



Design of Steel Structures

2nd Edition

Eurocode 3: Design of Steel Structures
Part 1-1: General rules and rules for buildings

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TABLE OF CONTENTS

FOREWORD	xiii
PREFACE	xv
 Chapter 1	
INTRODUCTION	1
1.1. General Observations	1
1.2. Codes of Practice and Normalization	3
1.2.1. Introduction	3
1.2.2. Eurocode 3	6
1.2.3. Other standards	7
1.3. Basis of Design	8
1.3.1. Basic concepts	8
1.3.2. Reliability management	9
1.3.3. Basic variables	13
1.3.3.1. <i>Introduction</i>	13
1.3.3.2. <i>Actions and environmental influences</i>	13
1.3.3.3. <i>Material properties</i>	14
1.3.3.4. <i>Geometrical data</i>	15
1.3.4. Ultimate limit states	15
1.3.5. Serviceability limit states	16
1.3.6. Durability	18
1.3.7. Sustainability	19
1.4. Materials	21

TABLE OF CONTENTS

1.4.1. Material specification	21
1.4.2. Mechanical properties	22
1.4.3. Toughness and through thickness properties	25
1.4.4. Fatigue properties	27
1.4.5. Corrosion resistance	27
1.5. Geometric Characteristics and Tolerances	28

Chapter 2

STRUCTURAL ANALYSIS **33**

2.1. Introduction	33
2.2. Structural Modelling	34
2.2.1. Introduction	34
2.2.2. Choice of member axis	36
2.2.3. Influence of eccentricities and supports	38
2.2.4. Non-prismatic members and members with curved axis	39
2.2.5. Influence of joints	44
2.2.6. Combining beam elements together with two and three dimensional elements	51
2.2.7. Worked examples	52
2.3. Global Analysis of Steel Structures	75
2.3.1. Introduction	75
2.3.2. Structural stability of frames	77
2.3.2.1. <i>Introduction</i>	77
2.3.2.2. <i>Elastic critical load</i>	80
2.3.2.3. <i>2nd order analysis</i>	86
2.3.3. Imperfections	88
2.3.4. Worked example	94
2.4. Classification of Cross Sections	108

Chapter 3

DESIGN OF MEMBERS	119
3.1. Introduction	119
3.1.1. General	119
3.1.2. Resistance of cross sections	120
3.1.2.1. <i>General criteria</i>	120
3.1.2.2. <i>Section properties</i>	121
3.1.3. Buckling resistance of members	125
3.2. Tension	125
3.2.1. Behaviour in tension	125
3.2.2. Design for tensile force	128
3.2.3. Worked examples	131
3.3. Laterally Restrained Beams	138
3.3.1. Introduction	138
3.3.2. Design for bending	139
3.3.2.1. <i>Elastic and plastic bending moment resistance</i>	139
3.3.2.2. <i>Uniaxial bending</i>	141
3.3.2.3. <i>Bi-axial bending</i>	142
3.3.2.4. <i>Net area in bending</i>	142
3.3.3. Design for shear	143
3.3.4. Design for combined shear and bending	144
3.3.5. Worked examples	146
3.4. Torsion	157
3.4.1. Theoretical background	157
3.4.1.1. <i>Introduction</i>	157
3.4.1.2. <i>Uniform torsion</i>	159
3.4.1.3. <i>Non-uniform torsion</i>	161

TABLE OF CONTENTS

3.4.1.4. <i>Cross section resistance in torsion</i>	166
3.4.2. Design for torsion	173
3.4.3. Worked examples	176
3.5. Compression	186
3.5.1. Theoretical background	186
3.5.1.1. <i>Introduction</i>	186
3.5.1.2. <i>Elastic critical load</i>	186
3.5.1.3. <i>Effect of imperfections and plasticity</i>	192
3.5.2. Design for compression	198
3.5.3. Worked examples	204
3.6. Laterally Unrestrained Beams	212
3.6.1. Introduction	212
3.6.2. Lateral-torsional buckling	212
3.6.2.1. <i>Introduction</i>	212
3.6.2.2. <i>Elastic critical moment</i>	213
3.6.2.3. <i>Effect of imperfections and plasticity</i>	223
3.6.3. Lateral-torsional buckling resistance	225
3.6.4. Worked examples	229
3.7. Beam-Columns	242
3.7.1. Introduction	242
3.7.2. Classification of cross sections under bending and axial force	243
3.7.3. Cross section resistance	247
3.7.3.1. <i>Theoretical background</i>	247
3.7.3.2. <i>Design resistance</i>	249
3.7.4. Buckling resistance	253
3.7.4.1. <i>Theoretical background</i>	253
3.7.4.2. <i>Design resistance</i>	256
3.7.5. Worked examples	265

Chapter 4

ELASTIC DESIGN OF STEEL STRUCTURES	293
4.1. Introduction	293
4.2. Simplified Methods of Analysis	295
4.2.1. Introduction	295
4.2.2. Amplified sway-moment method	297
4.2.3. Sway-mode buckling length method	299
4.2.4. Worked example	300
4.3. Member Stability of Non-prismatic Members and Components	310
4.3.1. Introduction	310
4.3.2. Non-prismatic members	310
4.3.3. Members with intermediate restraints	316
4.3.4. General method	322
4.3.5. Worked example	325
4.4. Design Example 1: Elastic Design of Braced Steel-Framed Building	340
4.4.1. Introduction	340
4.4.2. Description of the structure	342
4.4.3. General safety criteria, actions and combinations of actions	344
4.4.3.1. <i>General safety criteria</i>	344
4.4.3.2. <i>Permanent actions</i>	345
4.4.3.3. <i>Imposed loads</i>	345
4.4.3.4. <i>Wind actions</i>	346
4.4.3.5. <i>Summary of basic actions</i>	353
4.4.3.6. <i>Frame imperfections</i>	353
4.4.3.7. <i>Load combinations</i>	356
4.4.3.8. <i>Load arrangement</i>	358
4.4.4. Structural analysis	359

TABLE OF CONTENTS

4.4.4.1. <i>Structural model</i>	359
4.4.4.2. <i>Linear elastic analysis</i>	360
4.4.4.3. <i>Susceptibility to 2nd order effects: elastic critical loads</i>	361
4.4.4.4. <i>2nd order elastic analysis</i>	362
4.4.5. <i>Design checks</i>	363
4.4.5.1. <i>General considerations</i>	363
4.4.5.2. <i>Cross section resistance</i>	365
4.4.5.3. <i>Buckling resistance of beams</i>	366
4.4.5.4. <i>Buckling resistance of columns and beam-columns</i>	366
Chapter 5	
PLASTIC DESIGN OF STEEL STRUCTURES	367
5.1. General Principles for Plastic Design	367
5.1.1. Introduction	367
5.1.2. Plastic limit analysis: method of mechanisms	368
5.1.3. Code requirements for plastic analysis	372
5.2. Methods of Analysis	376
5.2.1. Introduction	376
5.2.2. Approximate methods for pre-design	376
5.2.3. Computational analysis	388
5.2.4. 2 nd order effects	393
5.2.4.1. <i>Introduction</i>	393
5.2.4.2. <i>Elastic critical load</i>	394
5.2.4.3. <i>2nd order computational analysis</i>	397
5.2.4.4. <i>Simplified methods for analysis</i>	397
5.2.5. Worked example	400
5.3. Member Stability and Buckling Resistance	410

TABLE OF CONTENTS

5.3.1. Introduction	410
5.3.2. General criteria for the verification of the stability of members with plastic hinges	410
5.3.3. Bracings	411
5.3.4. Verification of the stability of members with plastic hinges	414
5.3.4.1. <i>Introduction</i>	414
5.3.4.2. <i>Prismatic members constituted by hot-rolled or equivalent welded I sections</i>	415
5.3.4.3. <i>Haunched or tapered members made of rolled or equivalent welded I sections</i>	417
5.3.4.4. <i>Modification factors for moment gradients in members laterally restrained along the tension flange</i>	420
5.3.5. Worked examples	423
5.4. Design Example 2: Plastic Design of Industrial Building	432
5.4.1. Introduction	432
5.4.2. General description	433
5.4.3. Quantification of actions, load combinations and general safety criteria	434
5.4.3.1. <i>General criteria</i>	434
5.4.3.2. <i>Permanent actions</i>	434
5.4.3.3. <i>Imposed loads</i>	434
5.4.3.4. <i>Snow loads</i>	435
5.4.3.5. <i>Wind loads</i>	435
5.4.3.6. <i>Summary of basic actions</i>	440
5.4.3.7. <i>Imperfections</i>	440
5.4.3.8. <i>Load combinations</i>	441
5.4.4. Pre-design	443
5.4.5. Structural analysis	446

TABLE OF CONTENTS

5.4.5.1. <i>Linear elastic analysis</i>	446
5.4.5.2. <i>2nd order effects</i>	448
5.4.5.3. <i>Elastic-plastic analysis</i>	449
5.4.6. Code checks	451
5.4.6.1. <i>General considerations</i>	451
5.4.6.2. <i>Cross section resistance</i>	451
5.4.6.3. <i>Buckling resistance of the rafters</i>	451
5.4.6.4. <i>Buckling resistance of the columns</i>	454
5.4.7. Synthesis	454
REFERENCES	455

Annex A

FORMULAS FOR COMMON TORSIONAL CASES	465
A.1. Cross Sectional Properties for Torsion	465
A.2. Solution of Differential Equation for Torsion	467
A.2.1 Concentrated torsional moment	467
A.2.2 Distributed torsional moment	474

xii**Annex B**

ELASTIC CRITICAL MOMENT	483
B.1. Abacus to Calculate the Coefficients C_1 , C_2 and C_3	483
B.1.1 Elastic critical moment in beams submitted to end moments simultaneously with transverse loads	483
B.1.2 Elastic critical moment of unbraced cantilevers	487
B.2. Alternative Equations for the Determination of the Elastic Critical Moment	490

FOREWORD

The development program for the design manuals of the European Convention for Constructional Steelwork (ECCS) represents a major effort for the steel construction industry and the engineering profession in Europe. Conceived by the ECCS Technical Activities Board under the leadership of its chairman, Professor Luis Simões da Silva, the manuals are being prepared in close agreement with the final stages of Eurocode 3 and its national Annexes. The scope of the development effort is vast, and reflects a unique undertaking in the world.

The publication of the first of the manuals, *Design of Steel Structures*, is a signal achievement which heralds the successful completion of the Eurocode 3 work and brings it directly to the designers who will implement the actual use of the code. As such, the book is more than a manual – it is a major textbook that details the fundamental concepts of the code and their practical application. It is a unique publication for a major construction market.

Following a discussion of the Eurocode 3 basis of design, including the principles of reliability management and the limit state approach, the steel material standards and their use under Eurocode 3 are detailed. Structural analysis and modeling are presented in a chapter that will assist the design engineer in the first stages of a design project. This is followed by a major chapter that provides the design criteria and approaches for the various types of structural members. The theories of behavior and strength are closely tied to the Eurocode requirements, making for a unique presentation of theory into practice. The following chapters expand on the principles and applications of elastic and plastic design of steel structures.

The many design examples that are presented throughout the book represent a significant part of the manual. These will be especially well received by the design profession. Without a doubt, the examples will facilitate the acceptance of the code and provide for a smooth transition from earlier national codes to the Eurocode.

Reidar Bjorhovde
Member, ECCS Editorial Board

PREFACE 2nd EDITION

The first edition of Design of Steel Structures was published by ECCS as a paperback in 2010. Since 2012, this publication is also available in electronic format as an e-book. The first edition was sold in over 100 countries and the interest for this publication was so high that a second edition would have to be printed.

The authors took the opportunity of this second edition to revise their manuscript. The standard that constitutes the object of this book, namely EN 1993-1-1, is still in application in the same versions as those that prevailed at the time of writing the first edition except for a minor amendment published in 2013. However, many comments were received by readers that resulted in the correction of some small mistakes and the rephrasing of some sentences or sections and the addition of some new material.

The new material comprises:

- A revised section dealing with the design for torsion of steel members, including a new worked example illustrating an open cross section beam subject to bending and torsional moments;
- A revised section on the elastic critical moment of beams;
- An improved explanation on the classification of cross sections subject to bending and axial force;
- An additional worked example of a beam-column with transversal loads and end moments;
- A new Annex containing formulas for common torsional cases;
- A revised and expanded Annex with formulas for elastic critical moment calculation.

xv

The authors are indebted to Profs. E. Mirambell and K. Rasmussen for their thorough revision of the section on torsion.

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Coimbra, 2016

PREFACE 1st EDITION

The General rules and rules for buildings of part 1-1 of Eurocode 3 constitute the core of the code procedures for the design of steel structures. They contain the basic guidance for structural modeling and analysis of steel frameworks and the rules for the evaluation of the resistance of structural members and components subject to different loading conditions.

According to the objectives of the ECCS Eurocode Design Manuals, it is the objective of this book to provide mix of “light” theoretical background, explanation of the code prescriptions and detailed design examples. Consequently, this book is more than a manual: it provides an all-in-one source for an explanation of the theoretical concepts behind the code and detailed design examples that try to reproduce real design situations instead of the usually simplified examples that are found in most textbooks.

This book evolved from the experience of teaching Steel Structures according to ENV 1993-1-1 since 1993. It further benefited from the participation in Technical Committees TC8 and TC10 of ECCS where the background and the applicability of the various clauses of EN 1993-1-1 was continuously questioned. This book covers exclusively part 1-1 of Eurocode 3 because of the required level of detail. Forthcoming volumes discuss and apply most of the additional parts of Eurocode 3 using a consistent format.

xvii

Chapter 1 introduces general aspects such as the basis of design, material properties and geometric characteristics and tolerances, corresponding to chapters 1 to 4 and chapter 7 of EN 1993-1-1. It highlights the important topics that are required in the design of steel structures. Structural analysis is discussed in chapter 2, including structural modelling, global analysis and classification of cross sections, covering chapter 5 of EN 1993-1-1. The design of steel members subjected to various types of internal force (tension, bending and shear, compression and torsion) and their combinations is described in chapter 3, corresponding to chapter 6 of EN 1993-1-1. Chapter 4 presents the design of steel structures using 3D elastic analysis based on the case study of a

PREFACE

real building. Finally, chapter 5 discusses plastic design, using a pitched-roof industrial building to exemplify all relevant aspects.

Furthermore, the design examples provided in this book are chosen from real design cases. Two complete design examples are presented: i) a braced steel-framed building; and ii) a pitched-roof industrial building. The chosen design approach tries to reproduce, as much as possible, real design practice instead of more academic approaches that often only deal with parts of the design process. This means that the design examples start by quantifying the actions. They then progress in a detailed step-by-step manner to global analysis and individual member verifications. The design tools currently available and adopted in most design offices are based on software for 3D analysis. Consequently, the design example for multi-storey buildings is analysed as a 3D structure, all subsequent checks being consistent with this approach. This is by no means a straightforward implementation, since most global stability verifications were developed and validated for 2D structures.

The authors are indebted to Prof. Reidar Bjorhovde who carried out a detailed technical review of the manuscript and provided many valuable comments and suggestions. Warm thanks to Prof. David Anderson who carried out an additional detailed revision of the book and also made sure that the English language was properly used. Further thanks to Liliana Marques and José Alexandre Henriques, PhD students at the University of Coimbra, for the help with the design examples of chapter 4. Additional thanks to Prof. Tiago Abecasis who spotted innumerable “bugs” in the text. Finally, thanks to Filipe Dias and the staff of cmm and ECCS for all the editorial and typesetting work, making it possible to bring to an end two years of work in this project.

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Chapter 3

DESIGN OF MEMBERS

3.1. INTRODUCTION

3.1.1. General

According to the general framework established in EN 1990 for the safety of structures, the safety of steel members at ultimate limit state is ensured by applying partial safety factors γ_M to the various characteristic values of resistance. The safety factors are defined in accordance with the potential failure modes. For steel members, the following three failure modes are considered (clause 6.1(1)): i) resistance of cross sections, whatever the class; ii) resistance of members to instability assessed by member checks and iii) resistance of cross sections in tension to fracture. Specific partial safety factors γ_{M0} , γ_{M1} and γ_{M2} , deemed to guarantee the reliability targets of EN 1990, correspond to each failure mode, respectively. The following values of the partial safety factors γ_{Mi} are recommended for buildings¹: $\gamma_{M0} = 1.00$; $\gamma_{M1} = 1.00$ and $\gamma_{M2} = 1.25$ and will be used throughout this book. It is noted that for other types of structures, recommended values are given in Parts 2 to 6 of EN 1993. For structures not covered by Parts 2 to 6 of EN 1993, the National Annexes may define the partial factors γ_{Mi} ; it is recommended in this case to take the partial factors γ_{Mi} from EN 1993-2 (CEN, 2006d).

This chapter describes the basic theoretical concepts, as well as the normative design rules (according to EC3-1-1) concerning the verification of the resistance of steel members. In particular, the evaluation of the resistance

¹ These values were adopted by the majority of the National Annexes.

3. DESIGN OF MEMBERS

of cross sections of classes 1 to 3 subjected to the various combinations of internal forces is presented in accordance with clauses 6.2. Additionally, the assessment of the resistance of members subject to instability phenomena is also covered in accordance with clauses 6.3. Finally, as in the previous chapter, several detailed worked examples are presented.

3.1.2. Resistance of cross sections

3.1.2.1. General criteria

The resistance of cross sections depends on their class (clause 6.2.1(3)). According to the definition of the four cross section classes (see sub-chapter 2.4), cross section classes 1 and 2 reach their full plastic resistance, while class 3 cross sections only reach their elastic resistance. Class 4 cross sections are not able to reach their elastic resistance because of local buckling and they are outside the scope of EC3-1-1 and of this book. Nevertheless, using the concept of effective section (CEN, 2006c), they are effectively treated as class 3 cross sections and their resistance is evaluated as an elastic resistance.

The design value of an action effect, at each cross section, should not exceed the corresponding design resistance, and if several action effects act simultaneously, the combined effect should not exceed the resistance for that combination (clause 6.2.1(1)). Shear lag effects and local buckling effects should be included according to the concept of effective section of EC3-1-5 (CEN, 2006c). Shear buckling effects should also be considered according to EC3-1-5 (clause 6.2.1(2)).

An elastic verification according to the elastic resistance may be carried out for all cross sectional classes provided that the effective cross sectional properties are used for the verification of class 4 cross sections (clause 6.2.1(4)). In the most general case and as a conservative approach, where local longitudinal, transverse and shear stresses coexist at the critical point of the cross section, the following yield criterion may be used in the context of an elastic verification (clause 6.2.1(5)).

$$\left(\frac{\sigma_{x,Ed}}{f_y/\gamma_{M0}}\right)^2 + \left(\frac{\sigma_{z,Ed}}{f_y/\gamma_{M0}}\right)^2 - \left(\frac{\sigma_{x,Ed}}{f_y/\gamma_{M0}}\right)\left(\frac{\sigma_{z,Ed}}{f_y/\gamma_{M0}}\right) + 3\left(\frac{\tau_{Ed}}{f_y/\gamma_{M0}}\right)^2 \leq 1, \quad (3.1)$$

where $\sigma_{x,Ed}$ is the design value of the local longitudinal stress, $\sigma_{z,Ed}$ is the design value of the local transverse stress and τ_{Ed} is the design value of the local shear stress, all values at the point of consideration.

For classes 1 or 2, the resistance of cross sections may be evaluated on the basis of their plastic resistance by finding a stress distribution which is in equilibrium with the internal forces and moments without exceeding the yield strength. This stress distribution should be compatible with the associated plastic deformations (clause 6.2.1(6)).

For class 3 cross sections, where all the compression parts of a cross section are class 3, its resistance should be based on an elastic distribution of strains across the cross section. Compressive stresses should be limited to the yield strength at the extreme fibres (clause 6.2.1(9)). These extreme fibres may be assumed at the midplane of the flanges for ULS checks. However, whenever yielding first occurs on the tension side of the cross section, the plastic reserves of the tension zone may be utilized by accounting for partial plastification when determining the resistance of a class 3 cross section (clause 6.2.1(10)).

For both plastic and elastic verifications of safety, interaction formulae on the basis of resistances (N_{Rd} , M_{Rd} , V_{Rd}) are favoured since they may lead to less conservative results. As a conservative approximation for all cross section classes, a linear summation of the utilization ratios for each stress resultant may be used. For class 1, class 2 or class 3 cross sections subjected to a combination of N_{Ed} , $M_{y,Ed}$, $M_{z,Ed}$ this method may be applied by using the following criterion (clause 6.2.1(7)):

$$\frac{N_{Ed}}{N_{Rd}} + \frac{M_{y,Ed}}{M_{y,Rd}} + \frac{M_{z,Ed}}{M_{z,Rd}} \leq 1. \quad (3.2)$$

Class 3 cross sections exhibit a gradual transition from plastic to elastic resistance because of residual stress effects and local yielding. This is not currently recognized by Eurocode 3 whose provisions result in a sudden transition from plastic resistance to elastic resistance. Extensive research was recently carried out to provide a safe smooth transition between classes 2 and 3 (Greiner *et al*, 2011).

3.1.2.2. Section properties

The properties of the gross cross section should be determined using the nominal dimensions. Holes for fasteners need not be deducted, but

3. DESIGN OF MEMBERS

allowance should be made for larger openings. Splice materials should not be included (clause 6.2.2.1(1)).

Because of the existence of holes and other openings, it is necessary to define the net area of a cross section. Generally, it is defined as its gross area less appropriate deductions for all holes and other openings (clause 6.2.2.2(1)). For calculating net section properties, the deduction for a single fastener hole should be the gross cross sectional area of the hole in the plane of its axis. For countersunk holes, appropriate allowance should be made for the countersunk portion (clause 6.2.2.2(2)).

In the case of multiple fastener holes, provided that the fastener holes are not staggered, the total area to be deducted for fastener holes should be the maximum sum of the sectional areas of the holes in any cross section perpendicular to the member axis (clause 6.2.2.2(3)). Where the fastener holes are staggered (Figure 3.1), the net area A_{net} should be the minimum of (clause 6.2.2.2(4)):

$$A - n_p t d_0 \quad (\text{fracture section 1}); \quad (3.3)$$

$$A - n t d_0 + t \sum \left(\frac{s^2}{4p} \right) \quad (\text{fracture section 2}), \quad (3.4)$$

122

where:

- | | |
|-------|---|
| A | is the gross area of the section; |
| n_p | is the number of non-staggered holes in any cross section perpendicular to the member axis; |
| n | is the number of holes extending in any diagonal or zig-zag line progressively across the member or part of the member, see Figure 3.1; |
| t | is the thickness; |
| d_0 | is the hole diameter; |
| s | is the staggered pitch, the spacing of the centres of two consecutive holes in the chain measured parallel to the member axis; |
| p | is the spacing of the centres of the same two holes measured perpendicular to the member axis. |
-

The summation in expression (3.4) represents the number of segments between staggered holes, as it is exemplified in example 3.1.

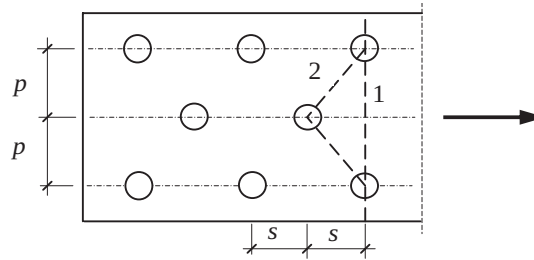


Figure 3.1 – Definition of the net area of a cross section

In the case of angles, or other member with holes in more than one plane, the spacing p should be measured along the mid-plane of the legs, as illustrated in Figure 3.2.

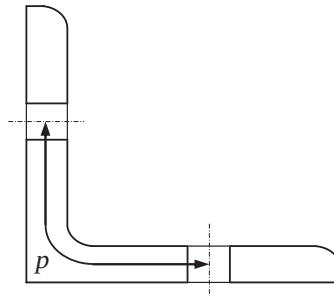


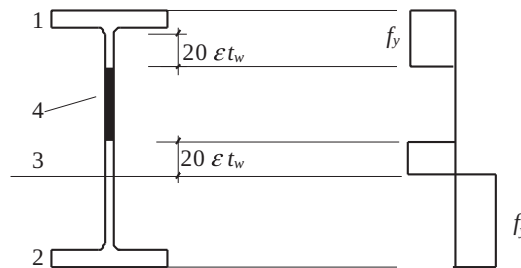
Figure 3.2 – Angle with holes in both legs

Cross sections with a class 3 web and flanges with class 1 or 2 may be classified and designed as class 2, considering a reduced effective area for the web. The effective area is obtained according to Figure 3.3 and the following iterative procedure: by replacing the portion of the web in compression by a part of $20 \varepsilon t_w$ adjacent to the compression flange and another equal part adjacent to the plastic neutral axis of the effective cross section. Iteration results from the definition of the neutral axis as being that of the effective section (Figure 3.3).

The resistance of class 4 cross sections is limited by local instability phenomena that prevent the development of the elastic resistance of the cross section. According to EC3-1-1, local instability phenomena, in class 4 cross

3. DESIGN OF MEMBERS

sections, should be taken into account by replacing the gross section by an effective cross section, obtained from a reduced area of the compression parts. The effective cross section should be based on effective widths, according to EC3-1-5. The effective cross section of circular hollow sections should be obtained from EC3-1-6 (CEN, 2007). Figures 3.4 and 3.5 represent, in a qualitative way, the effective cross section of a U section subject to compression and an I section subject to major axis bending, respectively. In these figures, the portions of the gross area to be deducted are indicated in black. For the I section, it is assumed that only the web is class 4.



1 - compression; 2 - tension; 3 - plastic neutral axis; 4 - neglected part

Figure 3.3 – Effective class 2 web

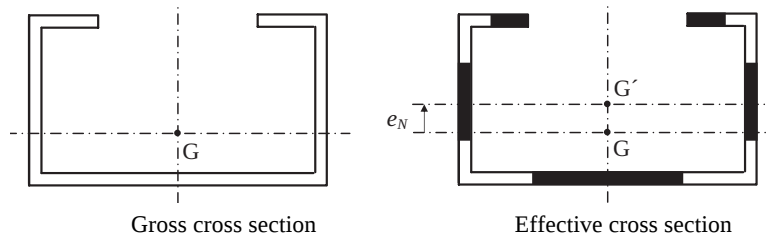


Figure 3.4 – Class 4 cross section submitted to a compressive axial force

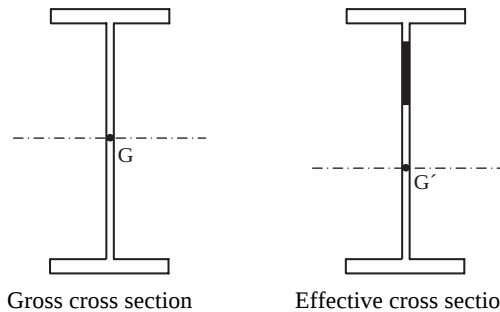


Figure 3.5 – Class 4 cross section submitted to bending moment

An axial compression force in a cross section in class 4 due to the possible shift, e_N , of the centroid of the effective area, A_{eff} , relative to the centre of gravity of the gross cross section, results in an additional bending moment $\Delta M_{Ed} = N_{Ed} e_N$.

The analysis of cross sections with class 4 is not included in the scope of this book. The analysis may be performed according to EC3-1-3, for cold formed sections; according to EC3-1-5, for hot rolled sections and welded sections; and according to EC3-1-6, for circular hollow sections.

3.1.3. Buckling resistance of members

In addition to verification of the cross section resistance, the buckling resistance of members must also be checked, according to clauses 6.3 and 6.4. The buckling phenomenon depends on the presence of compressive stresses and therefore it must be checked for all members subjected to axial compression, bending moment or a combination of both. Shear buckling effects should also be considered according to EC3-1-5.

For a member under pure compression the buckling modes to take into account are: i) flexural buckling; ii) torsional buckling and iii) torsional-flexural buckling. A member under bending moment must be checked against lateral-torsional buckling. A member under a combination of compression force and bending moment must be checked against all the buckling modes mentioned above. The theoretical background, the design rules and several applications relating to the buckling resistance of steel members are presented in the sub-chapters 3.5, 3.6 and 3.7.

3.2. TENSION

3.2.1. Behaviour in tension

Figure 3.6 illustrates various examples of structures with some members that are commonly assumed to be loaded only in tension. Figure 3.7 shows typical cross sections of tension members. Simple or built-up rolled sections are commonly used in trusses, lattice girders and as bracing members. Cables, flats or bars are used in bracing systems. Cables,

3. DESIGN OF MEMBERS

flats or bars are some times used in bridges or long-span roofs; such member types are discussed in detail in EC3-1-11 (CEN, 2006e).

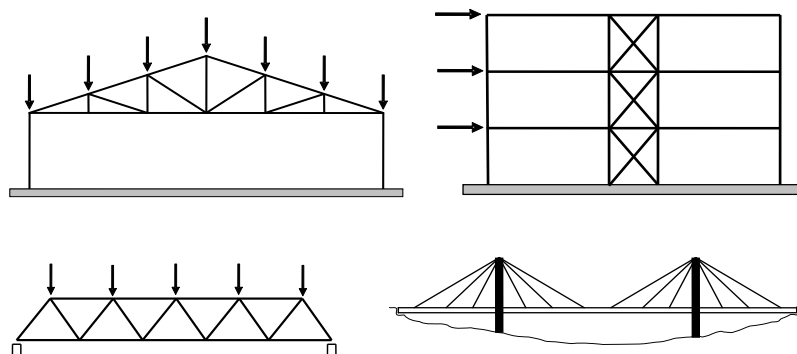


Figure 3.6 – Structures with some members in tension

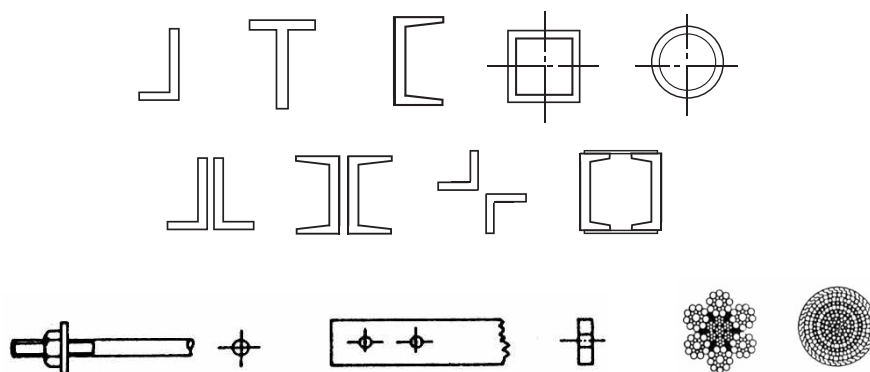


Figure 3.7 – Typical cross sections of members in tension

The behaviour of members in tension is closely related to the stress-strain behaviour of steel subjected to uniaxial tensile forces. Recalling the stress-strain relationship shown in Figure 1.7, the ultimate cross section resistance corresponds to the tensile strength R_m , although the plastic resistance is also often considered as the ultimate tensile resistance of the member, especially when ductility is of concern.

Typically, the governing design situation for members subject to tension corresponds to the location of the joints (either the connection to other parts of the structure or splices within the tension member). In these cross sections, either because of bolting or because of a change of cross sectional shape, the net area of the cross section must be taken into account.

The calculation of the net area in tension was described in section 3.1.2.2. In addition, it is noted that stress concentrations occur in the neighbourhood of holes or discontinuities, as shown in Figure 3.8.

Bolted or welded connections often induce second-order moments because of small eccentricities, as shown in Figure 3.9. These second-order effects should be taken into account. Alternatively, careful detailing should be specified to eliminate these eccentricities, as illustrated in Figure 3.10.

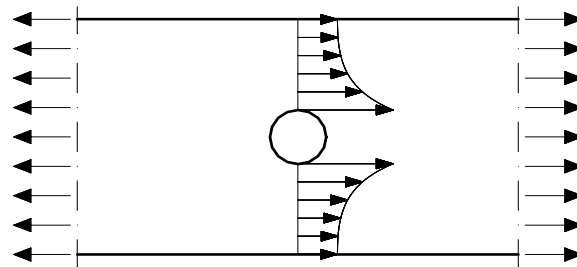


Figure 3.8 – Concentration of tension next to a hole

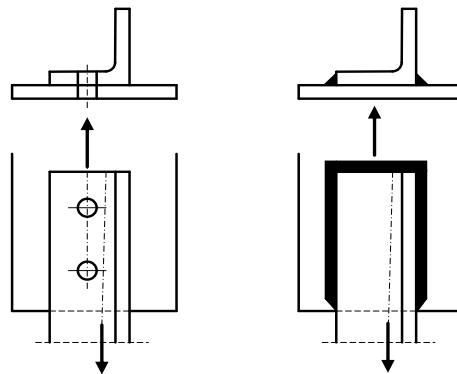


Figure 3.9 – Eccentric connections

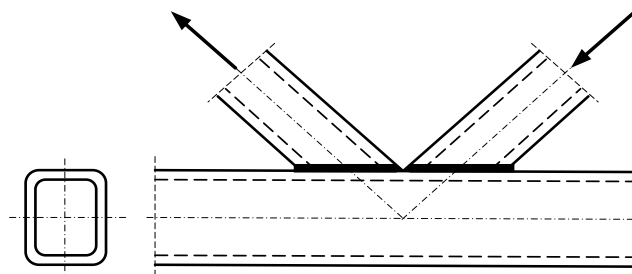


Figure 3.10 – Welded connections between hollow sections

3.2.2. Design for tensile force

A member exclusively subject to a tension force is under a uniaxial stress state. According to clause 6.2.3(1), the design value of the tension force N_{Ed} at each cross section, including cross sections in the vicinity of the connections, should satisfy:

$$\frac{N_{Ed}}{N_{t,Rd}} \leq 1.0, \quad (3.5)$$

where $N_{t,Rd}$ is the design tension resistance. For sections with holes the design tension resistance $N_{t,Rd}$ should be taken as the smallest of:

- design plastic resistance of the gross cross section,

$$N_{pl,Rd} = A f_y / \gamma_{M0}, \quad (3.6)$$

where:

- A is the gross cross section area,
- f_y is the yield strength of steel and
- γ_{M0} is the partial safety factor.

- design ultimate resistance of the net cross section at holes for fasteners,

$$N_{u,Rd} = 0.9 A_{net} f_u / \gamma_{M2} \quad (3.7)$$

where:

- A_{net} is the net cross section area,
- f_u is the ultimate strength of steel and
- γ_{M2} is the partial safety factor.

Whenever dissipative behaviour is required under cyclic loading, such as in the case of capacity design (CEN, 2004a), the design plastic resistance $N_{pl,Rd}$ should be less than the design ultimate resistance of the net section at fasteners holes $N_{u,Rd}$ (clause 6.2.3(3)), that is,

$$N_{u,Rd} > N_{pl,Rd} \Leftrightarrow \frac{A_{net}}{A} > \frac{f_y}{0.9 f_u} \frac{\gamma_{M2}}{\gamma_{M0}}. \quad (3.8)$$

In the case of members with Category C preloaded bolted connections loaded in shear², the design tension resistance $N_{t,Rd}$ at the cross section with holes for fasteners should be taken as $N_{net,Rd}$ (clause 6.2.3(4)):

$$N_{net,Rd} = A_{net} f_y / \gamma_{M0} . \quad (3.9)$$

For angles connected by one leg and other unsymmetrically connected members in tension (such as T sections or channel sections), the eccentricity in joints and the effects of the spacing and edge distances of the bolts should be taken into account in determining the design resistance (clause 3.10.3(1) of EC3-1-8). According to clause 3.10.3(2) of EC3-1-8, a single angle in tension connected by a single row of bolts, see Figure 3.11, may be treated as concentrically loaded over an effective net section for which the design ultimate resistance should be determined as follows:

$$N_{u,Rd} = \frac{2.0(e_2 - 0.5 d_0) t f_u}{\gamma_{M2}}; \quad (1 \text{ bolt}) \quad (3.10)$$

$$N_{u,Rd} = \frac{\beta_2 A_{net} f_u}{\gamma_{M2}}; \quad (2 \text{ bolts}) \quad (3.11)$$

$$N_{u,Rd} = \frac{\beta_3 A_{net} f_u}{\gamma_{M2}}. \quad (3 \text{ bolts or more}) \quad (3.12)$$

129

In these equations,

- t is the thickness of the leg of an angle;
- f_u is the ultimate strength of steel;
- d_0 is the hole diameter;
- e_2 is the distance of the centre of the fastener holes to the adjacent edge of the angle, perpendicular to the direction of load transfer (as illustrated in Figure 3.11);
- γ_{M2} is a partial safety factor, defined according to EC3-1-8.

² Connections slip-resistant at ultimate limit state (clause 3.4.1(1)c of EC3-1-8).

3. DESIGN OF MEMBERS

The net area, A_{net} , is calculated according to sub-section 3.1.2.2 (clause 6.2.2); in angles of unequal legs, connected by the smaller leg, A_{net} should be taken as equal to the net section area of an equivalent equal-leg angle of leg size equal to that of the smaller leg. Parameters β_2 and β_3 are reduction factors which are defined depending on the distance between holes (pitch p_1), according to Table 3.1; for values of $2.5d_0 < p_1 < 5d_0$, these parameters can be determined by linear interpolation.

Table 3.1 – Reduction factors β_2 and β_3

Pitch	p_1	$\leq 2.5 d_0$	$\geq 5.0 d_0$
2 bolts	β_2	0.4	0.7
3 bolts or more	β_3	0.5	0.7

It is reminded that no matter what value is given by equations (3.10) to (3.12), the resistance is limited by equation (3.6).

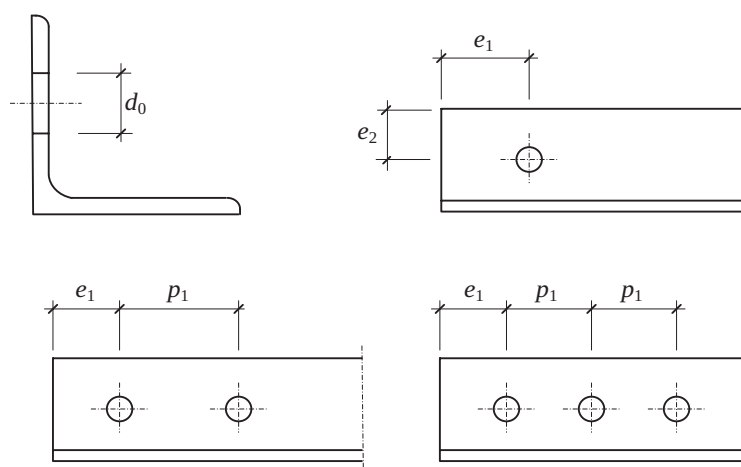


Figure 3.11 – Angles connected by one leg

Members that comprise angles connected by welding only in one leg can be treated as being concentrically loaded. Resistance is determined using equation (3.6), but based on an effective cross section area. The area of the effective cross section, according to clause 4.13 of EC3-1-8, must be evaluated as follows: i) for angles of equal legs or unequal legs that are

connected by the larger leg, the area of the effective section may be considered as equal to the gross area; ii) for angles of unequal legs, connected by the smaller leg, the area of the effective section should be taken as equal to the gross area of an equivalent angle, with legs that are equal to the smaller of the legs.

3.2.3. Worked examples

Example 3.1: Calculate the net area A_{net} of the bolted section of the plate represented in Figure 3.12. Assume a plate with thickness t and the remaining dimensions (in mm), as indicated in Figure 3.12.

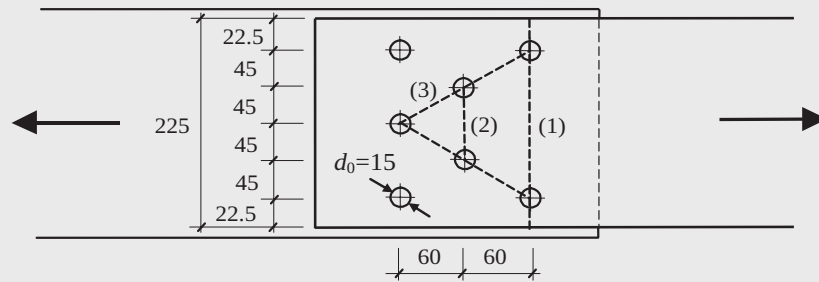


Figure 3.12 – Net area of a plate

The connection is loaded by a concentric axial force, therefore the load may be assumed to be uniformly distributed amongst the fasteners. Due to the position of the fasteners, the net area is evaluated considering fracture sections 1, 2 and 3, as illustrated in Figure 3.12. Fracture section 1 is perpendicular to the direction of the force, whereas fracture sections 2 and 3 include staggered pitches. However, all of these sections correspond to net sections subjected to the totality of the applied axial force. Hence, from expressions (3.3) and (3.4):

$$\text{Fracture section 1} \rightarrow A_{net}^{(1)} = 225 \times t - 2 \times t \times 15 = 195 t .$$

$$\text{Fracture section 2} \rightarrow A_{net}^{(2)} = 225 \times t - 4 \times t \times 15 + 2 \times t \times \frac{60^2}{4 \times 45} = 205 t .$$

$$\text{Fracture section 3} \rightarrow A_{net}^{(3)} = 225 \times t - 5 \times t \times 15 + 4 \times t \times \frac{60^2}{4 \times 45} = 230 t .$$

3. DESIGN OF MEMBERS

The net area of the plate is given by the minimum value, $A_{net} = 195 t$.

Example 3.2: Consider the chord AB of the steel truss, indicated in Figure 3.13, assuming it is submitted to a design tensile axial force of $N_{Ed} = 220$ kN. The cross section consists of two angles of equal legs, in steel grade S 235. Design chord AB assuming two distinct possibilities for the connections:

- a) welded connections;
- b) bolted connections.

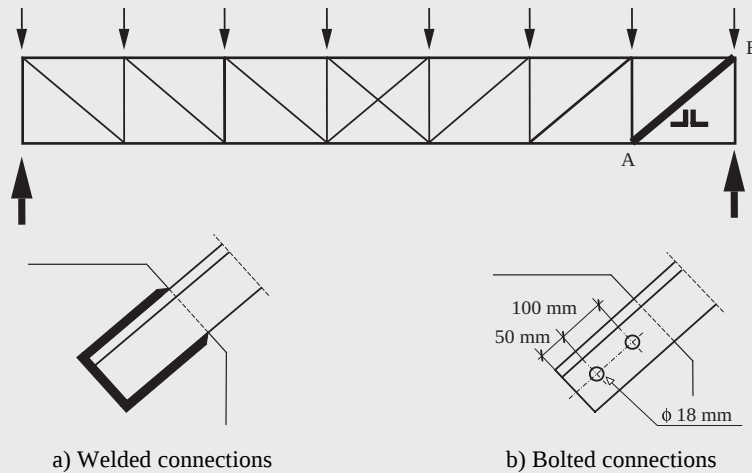


Figure 3.13 – Steel truss

a) Welded connections

The chord is made up by two angles of equal legs, but the connection is made only in one leg of the angle. Thus, according to clause 4.13 of EC3-1-8, the effective area can be considered equal to the gross area. Therefore, the following conditions must be satisfied:

$$N_{Ed} \leq N_{t,Rd} = \frac{A f_y}{\gamma_{M0}},$$

where $\gamma_{M0} = 1.0$, $f_y = 235$ MPa and A is the gross area of the section.

Considering the design axial force, $N_{Ed} = 220$ kN, then:

$$220 \text{ kN} \leq \frac{A \times 235 \times 10^3}{1.0} \Rightarrow A \geq 9.36 \times 10^{-4} \text{ m}^2 = 9.36 \text{ cm}^2.$$

From a table of commercial profiles, a solution with two angles $50 \times 50 \times 5$ mm, with a total area of $2 \times 4.8 = 9.6 \text{ cm}^2$, satisfies the above safety requirement.

b) Bolted connections

In this case, the chord, made up by angles of equal legs, is connected by 2 bolts only in one leg. According to clause 3.10.3 of EC3-1-8, the following design conditions must be ensured:

$$N_{Ed} \leq N_{t,Rd}, \text{ with } N_{t,Rd} = \min \left[N_{pl,Rd} = \frac{A f_y}{\gamma_{M0}}; N_{u,Rd} = \frac{\beta_2 A_{net} f_u}{\gamma_{M2}} \right],$$

where, $\gamma_{M0} = 1.0$, $\gamma_{M2} = 1.25$, $f_y = 235$ MPa, $f_u = 360$ MPa, A is the gross area of the cross section, A_{net} is the net area of the bolted section, and β_2 is a factor obtained from Table 3.1 (or Table 3.8 of EC3-1-8). A first check based on the plastic design of the gross cross section leads to:

$$220 \text{ kN} \leq \frac{A \times 235 \times 10^3}{1.0} \Rightarrow A \geq 9.36 \times 10^{-4} \text{ m}^2 = 9.36 \text{ cm}^2.$$

Hence, the section obtained in the previous design, two angles $50 \times 50 \times 5$ mm ($A = 9.6 \text{ cm}^2$), also satisfies this safety requirement.

The second condition (equation (3.11), reproduced above) requires the evaluation of the net area A_{net} , (illustrated in Figure 3.14) and the factor β_2 , both evaluated according to clause 3.10.3 of EC3-1-8.

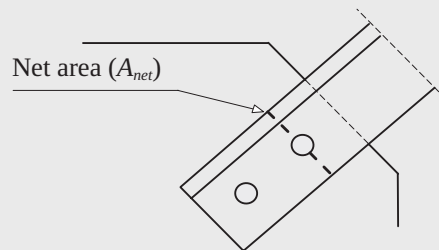


Figure 3.14 – A_{net} in the bolted connection

3. DESIGN OF MEMBERS

For $d_0 = 18$ mm, $2.5d_0 = 45$ mm and $5d_0 = 90$ mm.

As $p_1 = 100$ mm > 90 mm, then $\beta_2 = 0.70$.

The net area of the bolted section made up of two angles is given by:

$$A_{net} = A - 2 t d_0 = 9.6 - 2 \times 0.5 \times 1.8 = 7.8 \text{ cm}^2.$$

Thus, the design ultimate resistance is given by:

$$N_{u,Rd} = \frac{0.7 \times 7.8 \times 10^{-4} \times 360 \times 10^3}{1.25} = 157.2 \text{ kN}.$$

However, $N_{Ed} = 220$ kN $> N_{u,Rd} = 157.2$ kN; therefore, the chosen cross section is not appropriate. By adopting a cross section with enhanced resistance, for example, two angles $60 \times 60 \times 6$ mm ($A = 13.82 \text{ cm}^2$ and $A_{net} = 11.66 \text{ cm}^2$), then:

$$N_{pl,Rd} = 13.82 \times 10^{-4} \times 235 \times 10^3 / 1.0 = 324.8 \text{ kN} > N_{Ed} = 220 \text{ kN} ;$$

$$N_{u,Rd} = \frac{0.7 \times 11.66 \times 10^{-4} \times 360 \times 10^3}{1.25} = 235.1 \text{ kN} > N_{Ed} = 220 \text{ kN} .$$

As $N_{pl,Rd} = 324.8$ kN $> N_{u,Rd} = 235.1$ kN, failure is non-ductile; however, since this is not a design condition, the section defined by two angles $60 \times 60 \times 6$ mm can be accepted.

Example 3.3: Figure 3.15 represents a lattice girder in steel grade S 275, supporting a reinforced concrete floor. The loading, applied on the floor and transmitted to the truss as concentrated loads applied in the nodes, is defined by the following distributed loads:

Permanent action on the floor = 5.75 kN/m^2 ($\gamma_G = 1.35$);

Variable action on the floor = 4.00 kN/m^2 ($\gamma_Q = 1.50$).

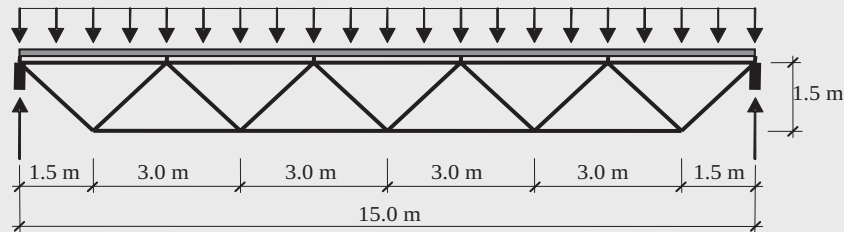


Figure 3.15 – Lattice girder

The distance between lattice girders is 3.00 m; the nodes of the truss are braced in the perpendicular direction to the plane of the structure; the loading already includes the selfweight of the steel truss. Design the tension members of the truss, assuming the following alternatives:

- Square hollow sections (SHS), and welded connections for the members of the structure.
- HEA profiles in the upper and lower chords (horizontal members) and 2 UPN channel profiles for the diagonal members. The diagonal members are bolted to gusset plates, which are welded to the HEA profiles in the upper and lower chords.

For design at the ultimate limit state, the following combination of actions is considered (according to EN 1990):

$$p_{Ed} = 1.35 \times 5.75 + 1.5 \times 4.00 = 13.76 \text{ kN/m}^2,$$

where p_{Ed} is the design load, uniformly distributed on the floor.

The concentrated loads, represented in Figure 3.16, were calculated based on the influence areas of each node, and considering a distance of 3.00 m between the lattice girders. In the same figure, the internal forces in the members, obtained by node equilibrium or any other appropriate method, are represented.

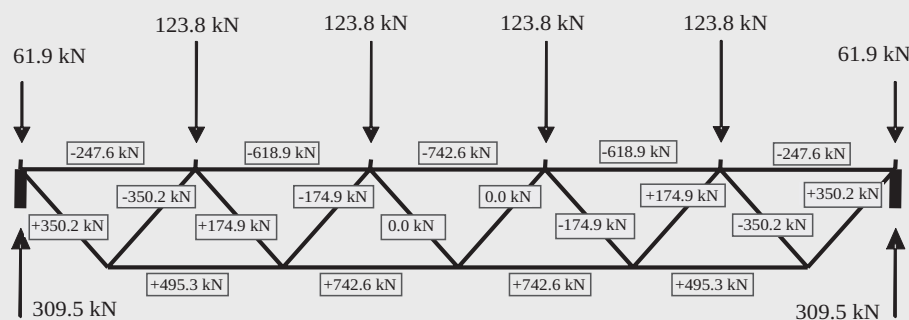


Figure 3.16 – Actions and internal forces on the structure

- According to the distribution of internal forces illustrated in Figure 3.16, two distinct cross sections are adopted: one for the bottom tension chord,

3. DESIGN OF MEMBERS

designed for an axial force of $N_{Ed} = 742.6$ kN, and another for the diagonal tension members, designed for an axial force of $N_{Ed} = 350.2$ kN.

The members comprise square hollow sections with welded connections. Assuming that the axis of the members, in each node, converges in a point, there is no reduction of the resistance due to eccentricities in the connections (note, however, that reductions of the resistance due to local stresses in the connection may be necessary, as given in EC3-1-8). Hence, the design of the tension members is given by:

$$N_{Ed} \leq N_{t,Rd} = \frac{A f_y}{\gamma_{M0}},$$

where $\gamma_{M0} = 1.0$, $f_y = 275$ MPa and A is the gross area of the cross section.

Considering the design axial forces leads to:

- for the bottom chord,

$$742.6 \leq \frac{A \times 275 \times 10^3}{1.0} \Rightarrow A \geq \frac{742.6 \times 1.0}{275 \times 10^3} = 27.0 \times 10^{-4} \text{ m}^2 = 27.0 \text{ cm}^2,$$

- for the diagonal members,

$$350.2 \leq \frac{A \times 275 \times 10^3}{1.0} \Rightarrow A \geq \frac{350.2 \times 1.0}{275 \times 10^3} = 12.7 \times 10^{-4} \text{ m}^2 = 12.7 \text{ cm}^2.$$

From a table of commercial profiles for square hollow cross sections (SHS), the following solution is adopted: a SHS 120×120×6.3 mm ($A = 28.5 \text{ cm}^2$) cross section for the bottom chord and a SHS 80×80×5 mm ($A = 14.9 \text{ cm}^2$) cross section for the diagonal members.

b) Taking into account the type of connection, the bottom tension chord is designed for an axial force $N_{Ed} = 742.6$ kN, considering the gross cross section; while the diagonals in tension are designed for an axial force $N_{Ed} = 350.2$ kN, but considering a possible reduction of the resistance in the bolted cross section, due to the holes. The design plastic resistance of the bottom chord, considering the gross cross section, is given by:

$$742.6 \leq \frac{A \times 275 \times 10^3}{1.0} \Rightarrow A \geq \frac{742.6 \times 1.0}{275 \times 10^3} = 27.0 \times 10^{-4} \text{ m}^2 = 27.0 \text{ cm}^2.$$

Similarly, for the diagonal members:

$$350.2 \leq \frac{A \times 275 \times 10^3}{1.0} \Rightarrow A \geq \frac{350.2 \times 1.0}{275 \times 10^3} = 12.7 \times 10^{-4} \text{ m}^2 = 12.7 \text{ cm}^2.$$

Hence, a HEA 140 ($A = 31.42 \text{ cm}^2$) cross section is adopted for the bottom chord, and a 2 UPN 80 ($A = 22.00 \text{ cm}^2$) cross section is initially proposed for the diagonal members. The overdesign of the section of the diagonal members is due to the fact that the dimensions of the bolts required to resist the applied axial force (considering the connection illustrated in Figure 3.17) are not compatible with the dimensions of smaller UPN cross sections.

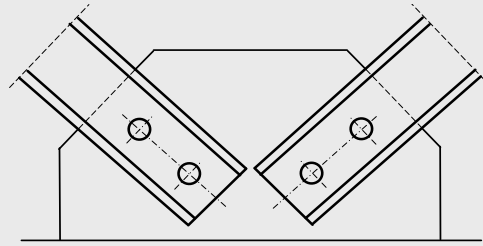


Figure 3.17 – Connection of the diagonal bars of the truss

In the case of the diagonal members, considering the connection with M20 bolts ($d_0 = 22 \text{ mm}$) illustrated in Figure 3.17, the design ultimate resistance of the bolted cross section should also be checked. The net area of the bolted cross section made up of two UPN is given by:

$$A_{net} = A - 2td_0 = 22.0 - 2 \times 0.6 \times 2.2 = 19.36 \text{ cm}^2.$$

The design ultimate resistance is given by:

$$N_{u,Rd} = \frac{0.9 \times 19.36 \times 10^{-4} \times 430 \times 10^3}{1.25} = 506.5 \text{ kN} > N_{Ed} = 350.2 \text{ kN}.$$

As the design ultimate resistance in the bolted section exceeds the applied axial force, the section defined by two UPN 80 satisfies the safety criterion. However, as $N_{pl,Rd} = 605 \text{ kN} > N_{u,Rd} = 506.5 \text{ kN}$, the failure of the diagonal members would be non-ductile. This situation often occurs in tension members with bolted connections.

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