr>
<tr><td colspan="2">Out-of-plane bending moment,
Chord face failure</td></tr>
<tr><td>$M_{op.1.Rd} = \frac{k_p \cdot f_{y0} \cdot t_0^2 \cdot d_1}{\sin\theta_1} \cdot \frac{2,7}{1 - 0{,}81\beta} / \gamma_{M5}$</td><td></td></tr>
<tr><td colspan="2">Out-of-plane bending moment,
$d_1 \le d_0 - 2t_0$, Chord face punching shear</td></tr>
<tr><td>$M_{op.1.Rd} = \frac{f_{y0} \cdot t_0 \cdot d_1^2}{\sqrt{3}} \cdot \frac{3 + 3\sin\theta_1}{4\sin^2\theta_1} / \gamma_{M5}$</td><td></td></tr>
<tr><td>Reduction factor for above resistances:</td><td colspan="2">S235 - S355: 1,0
S420 - S460: 0,9</td></tr>
</table>

Table 11.3.13 Resistance of T and X joints for bending moment [1...5]:
- chords are circular hollow sections
- brace members are square or rectangular hollow sections





<table>
<tr><th>Resistance [i = 1]</th><th>Parameters</th><th>Validity range</th></tr>
<tr><td colspan="2">T joint: Chord face failure</td><td rowspan="7">In general:
$t_0 \geq 2{,}5$ mm
$t_i \geq 2{,}5$ mm
$L_i \geq 6h_i$

Brace members:
- in general:
$\beta = b_i / d_0 \geq 0{,}4$
$\eta = h_i / d_0 \leq 4$
$b_i / t_i \leq 35$
$h_i / t_i \leq 35$
$0{,}5 \leq h_i / b_i \leq 2{,}0$
- compression brace member: Class 1 or 2 (based on pure axial compression)

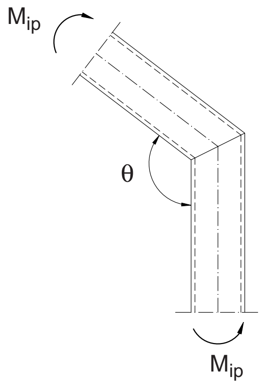
Chords:
- T joint:
$10 \leq d_0 / t_0 \leq 50$
- X joint:
$10 \leq d_0 / t_0 \leq 40$
- compression chord: Class 1 or 2 (based on pure axial compression)</td></tr>
<tr><td>$N_{1.Rd} = k_p \cdot f_{y0} \cdot t_0^2 \cdot (4 + 20\beta^2)(1 + 0{,}25\eta) / \gamma_{M5}$
$M_{ip1.Rd} = N_{1.Rd} \cdot h_1$
$M_{op1.Rd} = 0{,}5 \cdot N_{1.Rd} \cdot b_1$</td><td>$\beta = b_i / d_0$
$\eta = h_i / d_0$

Tension chord:
$k_p = 1{,}0$

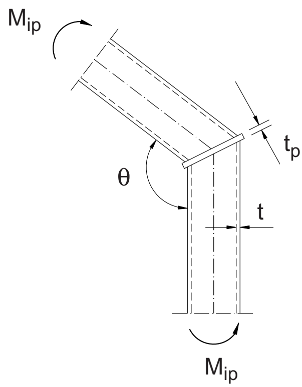
Compression chord:
$k_p = 1{,}0 - 0{,}3|n_p| - 0{,}3n_p^2 \leq 1{,}0$
$n_p = \dfrac{N_{p.Ed}}{A_0 \cdot f_{y0} / \gamma_{M5}} + \dfrac{M_{0.Ed}}{W_{el.0} \cdot f_{y0} / \gamma_{M5}}$</td></tr>
<tr><td colspan="2">X joint: Chord face failure</td></tr>
<tr><td>$N_{1.Rd} = \dfrac{5 \cdot k_p \cdot f_{y0} \cdot t_0^2}{1 - 0{,}81\beta} \cdot (1 + 0{,}25\eta) / \gamma_{M5}$
$M_{ip1.Rd} = N_{1.Rd} \cdot h_1$
$M_{op1.Rd} = 0{,}5 \cdot N_{1.Rd} \cdot b_1$</td><td></td></tr>
<tr><td colspan="2">T and X joint: Chord face punching shear</td></tr>
<tr><td>$\sigma_{max} \cdot t_1 = \left(\dfrac{N_{1.Ed}}{A_1} + \dfrac{M_{1.Ed}}{W_{el.1}}\right) \cdot t_1 \leq \dfrac{f_{y0} \cdot t_0}{\sqrt{3} \cdot \gamma_{M5}}$</td><td></td></tr>
<tr><td colspan="2"></td></tr>
<tr><td>Reduction factor for above resistances:</td><td colspan="2">S235 - S355: 1,0
S420 - S460: 0,9</td></tr>
</table>

Table 11.3.14 Resistance of Knee joint for bending moment [1...5]:
- chords are square or rectangular hollow sections
- brace members are square, rectangular or circular hollow sections

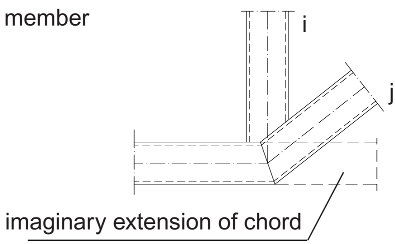
A) Knee joint



B) Knee joint with reinforcing plate

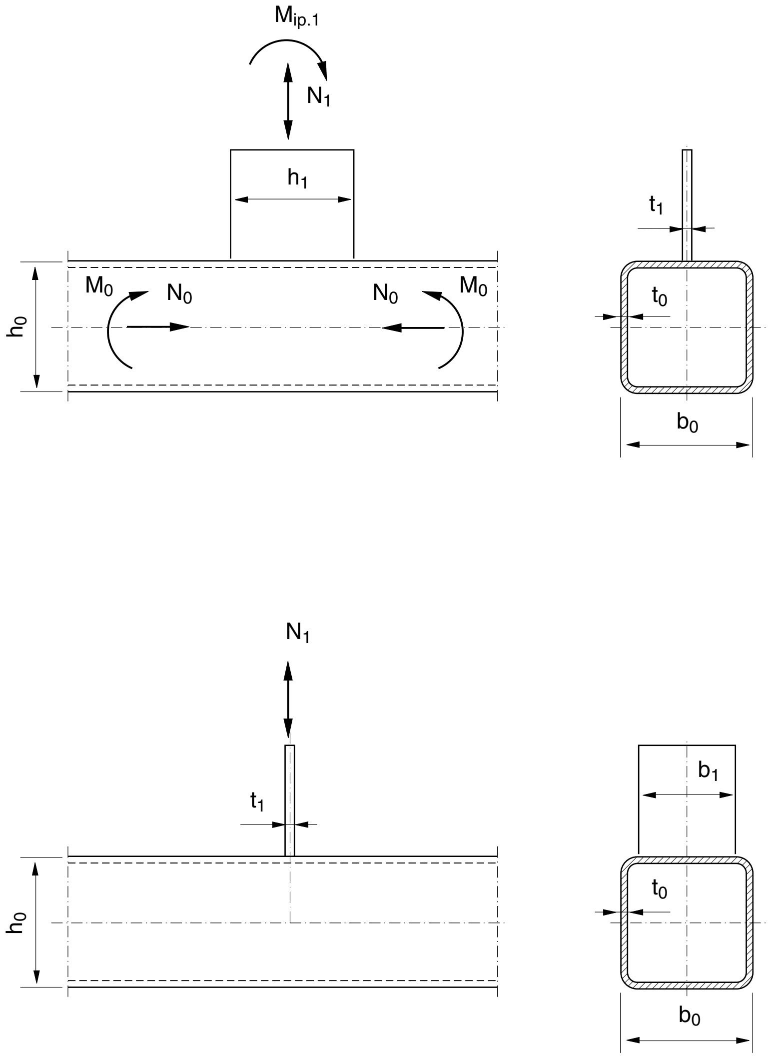


C) Cranked-chord with brace member



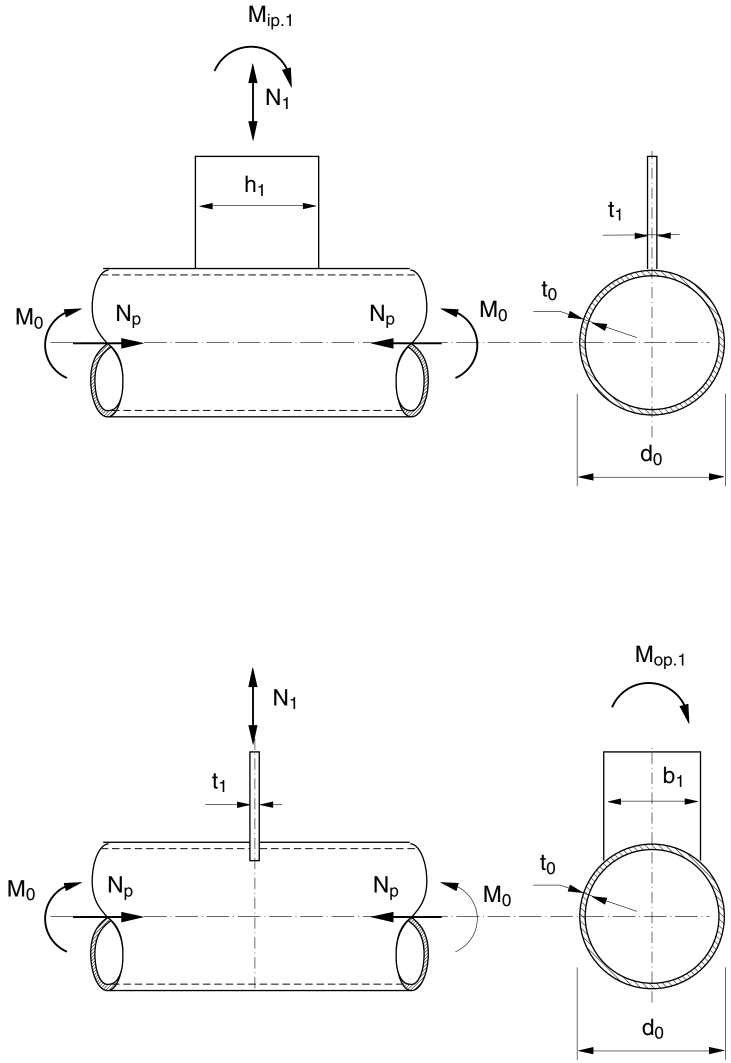
Resistance	Parameters	Validity range
Knee joint (Figure A)		
$N_{Ed} \le 0,2 N_{pl.Rd}$ $\frac{N_{Ed}}{N_{pl.Rd}} + \frac{M_{ip.Ed}}{M_{pl.Rd}} \le \kappa$	$N_{pl.Rd} = \frac{f_y \cdot A}{\gamma_{M0}}$ $M_{pl.Rd} = \frac{f_y \cdot W_{pl}}{\gamma_{M0}}$ If $\theta \le 90^\circ$: $\kappa = \frac{3\sqrt{b_0/h_0}}{(b_0/t_0)^{0,8}} + \frac{1}{1+2 \cdot b_0/h_0}$ If $90^\circ < \theta \le 180^\circ$: $\kappa = 1 - [\sqrt{2} \cdot \cos(\theta/2)] \cdot (1 - \kappa_{90})$ where κ_{90} is κ when $\theta = 90^\circ$	Chords: Class 1 (based on pure axial compression)
Knee joint with reinforcing plate (Figure B)		
$\frac{N_{Ed}}{N_{pl.Rd}} + \frac{M_{ip.Ed}}{M_{pl.Rd}} \le 1,0$		Chords: Class 1 (based on pure axial compression) Plate: $t_p \ge 10$ mm $t_p \ge 1,5 t_0$
Cranked-chord with brace member (Figure C)		
$N_{i.Ed} \le N_{i.Rd}$	$N_{i.Rd}$ shall be determined as for overlapping brace member (i = 1) in overlap N and K joint (Table 11.3.3).	Validity range: see Table 11.3.3
Reduction factor for above resistances:	S235 - S355: 1,0 S420 - S460: 0,9	

Table 11.3.15 Joints between plate and square or rectangular hollow section [1...5]



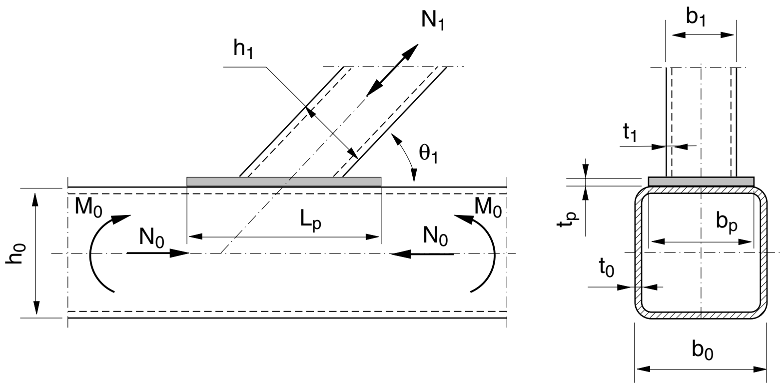
Resistance [i = 1]	Parameters	Validity range
Longitudinal plate, Chord face failure		In general: $t_0 \geq 2{,}5$ mm Longitudinal plates: $t_i / b_0 \leq 0{,}2$ Transverse plates: $0{,}5 \leq b_i / b_0 \leq 1{,}0$ Chords: $b_0 / t_0 \leq 35$ $h_0 / t_0 \leq 35$ $0{,}5 \leq h_0 / b_0 \leq 2{,}0$ Class 1 or 2 (based on pure axial compression)
$N_{1.Rd} = k_m \cdot f_{y0} \cdot t_0^2 \cdot (2h_1/b_0 + 4\sqrt{1 - t_1/b_0})/\gamma_{M5}$ $M_{ip.1.Rd} = 0{,}5 N_{1.Rd} h_1$	Tension chord: $k_m = 1{,}0$ Compression chord: $k_m = 1{,}3 - 1{,}3\lvert n\rvert \leq 1{,}0$ $n = \dfrac{N_{0.Ed}}{A_0 \cdot f_{y0}/\gamma_{M5}} + \dfrac{M_{0.Ed}}{W_{el.0} \cdot f_{y0}/\gamma_{M5}}$	
Transverse plate, $\beta \leq 0{,}85$, Chord face failure		
$N_{1.Rd} = k_m \cdot f_{y0} \cdot t_0^2 \cdot \dfrac{2 + 2{,}8\beta}{\sqrt{1 - 0{,}9\beta}}/\gamma_{M5}$	$\beta = b_i / b_0$	
Transverse plate, $b_1 \leq b_0 - 2t_0$, Chord face punching shear		
$N_{1.Rd} = \dfrac{f_{y0} \cdot t_0}{\sqrt{3}} \cdot (2t_1 + 2b_{e.p})/\gamma_{M5}$	$b_{e.p} = \dfrac{10}{b_0/t_0} b_i \leq b_i$	
Transverse plate, $b_1 \geq b_0 - 2t_0$, Chord side wall failure		
$N_{1.Rd} = k_m \cdot f_{y0} \cdot t_0 \cdot (2t_1 + 10t_0)/\gamma_{M5}$		
Reduction factor for above resistances:	S235 - S355: 1,0 S420 - S460: 0,9	

Tauble 11.3.16 Joints between plate and circular hollow section [1...5]



Resistance [i = 1]	Parameters	Validity range
Longitudinal plate, Chord face failure		In general: $t_0 \geq 2{,}5$ mm Longitudinal plates: $\eta = h_i / d_0 \leq 4$ Transverse plates: $\beta = b_i / d_0 \geq 0{,}4$ Chords: - if plate on one side: $10 \leq d_0 / t_0 \leq 50$ - if plate on both sides: $10 \leq d_0 / t_0 \leq 40$ - compression chord: Class 1 or 2 (based on pure axial compression)
$N_{1.Rd} = 5 \cdot k_p \cdot f_{y0} \cdot t_0^2 \cdot (1 + 0{,}25\eta) / \gamma_{M5}$ $M_{ip1.Rd} = N_{1.Rd} \cdot h_1$ $M_{op1.Rd} = 0$	$\eta = h_i / d_0$ Tension chord: $k_p = 1{,}0$ Compression chord: $k_p = 1{,}0 - 0{,}3\lvert n_p \rvert - 0{,}3 n_p^2 \leq 1{,}0$ $n_p = \dfrac{N_{p.Ed}}{A_0 \cdot f_{y0} / \gamma_{M5}} + \dfrac{M_{0.Ed}}{W_{el.0} \cdot f_{y0} / \gamma_{M5}}$	
Longitudinal plate on both sides of the hollow section, Chord face failure		
$N_{1.Rd} = 5 \cdot k_p \cdot f_{y0} \cdot t_0^2 \cdot (1 + 0{,}25\eta) / \gamma_{M5}$ $M_{ip1.Rd} = N_{1.Rd} \cdot h_1$ $M_{op1.Rd} = 0$		
Transverse plate, Chord face failure		
$N_{1.Rd} = k_p \cdot f_{y0} \cdot t_0^2 \cdot (4 + 20\beta^2) / \gamma_{M5}$ $M_{ip1.Rd} = 0$ $M_{op1.Rd} = 0{,}5 \cdot N_{1.Rd} \cdot b_1$	$\beta = b_i / d_0$	
Transverse plate on both sides of the hollow section, Chord face failure		
$N_{1.Rd} = \dfrac{5 \cdot k_p \cdot f_{y0} \cdot t_0^2}{1 - 0{,}81\beta} / \gamma_{M5}$ $M_{ip1.Rd} = 0$ $M_{op1.Rd} = 0{,}5 \cdot N_{1.Rd} \cdot b_1$		
Longitudinal or transverse plate, Chord face punching shear		
$\sigma_{max} \cdot t_1 = \left(\dfrac{N_{1.Ed}}{A_1} + \dfrac{M_{1.Ed}}{W_{el.1}} \right) \cdot t_1 \leq \dfrac{2 \cdot f_{y0} \cdot t_0}{\sqrt{3} \cdot \gamma_{M5}}$		
Reduction factor for above resistances:	S235 - S355: 1,0 S420 - S460: 0,9	

Table 11.3.17 Resistance of T, Y and X joints with chord flange plate reinforcement [1...5]:
- chords are square or rectangular hollow sections
- brace members are square, rectangular or circular hollow sections



<table>
<tr><th>Resistance [i = 1]</th><th>Parameters</th><th>Validity range</th></tr>
<tr><td colspan="2">Chord face failure</td><td rowspan="11">In general:
$30^\circ \le \theta_i \le 90^\circ$
$t_0 \ge 2{,}5$ mm
$t_i \ge 2{,}5$ mm
$L_i \ge 6h_i$ or $6d_i$

Square and rectangular brace members:
- in general:
$b_i / b_0 \ge 0{,}25$
$b_i / t_i \le 35$
$h_i / t_i \le 35$
$0{,}5 \le h_i / b_i \le 2{,}0$
- compression brace member:
Class 1 or 2
(based on pure axial compression)

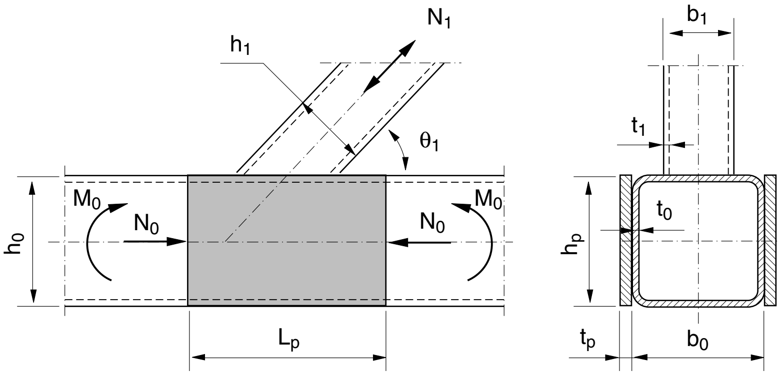
Circular brace members:
- in general:
$0{,}4 \le d_i / b_0 \le 0{,}8$
- tension brace member:
$d_i / t_i \le 50$
- compression brace member:
Class 1
(based on pure axial compression)

Chords:
$b_0 / t_0 \le 35$
$h_0 / t_0 \le 35$
$0{,}5 \le h_0 / b_0 \le 2{,}0$
Class 1 or 2
(based on pure axial compression)

Plate:
$f_{yp} \ge f_{y0}$
$b_p \ge b_0 - 2t_0$
$t_p \ge 2t_i$
$L_p \ge \frac{h_i}{\sin\theta_i} + \sqrt{b_p(b_p - b_i)}$</td></tr>
<tr><td>$N_{i.Rd} = \frac{f_{yp} \cdot t_p^2}{(1-\beta_p)\sin\theta_i}\left(\frac{2\eta_p}{\sin\theta_i} + 4\sqrt{1-\beta_p}\right)/\gamma_{M5}$</td><td>$\beta_p = b_i / b_p$
$\eta_p = h_i / b_p$</td></tr>
<tr><td colspan="2">Chord side wall buckling or yielding a)</td></tr>
<tr><td>$N_{i.Rd} = \frac{f_b \cdot t_0}{\sin\theta_i} \cdot \left(\frac{2h_i}{\sin\theta_i} + 10t_0\right)/\gamma_{M5}$</td><td>Tension chord:
$f_b = f_{y0}$

Compression chord:
$f_b = \chi \cdot f_{y0}$ (T and Y joint)
$f_b = 0{,}8\chi \cdot f_{y0} \cdot \sin\theta_i$ (X joint)
χ = reduction factor for flexural buckling using buckling curve c and non-dimensional slenderness determined from:
$\bar{\lambda} = 3,46 \cdot \frac{\left(\frac{h_0}{t_0} - 2\right) \cdot \sqrt{\frac{1}{\sin\theta_i}}}{\pi \cdot \sqrt{\frac{E}{f_{y0}}}}$</td></tr>
<tr><td colspan="2">Chord face punching shear</td></tr>
<tr><td>$N_{i.Rd} = \frac{f_{yp} \cdot t_p}{\sqrt{3}\sin\theta_i} \cdot \left(\frac{2h_i}{\sin\theta_i} + 2b_{e.p}\right)/\gamma_{M5}$</td><td>$b_{e.p} = \frac{10}{b_p/t_p} b_i \le b_i$</td></tr>
<tr><td colspan="2">Brace failure</td></tr>
<tr><td>$N_{i.Rd} = f_{yi} \cdot t_i \cdot (2h_i - 4t_i + 2b_{eff})/\gamma_{M5}$</td><td>$b_{eff} = \frac{10}{b_p/t_p} \cdot \frac{f_{yp} \cdot t_p}{f_{yi} \cdot t_i} \cdot b_i \le b_i$</td></tr>
<tr><td colspan="2">If circular brace members, multiply above resistance values by $\pi/4$ and replace b_i and h_i with the diameter d_i.</td></tr>
<tr><td colspan="2">a) For X joint with $\cos\theta_i > h_i / h_0$ use the smaller of following values:
this value or Table 11.3.2: gap N and K joint: chord shear.</td></tr>
<tr><td colspan="2">Note: The values in the table are valid, when $\beta_p \le 0{,}85$. The smallest value calculated from the different failure modes is chosen to be the joint's resistance.</td></tr>
<tr><td>Reduction factor for above resistances:</td><td colspan="2">S235 - S355: 1,0
S420 - S460: 0,9</td></tr>
</table>

Table 11.3.18 Resistance of T, Y and X joints with chord side plate reinforcement [1...5]:
- chords are square or rectangular hollow sections
- brace members are square, rectangular or circular hollow sections



<table>
<tr><th>Resistance [i = 1]</th><th>Parameters</th><th>Validity range</th></tr>
<tr><td colspan="2">$\beta \le 0{,}85$, Chord face failure</td><td rowspan="12">In general:
$30^\circ \le \theta_i \le 90^\circ$
$t_0 \ge 2{,}5$ mm
$t_i \ge 2{,}5$ mm
$L_i \ge 6h_i$ or $6d_i$

Square and rectangular brace members:
- in general:
$b_i / b_0 \ge 0{,}25$
$b_i / t_i \le 35$
$h_i / t_i \le 35$
$0{,}5 \le h_i / b_i \le 2{,}0$
- compression brace member:
Class 1 or 2
(based on pure axial compression)

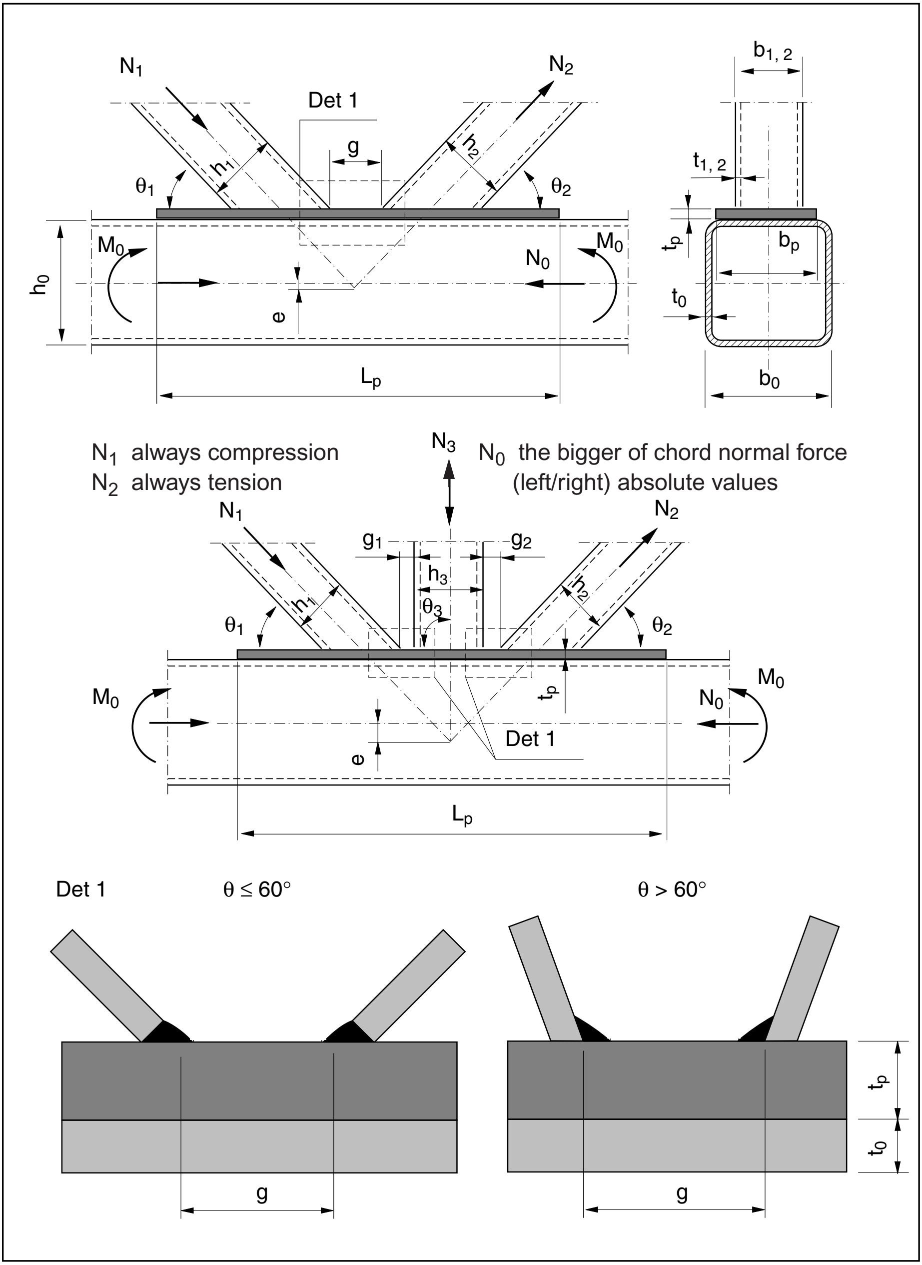
Circular brace members:
- in general:
$0{,}4 \le d_i / b_0 \le 0{,}8$
- tension brace member:
$d_i / t_i \le 50$
- compression brace member:
Class 1
(based on pure axial compression)

Chords:
$b_0 / t_0 \le 35$
$h_0 / t_0 \le 35$
$0{,}5 \le h_0 / b_0 \le 2{,}0$
Class 1 or 2
(based on pure axial compression)

Plates:
$f_{yp} \ge f_{y0}$
$t_p \ge 2t_i$
$L_p \ge 1{,}5\, h_i / \sin\theta_i$</td></tr>
<tr><td>$$N_{i.Rd} = \frac{k_n \cdot f_{y0} \cdot t_0^2}{(1-\beta)\sin\theta_i} \cdot \left(\frac{2\eta}{\sin\theta_i} + 4\sqrt{1-\beta}\right) / \gamma_{M5}$$</td><td>$\beta = b_i / b_0$
$\eta = h_i / b_0$
Tension chord:
$k_n = 1{,}0$
Compression chord:
$$k_n = 1{,}3 - \frac{0{,}4\,|n|}{\beta} \le 1{,}0$$
$$n = \frac{N_{0.Ed}}{A_0 \cdot f_{y0} / \gamma_{M5}} + \frac{M_{0.Ed}}{W_{el.0} \cdot f_{y0} / \gamma_{M5}}$$</td></tr>
<tr><td colspan="2">$0{,}85 < \beta < 1{,}0$</td></tr>
<tr><td colspan="2">Use linear interpolation between the following values:
- chord face failure when $\beta = 0{,}85$
- the critical of following values:
chord side wall buckling/yielding when $\beta = 1{,}0$, or Table 11.3.2: chord shear, replace wall thickness t_0 with thickness $(t_0 + t_p)$ in the formulas</td></tr>
<tr><td colspan="2">$\beta = 1{,}0$, Chord side wall buckling or yielding a)</td></tr>
<tr><td>$$N_{i.Rd} = \frac{k_n \cdot f_b \cdot (t_0 + t_p)}{\sin\theta_i} \cdot \left(\frac{2h_i}{\sin\theta_i} + 10(t_0 + t_p)\right) / \gamma_{M5}$$</td><td>Tension chord:
$f_b = f_{y0}$

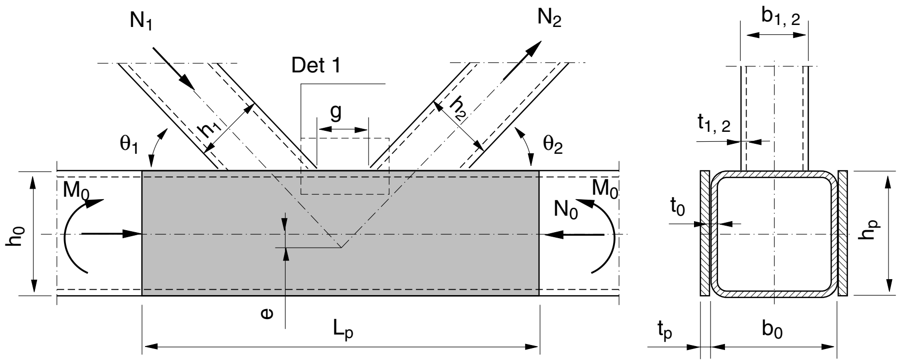
Compression chord:
$f_b = \chi \cdot f_{y0}$ (T and Y joint)
$f_b = 0{,}8\,\chi \cdot f_{y0} \cdot \sin\theta_i$ (X joint)
χ = reduction factor for flexural buckling using buckling curve c and non-dimensional slenderness determined from:
$$\bar{\lambda} = 3{,}46 \cdot \frac{\left(\frac{h_0}{t_0 + t_p} - 2\right) \cdot \sqrt{\frac{1}{\sin\theta_i}}}{\pi \cdot \sqrt{\frac{E}{f_{y0}}}}$$</td></tr>
<tr><td colspan="2">$0{,}85 \le \beta \le (1 - 1/\gamma)$, Chord face punching shear</td></tr>
<tr><td>$$N_{i.Rd} = \frac{f_{y0} \cdot t_0}{\sqrt{3}\sin\theta_i} \cdot \left(\frac{2h_i}{\sin\theta_i} + 2b_{e.p}\right) / \gamma_{M5}$$</td><td>$$b_{e.p} = \frac{10}{b_0 / t_0} b_i \le b_i$$
$$\gamma = \frac{b_0}{2t_0}$$</td></tr>
<tr><td colspan="2">$\beta \ge 0{,}85$, Brace failure</td></tr>
<tr><td>$$N_{i.Rd} = f_{yi} \cdot t_i \cdot (2h_i - 4t_i + 2b_{eff}) / \gamma_{M5}$$</td><td>$$b_{eff} = \frac{10}{b_0 / t_0} \cdot \frac{f_{y0} \cdot t_0}{f_{yi} \cdot t_i} \cdot b_i \le b_i$$</td></tr>
<tr><td colspan="2">If circular brace members, multiply above resistance values by $\pi/4$ and replace b_i and h_i with the diameter d_i .</td></tr>
<tr><td colspan="2">a) For X joint with $\cos\theta_i > h_i / h_0$ use the smaller of following values:
this value or Table 11.3.2: gap N and K joint: chord shear, replace wall thickness t_0 with thickness $(t_0 + t_p)$ in the formulas</td></tr>
<tr><td>Reduction factor for above resistances:</td><td colspan="2">S235 - S355: 1,0
S420 - S460: 0,9</td></tr>
</table>

Table 11.3.19 Resistance of gap N, K and KT joints with chord flange plate reinforcement [1...5]:
- chords are square or rectangular hollow sections
- brace members are square, rectangular or circular hollow sections

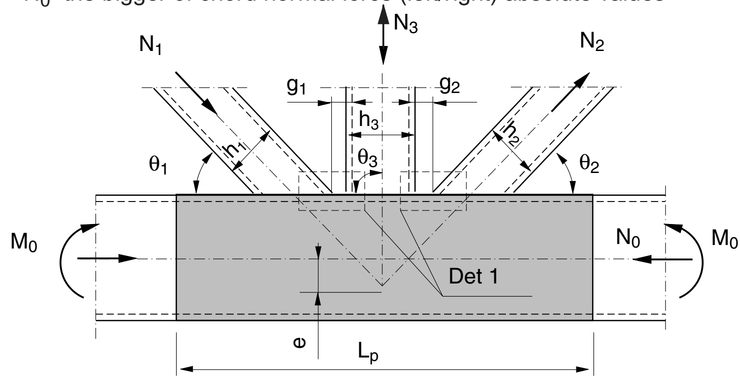


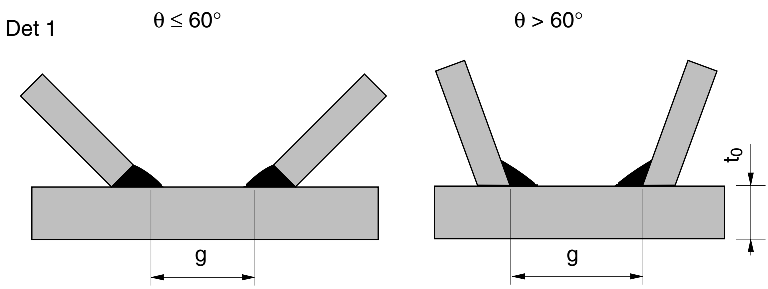
Resistance [i = 1 or 2]	Parameters	Validity range
Chord face failure		In general: $30^\circ \le \theta_i \le 90^\circ$ $t_0 \ge 2{,}5$ mm $t_i \ge 2{,}5$ mm $L_i \ge 6h_i$ or $6d_i$ $-0{,}55 \le e/h_0 \le 0{,}25$
$N_{i.Rd} = \frac{8{,}9 \cdot f_{yp} \cdot t_p^2 \cdot \sqrt{\gamma_p}}{\sin\theta_i} \cdot \left(\frac{\sum_{i=1}^{m} b_i + \sum_{i=1}^{m} h_i}{2m \cdot b_p} \right) / \gamma_{M5}$	$\beta_p = \frac{\sum_{i=1}^{m} b_i + \sum_{i=1}^{m} h_i}{2m \cdot b_p}$ m is the amount of brace members $\gamma_p = 0{,}5\, b_p / t_p$	Square and rectangular brace members: - in general: $b_i / b_0 \ge 0{,}35$
Chord shear		$b_i / b_0 \ge 0{,}1 + 0{,}01\, b_0 / t_0$
$N_{i.Rd} = \frac{f_{y0} \cdot A_{v0}}{\sqrt{3} \sin\theta_i} / \gamma_{M5}$ $N_{0.gap.Rd} =$ $\frac{\left((A_0 - A_{v0}) f_{y0} + A_{vo} f_{y0} \sqrt{1 - \left(\frac{V_{Ed}}{V_{pl.Rd}} \right)^2} \right)}{\gamma_{M5}}$	$A_{v0} = (2h_0 + \alpha b_0)\, t_0$ $\alpha = \sqrt{\frac{1}{1 + \frac{4g^2}{3t_0^2}}}$ $\alpha = 0$ for circular brace members V_{Ed} is chord shear at gap area $V_{pl.Rd} = \frac{f_{y0} \cdot A_{v0}}{\sqrt{3} \cdot \gamma_{M5}}$	$0{,}5 \le h_i / b_i \le 2{,}0$ $b_i / t_i \le 35$ $h_i / t_i \le 35$ - compression brace member: Class 1 or 2 (based on pure axial compression) Circular brace members: - in general: $0{,}4 \le d_i / b_0 \le 0{,}8$
$\beta_p \le (1 - 1/\gamma_p)$, Chord face punching shear		- tension brace member:
$N_{i.Rd} = \frac{f_{yp} \cdot t_p}{\sqrt{3} \sin\theta_i} \cdot \left(\frac{2h_i}{\sin\theta_i} + b_i + b_{e.p} \right) / \gamma_{M5}$	$b_{e.p} = \frac{10}{b_p / t_p} b_i \le b_i$	$d_i / t_i \le 50$ - compression brace member:
Brace failure		Class 1
$N_{i.Rd} = f_{yi}\, t_i\, (2h_i - 4t_i + b_i + b_{eff}) / \gamma_{M5}$	$b_{eff} = \frac{10}{b_p / t_p} \cdot \frac{f_{yp} \cdot t_p}{f_{yi} \cdot t_i} \cdot b_i \le b_i$	(based on pure axial compression) Chords:
If circular brace members, multiply above resistance values by $\pi/4$ and replace b_i and h_i with the diameter d_i.		$b_0 / t_0 \le 35$ $h_0 / t_0 \le 35$
KT joint: - For each brace member, check following condition: $N_{i.Rd} \ge N_{i.Ed}$ (i = 1,2,3) - Resistances for the brace members are calculated as for N joint using adjacent brace members in pairs as follows: A) brace members 1 & 3 B) brace members 2 & 3 If brace member 3 is in tension, its resistance is calculated from Case A. If brace member 3 is in compression, its resistance is calculated from Case B.		$0{,}5 \le h_0 / b_0 \le 2{,}0$ Class 1 or 2 (based on pure axial compression) Plate: $f_{yp} \ge f_{y0}$ $t_p \ge 2t_i$
a) If $g / b_0 > 1{,}5(1 - \beta)$ and $g \ge t_1 + t_2$ treat the joint as two separate T or Y joint.		$L_p \ge 1{,}5 \left(\sum_{i=1}^{m} \frac{h_i}{\sin\theta_i} + \sum_{i=1}^{m} g_i \right)$ Gap: $g \ge t_1 + t_2$ $g / b_p \ge 0{,}5\,(1 - \beta_p)$ $g / b_p \le 1{,}5\,(1 - \beta_p)$ a) If KT joint, check above conditions for both gaps separately.
Reduction factor for above resistances:	S235 - S355: 1,0 S420 - S460: 0,9	

Table 11.3.20 Resistance of gap N, K and KT joints with chord side plate reinforcement [1...5]:
- chords are square or rectangular hollow sections
- brace members are square, rectangular or circular hollow sections



N_1 always compression
N_2 always tension
N_0 the bigger of chord normal force (left/right) absolute values





<table>
<tr><th>Resistance [i = 1 or 2]</th><th>Parameters</th><th>Validity range</th></tr>
<tr><td colspan="2">Chord face failure</td><td rowspan="13">In general:
$30^\circ \le \theta_i \le 90^\circ$
$t_0 \ge 2{,}5$ mm
$t_i \ge 2{,}5$ mm
$L_i \ge 6h_i$ tai $6d_i$
$-0{,}55 \le e/h_0 \le 0{,}25$

Square and rectangular brace members:
- in general:
$b_i/b_0 \ge 0{,}35$
$b_i/b_0 \ge 0{,}1 + 0{,}01\,b_0/t_0$
$0{,}5 \le h_i/b_i \le 2{,}0$
$b_i/t_i \le 35$
$h_i/t_i \le 35$
- compression brace member:
Class 1 or 2
(based on pure axial compression)

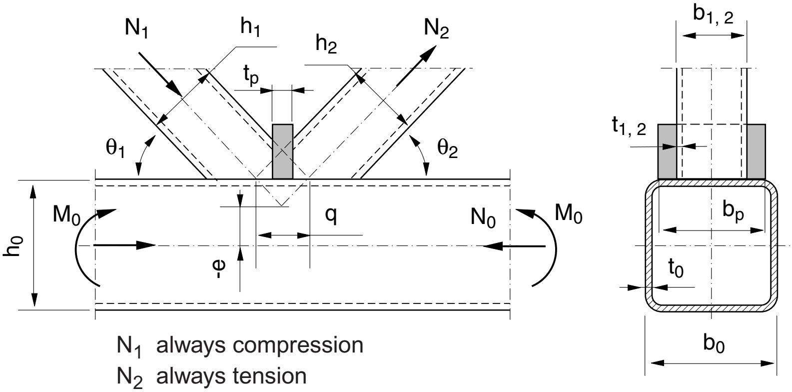
Circular brace members:
- in general:
$0{,}4 \le d_i/b_0 \le 0{,}8$
- tension brace member:
$d_i/t_i \le 50$
- compression brace member:
Class 1
(based on pure axial compression)

Chords:
$b_0/t_0 \le 35$
$h_0/t_0 \le 35$
$0{,}5 \le h_0/b_0 \le 2{,}0$
Class 1 or 2
(based on pure axial compression)

Plates:
$L_p \ge 1{,}5\left(\sum_{i=1}^{m}\frac{h_i}{\sin\theta_i} + \sum_{i=1}^{m} g_i\right)$

Gap:
$g \ge t_1 + t_2$
$g/b_0 \ge 0{,}5\,(1-\beta)$
$g/b_0 \le 1{,}5\,(1-\beta)$ a)
If KT joint, check above conditions for both gaps separately.</td></tr>
<tr><td>$N_{i.Rd} = \frac{8{,}9 \cdot k_n \cdot f_{y0} \cdot t_0^2 \cdot \sqrt{\gamma}}{\sin\theta_i} \cdot \left(\frac{\sum_{i=1}^{m} b_i + \sum_{i=1}^{m} h_i}{2m \cdot b_0}\right) / \gamma_{M5}$</td><td>$\beta = \frac{\sum_{i=1}^{m} b_i + \sum_{i=1}^{m} h_i}{2m \cdot b_0}$
m is the amount of brace members
$\gamma = 0{,}5\,b_0/t_0$
Tension chord:
$k_n = 1{,}0$
Compression chord:
$k_n = 1{,}3 - \frac{0{,}4\,\lvert n \rvert}{\beta} \le 1{,}0$
$n = \frac{N_{0.Ed}}{A_0 \cdot f_{y0}/\gamma_{M5}} + \frac{M_{0.Ed}}{W_{el.0} \cdot f_{y0}/\gamma_{M5}}$</td></tr>
<tr><td colspan="2">Chord shear</td></tr>
<tr><td>$N_{i.Rd} = \frac{f_{y0} \cdot A_{v0} + f_{yp} \cdot A_{vp}}{\sqrt{3}\sin\theta_i} / \gamma_{M5}$
$N_{0.gap.Rd} =$
$\frac{\left((A_0 - A_{v0})f_{y0} + (A_{v0}f_{y0} + A_{vp}f_{yp})\sqrt{1 - \left(\frac{V_{Ed}}{V_{pl.Rd}}\right)^2}\right)}{\gamma_{M5}}$</td><td>$A_{v0} = (2h_0 + \alpha b_0)\,t_0$
$\alpha = \sqrt{\frac{1}{1 + \frac{4g^2}{3t_0^2}}}$
$\alpha = 0$ for circular brace members
$A_{vp} = 2h_p t_p$
V_{Ed} is chord shear at gap area
$V_{pl.Rd} = \frac{f_{y0} \cdot A_{v0} + f_{yp} \cdot A_{vp}}{\sqrt{3} \cdot \gamma_{M5}}$</td></tr>
<tr><td colspan="2">$\beta \le (1 - 1/\gamma)$, Chord face punching shear</td></tr>
<tr><td>$N_{i.Rd} = \frac{f_{y0} \cdot t_0}{\sqrt{3}\sin\theta_i} \cdot \left(\frac{2h_i}{\sin\theta_i} + b_i + b_{e.p}\right) / \gamma_{M5}$</td><td>$b_{e.p} = \frac{10}{b_0/t_0}\,b_i \le b_i$</td></tr>
<tr><td colspan="2">Brace failure</td></tr>
<tr><td>$N_{i.Rd} = f_{yi}\,t_i\,(2h_i - 4t_i + b_i + b_{eff}) / \gamma_{M5}$</td><td>$b_{eff} = \frac{10}{b_0/t_0} \cdot \frac{f_{y0} \cdot t_0}{f_{yi} \cdot t_i} \cdot b_i \le b_i$</td></tr>
<tr><td colspan="2">If circular brace members, multiply above resistance values by $\pi/4$ and replace b_i and h_i with the diameter d_i.</td></tr>
<tr><td colspan="2">KT joint:
- For each brace member, check following condition:
$N_{i.Rd} \ge N_{i.Ed}$ (i = 1,2,3)
- Resistances for the brace members are calculated as for N joint using adjacent brace members in pairs as follows:
A) brace members 1 & 3
B) brace members 2 & 3
If brace member 3 is in tension, its resistance is calculated from Case A.
If brace member 3 is in compression, its resistance is calculated from Case B.</td></tr>
<tr><td colspan="2">a) If $g/b_0 > 1{,}5(1-\beta)$ and $g \ge t_1 + t_2$ treat the joint as two separate T or Y joint.</td></tr>
<tr><td>Reduction factor for above resistances:</td><td colspan="2">S235 - S355: 1,0
S420 - S460: 0,9</td></tr>
</table>

Table 11.3.21 Resistance of overlap N and K joints with intermediate plate reinforcement [1...5]:
- chords are square or rectangular hollow sections
- brace members are square, rectangular or circular hollow sections



N_1 always compression
N_2 always tension
N_0 the bigger of chord normal force (left/right) absolute values

<table>
<tr><th>Resistance [i = 1 or 2]</th><th>Parameters</th><th>Validity range</th></tr>
<tr><td colspan="2">25 % $\le \lambda_{ov} <$ 50 % , Brace failure</td><td rowspan="8">In general:
$30^\circ \le \theta_i \le 90^\circ$
$t_0 \ge 2{,}5$ mm
$t_i \ge 2{,}5$ mm
$L_i \ge 6h_i$ or $6d_i$
$-0{,}55 \le e/h_0 \le 0{,}25$

Square and rectangular brace members:
- in general:
$b_i/b_0 \ge 0{,}25$
$0{,}5 \le h_i/b_i \le 2{,}0$
$0{,}75 \le b_i/b_p \le 1{,}0$
$(t_i f_{yi}) / (t_p f_{yp}) \le 1{,}0$
- tension brace member:
$b_i/t_i \le 35$
$h_i/t_i \le 35$
- compression brace member:
Class 1 or 2
(based on pure axial compression)

Circular brace members:
- in general:
$0{,}4 \le d_i/b_0 \le 0{,}8$
$0{,}75 \le d_i/b_p \le 1{,}0$
$(t_i f_{yi}) / (t_p f_{yp}) \le 1{,}0$
- tension brace member:
$d_i/t_i \le 50$
- compression brace member:
Class 1
(based on pure axial compression)

Chords:
$b_0/t_0 \le 35$
$h_0/t_0 \le 35$
$0{,}5 \le h_0/b_0 \le 2{,}0$
Class 1 or 2
(based on pure axial compression)

Plate:
$f_{yp} \ge f_{y0}$
$t_p \ge 2t_i$

Overlap:
25 % $\le \lambda_{ov} <$ 80 % a)</td></tr>
<tr><td>$N_{i.Rd} = f_{yi} t_i \left(b_{eff} + b_{e.ov} + \frac{\lambda_{ov}}{50} \cdot 2h_i - 4t_i\right) / \gamma_{M5}$</td><td>$\lambda_{ov} = \frac{q_i \cdot \sin\theta_i}{h_i} \cdot 100\ \%$
$b_{eff} = \frac{10}{b_0/t_0} \cdot \frac{f_{y0} \cdot t_0}{f_{yi} \cdot t_i} \cdot b_i \le b_i$
$b_{e.ov} = \frac{10}{b_p/t_p} \cdot \frac{f_{yp} \cdot t_p}{f_{yi} \cdot t_i} \cdot b_i \le b_i$</td></tr>
<tr><td colspan="2">50 % $\le \lambda_{ov} <$ 80 % , Brace failure</td></tr>
<tr><td>$N_{i.Rd} = f_{yi} t_i \cdot (b_{eff} + b_{e.ov} + 2h_i - 4t_i) / \gamma_{M5}$</td><td>$\lambda_{ov} = \frac{q_i \cdot \sin\theta_i}{h_i} \cdot 100\ \%$
$b_{eff} = \frac{10}{b_0/t_0} \cdot \frac{f_{y0} \cdot t_0}{f_{yi} \cdot t_i} \cdot b_i \le b_i$
$b_{e.ov} = \frac{10}{b_p/t_p} \cdot \frac{f_{yp} \cdot t_p}{f_{yi} \cdot t_i} \cdot b_i \le b_i$</td></tr>
<tr><td colspan="2"></td></tr>
<tr><td colspan="2">If circular brace members, multiply above resistance values by $\pi/4$ and replace b_i and h_i with the diameter d_i.</td></tr>
<tr><td colspan="2">a) If the braces are rectangular sections with $h_1 < b_1$ and/or $h_2 < b_2$, the connection between braces and chord has to be checked for shear.</td></tr>
<tr><td>Reduction factor for above resistances:</td><td>S235 - S355: 1,0
S420 - S460: 0,9</td></tr>
</table>

References

[1] SFS-EN 1993-1-8:2005. (EN 1993-1-8:2005)
Eurocode 3. Design of steel structures.
Part 1-8: Design of joints.
Finnish Standards Association SFS. 149 pages.

[2] SFS-EN 1993-1-8:2005+AC:2005. (EN 1993-1-8:2005+AC:2005)
Includes also corrigendum AC:2005.
Finnish Standards Association SFS. 137 pages.

[3] SFS-EN 1993-1-8: AC:2009. (EN 1993-1-8: AC:2009)
Corrigendum AC:2009 for the standard EN 1993-1-8.
Finnish Standards Association SFS. 21 pages.

[4] CIDECT. 1991. Design guide for circular hollow section (CHS) joints under predominantly static loading. CIDECT Design Guide No 1.
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[5] CIDECT. 1992. Design guide for rectangular hollow section (RHS) joints under predominantly static loading. CIDECT Design Guide No 3.
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Annex 11.4 Estimating the stiffness of semi-rigid joints in frames

In frame structures the stiffness of the joints can be taken into account, causing a change for the bending moments transferred to the beam span. The following formulas are obtained for the bending moments of a beam subjected to uniform load and with a semi-rigid joint at both ends, when the supports are assumed non-deflecting [1,2]:

$$M_1 = \frac{S_j}{S_j + K} \cdot \frac{qL^2}{12} \qquad \textit{end moment} \tag{11.4.1}$$

$$M_0 = \frac{S_j + 3K}{S_j + K} \cdot \frac{qL^2}{24} \qquad \textit{field moment} \tag{11.4.2}$$

where

$$K = \frac{2EI}{L}$$

E is the Young's modulus of elasticity
I is the second moment of area of the beam
L is the length of the beam
S_j is the rotational stiffness of the joint [Nm/rad]
q is the load per unit length applied on the beam [N/m]

The deflection and the rotation of the beam are determined from:

$$\phi_1 = \frac{M_1}{S_j} = \frac{1}{S_j + K} \cdot \frac{qL^2}{12} \qquad \textit{rotation at support} \tag{11.4.3}$$

$$\delta_0 = \frac{5qL^4}{384EI} \cdot \frac{K + S_j/5}{S_j + K} \qquad \textit{deflection at mid-span} \tag{11.4.4}$$

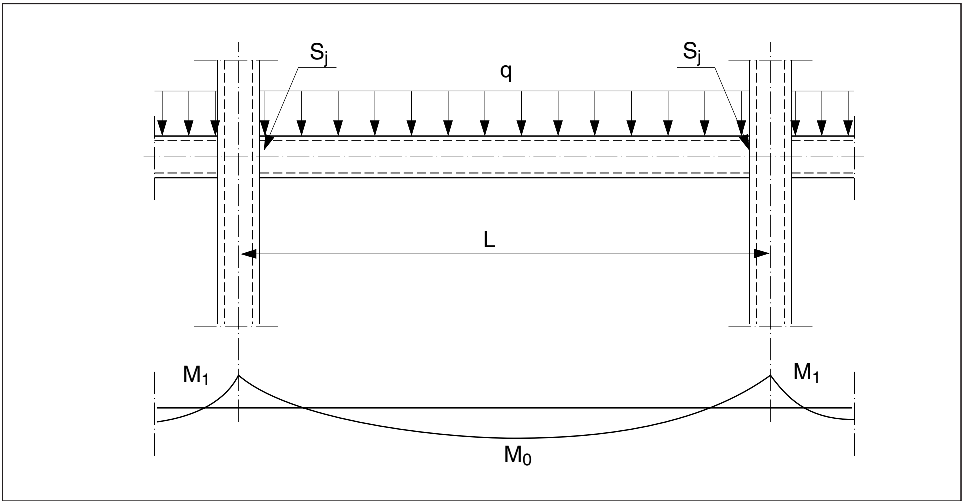


Figure 11.4.1 A beam with semi-rigid joints

The rotational stiffness S_j for a welded T joint in square or rectangular hollow sections can be determined according to guidance given in [3]:

$$S_j = \frac{1000 \cdot C^* \cdot t_0^3}{52} \tag{11.4.5}$$

where
- S_j is the rotational stiffness of the joint [Nm/rad]
- t_0 is the wall thickness of the hollow section [mm]
- C^* is the constant obtained from Figures 11.4.3 - 11.4.7 [N/mm^2]

With square hollow sections, the constant C^* is taken from Figure 11.4.3, when $b_1 / b_0 \leq 0{,}7$. In other cases it is taken from Figures 11.4.4 - 11.4.7.

With rectangular hollow sections, the constant C^* is determined as for square hollow sections. This result is finally multiplied by the correction factor obtained from Figure 11.4.7.

Formula (11.4.5) presents an approximation for the rotational stiffness, which best corresponds with the bending moment values of the joint, up to the yield moment ($M_{el.c}$) of the joint.

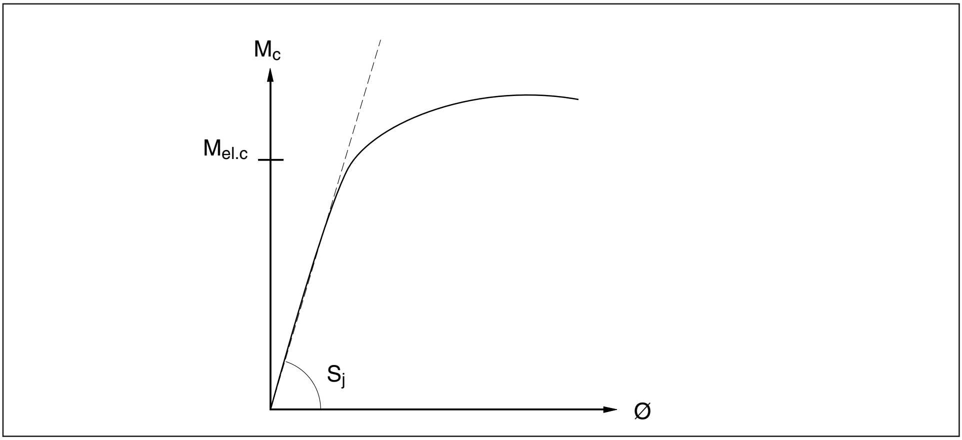


Figure 11.4.2 The moment-rotation curve of a semi-rigid joint

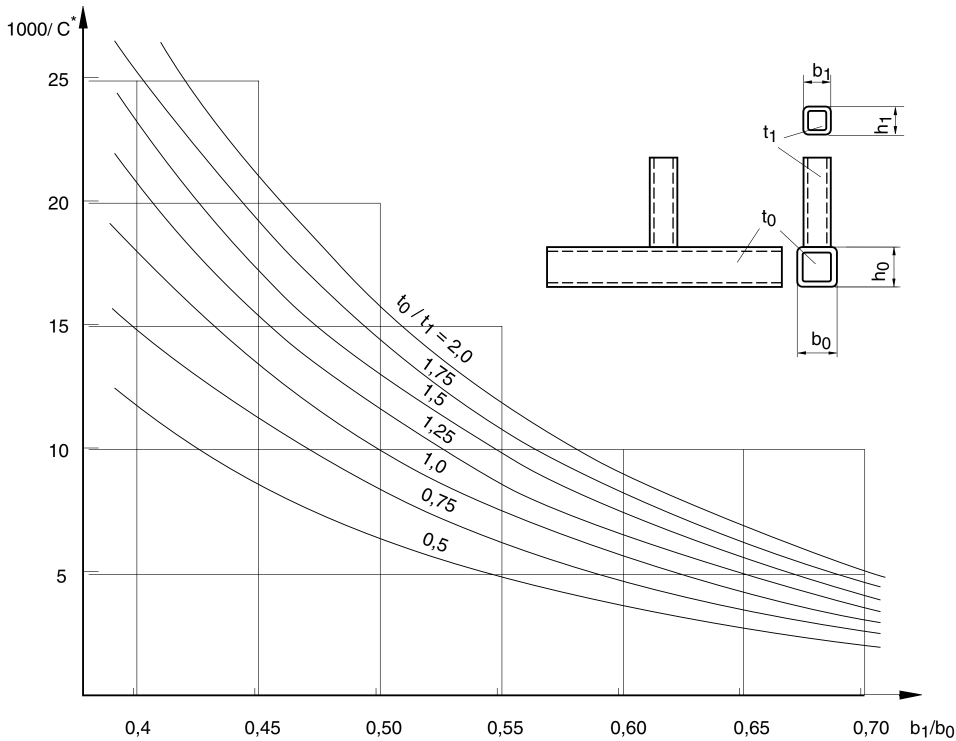


Figure 11.4.3 Values of constant $1000/C^*$ for T joints in square hollow sections, when $b_1/b_0 \le 0{,}7$ and $b_0/t_0 \ge 10$ [3]

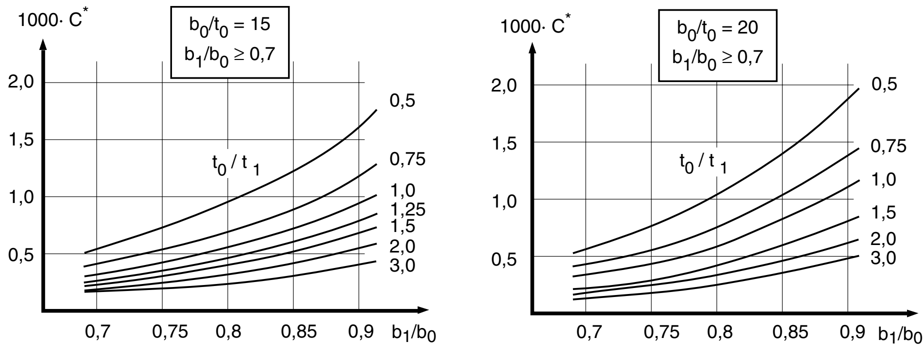


Figure 11.4.4 Values of constant C* for T joints in square hollow sections, when $b_1 / b_0 \geq 0{,}7$ and $b_0 / t_0 = 10$ or $b_0 / t_0 = 20$ [3]

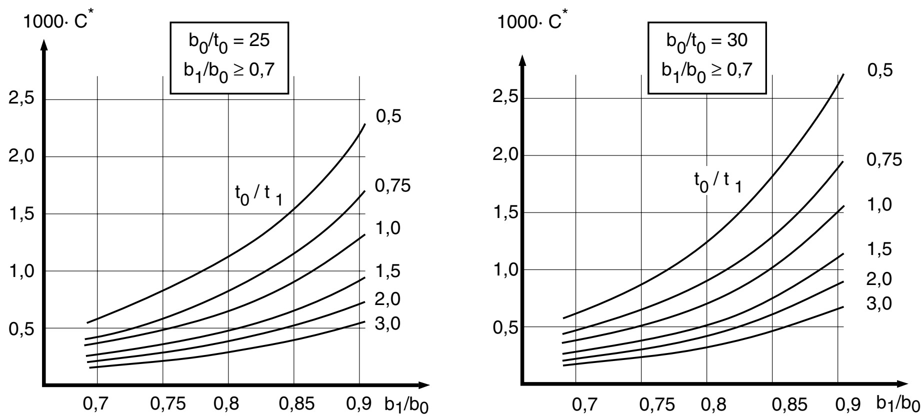
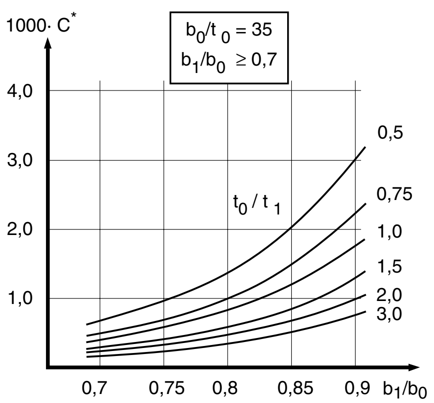


Figure 11.4.5 Values of constant C* for T joints in square hollow sections, when $b_1 / b_0 \geq 0{,}7$ and $b_0 / t_0 = 25$ or $b_0 / t_0 = 30$ [3]



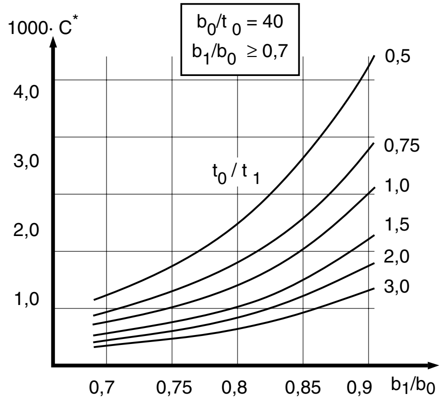


Figure 11.4.6 Values of constant C* for T joints in square hollow sections, when $b_1 / b_0 \geq 0,7$ and $b_0 / t_0 = 35$ or $b_0 / t_0 = 40$ [3]

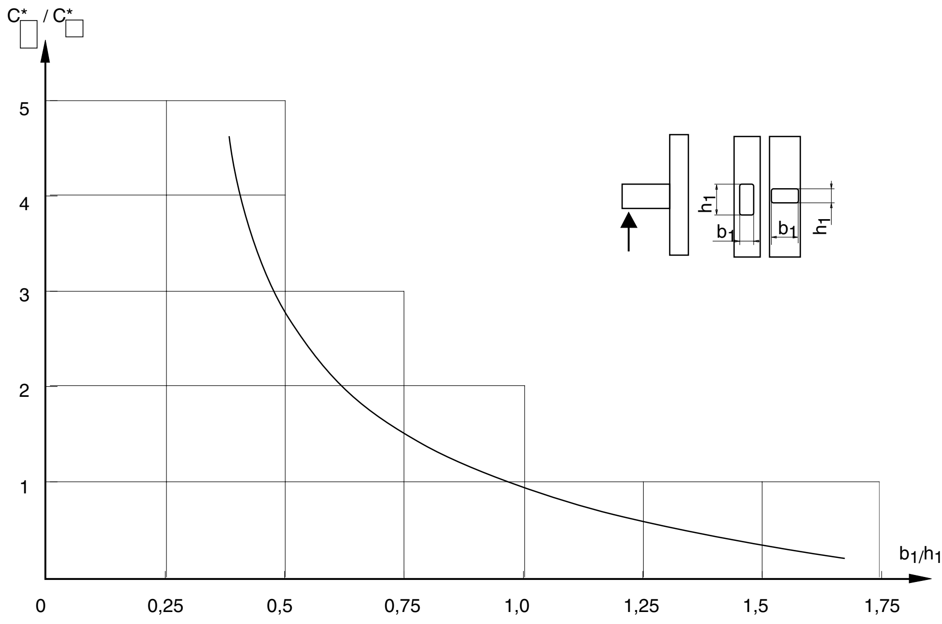


Figure 11.4.7 Values of constant C* for rectangular hollow sections to those for square hollow sections [3]

References

[1] ECCS. 1992. Analysis and Design of Steel frames with Semi-Rigid Joints.
ECCS Technical Committee 8 - Structural Stability.
Technical Working Group 8.1/8.2 Skeletal Structures.
ECCS No 67. First edition. 80 pages.

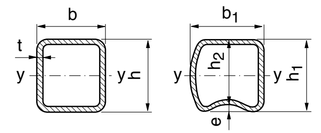
[2] CIDECT. 2004. Design guide for structural hollow section column connections.
CIDECT Design Guide No 9. Verlag TÜV Rheinland GmbH. 214 pages.

[3] CIDECT. 1983. The development of recommendations for the design of
welded joints between steel structural hollow sections (T- and X-type joints).
Final report on ECSC Agreement 7210 SA/109 and CIDECT Program 5AD.

Annex 11.5 Minimum bending radii for square and rectangular hollow sections

Tables 11.5.1 and 11.5.2 present the minimum bending radii for square and rectangular hollow sections when bending is made at room temperature in 3-roller cold bending. **The values in these tables are guideline minimum values that can be obtained with good equipment and careful workmanship.**

Table 11.5.1 Guideline values given in [1] for minimum bending radii of square hollow sections in 3-roller cold bending.
The sizes may differ from SSAB's manufacturing programme



a) before bending b) after bending

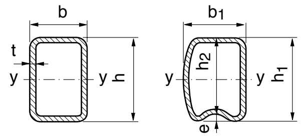
$$P_b = \frac{b_1 - b}{b} 100$$

$$P_e = \frac{e}{h} 100$$

P_b is the widening of the cross-section [%]
P_e is the concavity of the compression flange [%]
R_t is the internal bending radius [m]
I_{red} is the reduction of the second moment of area I_y due to cross-section distortion [%]

			P_b = 1 %		P_b = 2,5 %		P_b = 5 %		P_b = 7,5 %		P_e = 0,5 %		P_e = 1 %		P_e = 2,5 %		P_e = 5 %	
h	b	t	R_t	I_{red}	R_t	I_{red}	R_t	I_{red}	R_t	I_{red}	R_t	I_{red}	R_t	I_{red}	R_t	I_{red}	R_t	I_{red}
mm	mm	mm	m	%	m	%	m	%	m	%	m	%	m	%	m	%	m	%
40	40	4,0	1,44	1	0,31	1	0,22	1	0,22	1	0,22	1	0,22	2	0,22	4	0,22	8
50	50	4,0	2,88	1	0,61	2	0,22	3	0,22	3	0,79	1	0,29	2	0,22	4	0,22	7
50	50	5,0	2,72	1	0,58	1	0,22	2	0,22	2	0,43	1	0,22	2	0,22	4	0,22	8
60	60	4,0	5,07	1	1,08	2	0,34	3	0,22	5	2,55	1	0,95	2	0,26	4	0,22	7
60	60	5,0	4,79	1	1,02	2	0,32	3	0,22	3	1,37	1	0,51	2	0,22	4	0,22	7
70	70	4,0	8,17	1	1,74	3	0,54	5	0,27	7	6,85	1	2,55	2	0,69	4	0,26	7
70	70	5,0	7,72	1	1,65	2	0,51	3	0,26	6	3,68	1	1,37	2	0,37	4	0,22	7
80	80	4,0	12,36	2	2,64	3	0,82	6	0,41	8	16,12	1	5,99	2	1,62	4	0,60	6
80	80	5,0	11,68	1	2,49	3	0,77	5	0,39	7	8,66	1	3,22	2	0,87	4	0,32	7
80	80	6,3	11,02	1	2,35	2	0,73	3	0,73	5	4,55	1	1,69	2	0,46	4	0,22	7
90	90	4,0	17,80	2	3,80	3	1,18	7	0,60	10	34,29	1	12,75	2	3,45	3	1,28	6
90	90	5,0	16,83	2	3,59	3	1,11	6	0,56	8	18,42	1	6,85	2	1,85	4	0,69	7
90	90	6,3	15,87	1	3,38	3	1,05	4	0,53	6	9,68	1	3,60	2	0,97	4	0,36	7
100	100	4,0	24,67	2	5,26	4	1,63	8	0,82	12	67,38	1	25,05	2	6,77	3	2,52	6
100	100	5,0	23,32	2	4,97	4	1,54	6	0,78	9	36,20	1	13,46	2	3,64	4	1,35	6
100	100	6,3	22,00	1	4,69	3	1,46	6	0,74	7	19,03	1	7,07	2	1,91	4	0,71	7
100	100	8,0	20,71	1	4,42	2	1,37	3	0,69	6	9,78	1	3,64	2	0,98	4	0,37	7
120	120	4,0	43,41	3	9,26	6	2,88	12	1,45	18	216,84	1	80,60	2	21,79	3	8,10	6
120	120	5,0	41,03	3	8,75	5	2,72	8	1,37	13	116,51	1	43,31	2	11,71	3	4,35	6
120	120	6,3	38,70	2	8,25	4	2,56	7	1,29	11	61,23	1	22,76	2	6,15	4	2,29	7
120	120	8,0	36,44	1	7,77	3	2,41	6	1,22	7	31,49	1	11,70	2	3,16	4	1,18	7
150	150	5,0	81,91	3	17,47	7	5,43	13	2,74	20	487,11	1	181,06	2	48,94	3	18,19	6
150	150	6,3	77,27	3	16,48	5	5,12	9	2,58	14	255,98	1	95,15	2	25,72	3	9,56	6
150	150	8,0	72,75	2	15,51	4	4,82	7	2,43	11	131,64	1	48,93	2	13,23	4	4,92	7
150	150	10,0	68,76	2	14,66	3	4,55	6	2,30	8	70,73	1	26,29	2	7,11	4	2,64	7
180	180	6,3	123,65	3	20,04	6	5,06	11	2,26	17	831,60	1	251,71	2	51,86	3	15,70	6
180	180	8,0	81,47	3	13,21	6	3,33	10	1,49	15	416,40	1	126,03	2	25,96	4	7,86	6
180	180	10,0	55,18	3	8,94	6	2,26	10	1,01	14	218,22	1	66,05	2	13,61	4	4,12	7
200	200	6,3	176,76	3	28,66	6	7,24	10	3,23	16	1046,76	1	316,83	2	65,27	3	19,76	6
200	200	8,0	116,47	3	18,88	5	4,77	9	2,13	14	524,13	1	158,64	2	32,68	3	9,89	6
200	200	10,0	78,88	3	12,79	5	3,23	9	1,44	13	274,69	1	83,14	2	17,13	4	5,18	7
250	250	6,3	376,81	3	61,08	5	15,42	9	6,89	14	1704,11	1	515,80	2	106,26	3	32,16	6
250	250	8,0	248,27	3	40,25	4	10,16	8	4,54	12	853,27	1	258,27	2	53,21	3	16,10	6
250	250	10,0	168,15	2	27,26	4	6,88	7	3,08	10	447,18	1	135,35	2	27,88	3	8,44	6
300	300	8,0	460,71	2	74,69	4	18,86	7	8,43	11	1270,17	1	384,45	2	79,20	3	23,97	6
300	300	10,0	312,03	2	50,58	3	12,77	6	5,71	9	665,71	1	201,50	2	41,51	3	12,56	6

Table 11.5.2 Guideline values given in [1] for minimum bending radii of rectangular hollow sections in 3-roller cold bending.
The sizes may differ from SSAB's manufacturing programme



a) before bending b) after bending

$$P_b = \frac{b_1 - b}{b} 100$$

$$P_e = \frac{e}{h} 100$$

P_b is the widening of the cross-section [%]
P_e is the concavity of the compression flange [%]
R_t is the internal bending radius [m]
I_{red} is the reduction of the second moment of area I_y due to cross-section distortion [%]

			P_b = 1 %		P_b = 2,5 %		P_b = 5 %		P_b = 7,5 %		P_e = 0,5 %		P_e = 1 %		P_e = 2,5 %		P_e = 5 %	
H	B	T	R_t	I_{red}	R_t	I_{red}	R_t	I_{red}	R_t	I_{red}	R_t	I_{red}	R_t	I_{red}	R_t	I_{red}	R_t	I_{red}
mm	mm	mm	m	%	m	%	m	%	m	%	m	%	m	%	m	%	m	%
50	30	4,0	2,18	1	0,47	1	0,22	2	0,22	2	0,26	1	0,22	2	0,22	4	0,22	7
60	40	4,0	4,07	1	0,87	2	0,27	3	0,22	3	1,06	1	0,39	2	0,22	3	0,22	6
80	40	4,0	8,48	1	1,81	2	0,56	3	0,28	4	3,59	1	1,34	2	0,36	3	0,22	5
80	40	5,0	8,01	1	1,71	2	0,53	3	0,27	3	1,93	1	0,72	2	0,22	3	0,22	6
90	50	4,0	12,93	1	2,76	3	0,86	4	0,43	5	9,61	1	3,57	2	0,97	3	0,36	5
90	50	5,0	12,22	1	2,61	2	0,81	3	0,41	4	5,16	1	1,92	2	0,52	3	0,22	6
100	50	4,0	16,92	1	3,61	2	1,12	5	0,57	5	15,02	1	5,58	1	1,51	3	0,56	5
100	50	5,0	15,99	1	3,41	2	1,06	3	0,53	5	8,07	1	3,00	2	0,81	3	0,30	5
100	60	4,0	18,69	2	3,98	3	1,24	5	0,62	7	22,30	1	8,29	2	2,24	3	0,83	5
100	60	5,0	17,66	1	3,77	3	1,17	4	0,59	6	11,98	1	4,45	2	1,20	3	0,45	6
100	60	6,3	16,66	1	3,55	2	1,10	3	0,56	4	6,30	1	2,34	2	0,63	3	0,24	6
120	60	4,0	29,77	1	6,35	3	1,97	5	1,00	8	48,35	1	17,97	1	4,86	3	1,81	5
120	60	5,0	28,14	1	6,00	3	1,86	5	0,94	6	25,98	1	9,66	1	2,61	3	0,97	5
120	60	6,3	26,54	1	5,66	2	1,76	3	0,89	6	13,65	1	5,07	2	1,37	3	0,51	6
120	80	4,0	34,81	2	7,42	4	2,31	7	1,16	11	90,14	1	33,50	2	9,06	3	3,37	5
120	80	5,0	32,90	2	7,02	3	2,18	6	1,10	9	48,43	1	18,00	2	4,87	3	1,81	6
120	80	6,3	31,04	1	6,62	3	2,06	4	1,04	6	25,45	1	9,46	2	2,56	3	0,95	6
150	100	4,0	69,50	3	14,82	5	4,60	11	2,32	18	376,84	1	140,08	2	37,86	3	14,07	5
150	100	5,0	65,69	3	14,01	5	4,35	8	2,20	12	202,48	1	75,26	2	20,34	3	7,56	5
150	100	6,3	61,97	2	13,21	3	4,10	6	2,07	9	106,41	1	39,55	2	10,69	3	3,97	6
150	100	8,0	58,34	1	12,44	3	3,86	6	1,95	7	54,72	1	20,34	2	5,50	3	2,04	6
160	80	4,0	176,59	1	28,63	2	7,23	5	3,23	7	244,29	1	73,94	1	15,23	3	4,61	5
160	80	5,0	119,60	1	19,39	2	4,90	5	2,19	6	128,03	1	38,75	1	7,98	3	2,42	5
160	80	6,3	79,88	1	12,95	2	3,27	4	1,46	5	65,56	1	19,85	1	4,09	3	1,24	5
160	80	8,0	52,63	1	8,53	3	2,15	3	0,96	5	32,83	1	9,94	2	2,05	3	0,62	5
200	100	5,0	254,95	1	41,33	3	10,44	4	4,66	5	208,42	1	63,09	1	13,00	3	3,93	5
200	100	6,3	170,28	1	27,60	3	6,97	3	3,12	5	106,74	1	32,31	1	6,66	3	2,01	5
200	100	8,0	112,19	1	18,19	2	4,59	3	2,05	5	53,45	1	16,18	2	3,33	3	1,01	5
200	100	10,0	75,98	1	12,32	2	3,11	3	1,39	4	28,01	1	8,48	2	1,75	3	0,53	6
250	150	6,3	366,57	1	59,42	3	15,00	4	6,71	5	316,79	1	95,89	1	19,75	3	5,98	5
250	150	8,0	241,53	1	39,15	3	9,89	3	4,42	5	158,62	1	48,01	2	9,89	3	2,99	5
250	150	10,0	163,58	1	26,52	2	6,70	3	2,99	5	83,13	1	25,16	2	5,18	3	1,57	6
300	200	6,3	684,25	1	110,92	3	28,01	5	12,52	6	667,45	1	202,02	1	41,62	3	12,60	5
300	200	8,0	450,84	1	73,09	3	18,45	4	8,25	5	334,20	1	101,16	2	20,84	3	6,31	5
300	200	10,0	305,34	1	49,50	3	12,50	3	5,59	5	175,15	1	53,01	2	10,92	3	3,31	6
400	200	8,0	1177,85	1	190,94	1	48,21	2	21,55	3	242,86	1	73,51	1	15,14	3	4,58	5
400	200	10,0	797,71	1	129,32	1	32,65	3	14,60	3	127,28	1	38,52	1	7,94	3	2,40	5

References

[1] CIDECT. 1988. Minimum bending radii for square and rectangular hollow sections (3-roller cold bending). CIDECT report 11C-88/14E.

Annex 11.6 Design software for truss and frame structures - *WinRami*, *Section* and *Fire Protection System*

WinRami and ***Section*** softwares are intended for structural analysis and design of uniplanar truss and frame structures according to Eurocode 3 (EN 1993). The programs can be applied for cold-formed structural hollow sections, welded I-sections (in symmetrical and unsymmetrical shape) and hot-rolled HEA, HEB and IPE sections.

The actions and displacements of the structure are solved by ***WinRami*** and the structural design of the beam members is performed by the accompanying ***Section*** software. ***WinRami*** and ***Section*** softwares (as well as all the additional program elements included) are linked to each others through MS Windows OLE2-link, allowing the user to see them as only one program.

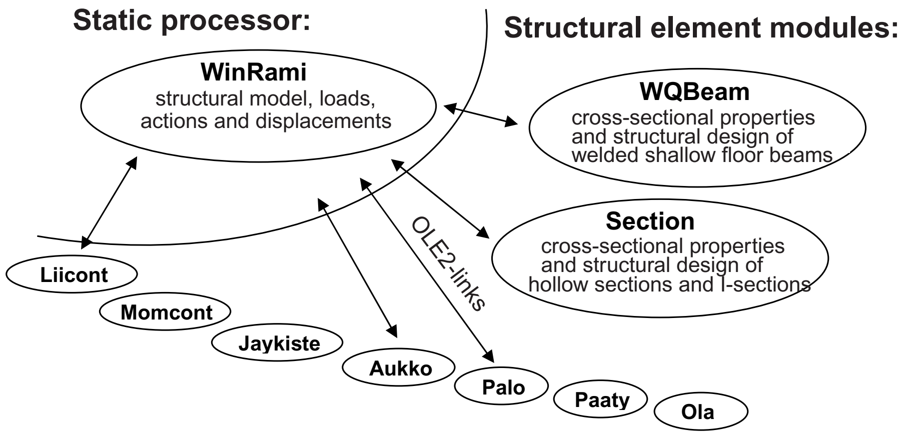
Section software has in its pre-programmed data-tables the complete set of SSAB's structural hollow sections, as well as the hot-rolled I-sections stated above. For the welded I-sections the cross-sectional dimensions can be defined entirely by the user.

WinRami software includes two program elements covering the design of joints:

- ***Liicont***: hollow section welded truss joints
- ***Momcont***: hollow section beam-to-column moment-resisting welded joints

Also following program elements are included in ***WinRami*** software:

- ***Palo***: structural design of fire protection (shall be installed with a separate ***Fire Protection System*** software)
- ***Paaty***: structural design of transverse stiffeners (end post / support) for an I-section
- ***Jaykiste***: structural design of longitudinal and transverse stiffeners for an I-section
- ***Aukko***: structural design of web openings of an I-section



WinRami software includes also macros for generating the structural model of truss structures and their joints. Hollow section lattice structures with N, K and KT joints are fast and easy to design by using these macros. The user only needs to define the main input-parameters in the macro (span length, structural height etc.), and after that ***WinRami*** generates the structural model, adds the loads, calculates the load combinations and preliminary defines the joints. The user selects any preferred steel sections from ***Section*** software, or utilizes a special macro which helps to define all the preferred sections in one action. More information about ***WinRami*** software can be found on Ruukki's web site [www.ruukki.com].

Annex 11.7 Design software for composite columns – *ColGraph*

ColGraph software is intended for structural design of concrete filled hollow section columns. The program calculates parametric-based capacity curves of concentrically loaded concrete filled steel composite columns according to design guidance presented in the FCSA-publication ‘Betonitäytteisen teräsliittopilarin suunnitteluohje’.

With the applied flexural buckling length and load of the column, the program can be used to define the appropriate cross-section of the composite column (square, rectangular or circular hollow section) and the amount of the reinforcing steel as required. The user may also define the spacing of the reinforcing steel to be different than determined in above referred FCSA-publication.

The capacity curves are calculated according to FCSA-publication ‘Betonitäytteisen teräsliittopilarin suunnitteluohje’ (published in 2004), which is based on Part EN 1994-1-2 of Eurocode. The column is assumed to be concentrically loaded. The capacity curves are defined for ambient temperature as well as for R30 - R120 standard-fire. When determining the capacity curves, the program allows to apply the limitation for the flexural buckling slenderness (lambda limit).

The main features of the program are:

- selection of strength classes of the materials (hollow sections, reinforcing steels, concrete)
- adjustment of concrete cover for main reinforcing steels
- allowance for local buckling
- ambient temperature and R30 - R120 standard-fire
- viewing of the selected capacity curves, then print-outs
- the program complements the FCSA-publication ‘Betonitäytteisen teräsliittopilarin suunnitteluohje’
- reference code is Eurocode 4 (EN 1994)
- user interface is written in finnish language

More information about ***ColGraph*** software can be found on Ruukki’s web site [www.ruukki.com].

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