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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ECCS Eurocode Design Manuals
# TABLE OF CONTENTS

**FOREWORD**  
xiii  
**PREFACE**  
xv

Chapter 1  
**INTRODUCTION**  
1  
1.1. General Observations  
1  
1.2. Codes of Practice and Normalization  
3  
\hspace{1em} 1.2.1. Introduction  
3  
\hspace{1em} 1.2.2. Eurocode 3  
6  
\hspace{1em} 1.2.3. Other standards  
7  
1.3. Basis of Design  
8  
\hspace{1em} 1.3.1. Basic concepts  
8  
\hspace{1em} 1.3.2. Reliability management  
9  
\hspace{1em} 1.3.3. Basic variables  
13  
\hspace{2em} 1.3.3.1. Introduction  
13  
\hspace{2em} 1.3.3.2. Actions and environmental influences  
13  
\hspace{2em} 1.3.3.3. Material properties  
14  
\hspace{2em} 1.3.3.4. Geometrical data  
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. Introduction 77  
2.3.2.2. Elastic critical load 80  
2.3.2.3. 2nd order analysis 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. General criteria  
120

##### 3.1.2.2. Section properties  
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. Elastic and plastic bending moment resistance  
139

##### 3.3.2.2. Uniaxial bending  
141

##### 3.3.2.3. Bi-axial bending  
142

##### 3.3.2.4. Net area in bending  
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. Introduction  
157

##### 3.4.1.2. Uniform torsion  
159

##### 3.4.1.3. Non-uniform torsion  
161
<table>
<thead>
<tr>
<th>Section</th>
<th>Description</th>
<th>Page</th>
</tr>
</thead>
<tbody>
<tr>
<td>3.4.1.4.</td>
<td>Cross section resistance in torsion</td>
<td>166</td>
</tr>
<tr>
<td>3.4.2.</td>
<td>Design for torsion</td>
<td>173</td>
</tr>
<tr>
<td>3.4.3.</td>
<td>Worked examples</td>
<td>176</td>
</tr>
<tr>
<td>3.5.</td>
<td>Compression</td>
<td>186</td>
</tr>
<tr>
<td>3.5.1.</td>
<td>Theoretical background</td>
<td>186</td>
</tr>
<tr>
<td>3.5.1.1.</td>
<td>Introduction</td>
<td>186</td>
</tr>
<tr>
<td>3.5.1.2.</td>
<td>Elastic critical load</td>
<td>186</td>
</tr>
<tr>
<td>3.5.1.3.</td>
<td>Effect of imperfections and plasticity</td>
<td>192</td>
</tr>
<tr>
<td>3.5.2.</td>
<td>Design for compression</td>
<td>198</td>
</tr>
<tr>
<td>3.5.3.</td>
<td>Worked examples</td>
<td>204</td>
</tr>
<tr>
<td>3.6.</td>
<td>Laterally Unrestrained Beams</td>
<td>212</td>
</tr>
<tr>
<td>3.6.1.</td>
<td>Introduction</td>
<td>212</td>
</tr>
<tr>
<td>3.6.2.</td>
<td>Lateral-torsional buckling</td>
<td>212</td>
</tr>
<tr>
<td>3.6.2.1.</td>
<td>Introduction</td>
<td>212</td>
</tr>
<tr>
<td>3.6.2.2.</td>
<td>Elastic critical moment</td>
<td>213</td>
</tr>
<tr>
<td>3.6.2.3.</td>
<td>Effect of imperfections and plasticity</td>
<td>223</td>
</tr>
<tr>
<td>3.6.3.</td>
<td>Lateral-torsional buckling resistance</td>
<td>225</td>
</tr>
<tr>
<td>3.6.4.</td>
<td>Worked examples</td>
<td>229</td>
</tr>
<tr>
<td>3.7.</td>
<td>Beam-Columns</td>
<td>242</td>
</tr>
<tr>
<td>3.7.1.</td>
<td>Introduction</td>
<td>242</td>
</tr>
<tr>
<td>3.7.2.</td>
<td>Classification of cross sections under bending and axial force</td>
<td>243</td>
</tr>
<tr>
<td>3.7.3.</td>
<td>Cross section resistance</td>
<td>247</td>
</tr>
<tr>
<td>3.7.3.1.</td>
<td>Theoretical background</td>
<td>247</td>
</tr>
<tr>
<td>3.7.3.2.</td>
<td>Design resistance</td>
<td>249</td>
</tr>
<tr>
<td>3.7.4.</td>
<td>Buckling resistance</td>
<td>253</td>
</tr>
<tr>
<td>3.7.4.1.</td>
<td>Theoretical background</td>
<td>253</td>
</tr>
<tr>
<td>3.7.4.2.</td>
<td>Design resistance</td>
<td>256</td>
</tr>
<tr>
<td>3.7.5.</td>
<td>Worked examples</td>
<td>265</td>
</tr>
<tr>
<td>Section</td>
<td>Title</td>
<td>Page</td>
</tr>
<tr>
<td>---------</td>
<td>----------------------------------------------------------------------</td>
<td>------</td>
</tr>
<tr>
<td>4.1</td>
<td>Introduction</td>
<td>293</td>
</tr>
<tr>
<td>4.2</td>
<td>Simplified Methods of Analysis</td>
<td></td>
</tr>
<tr>
<td>4.2.1</td>
<td>Introduction</td>
<td>295</td>
</tr>
<tr>
<td>4.2.2</td>
<td>Amplified sway-moment method</td>
<td>297</td>
</tr>
<tr>
<td>4.2.3</td>
<td>Sway-mode buckling length method</td>
<td>299</td>
</tr>
<tr>
<td>4.2.4</td>
<td>Worked example</td>
<td>300</td>
</tr>
<tr>
<td>4.3</td>
<td>Member Stability of Non-prismatic Members and Components</td>
<td>310</td>
</tr>
<tr>
<td>4.3.1</td>
<td>Introduction</td>
<td>310</td>
</tr>
<tr>
<td>4.3.2</td>
<td>Non-prismatic members</td>
<td>310</td>
</tr>
<tr>
<td>4.3.3</td>
<td>Members with intermediate restraints</td>
<td>316</td>
</tr>
<tr>
<td>4.3.4</td>
<td>General method</td>
<td>322</td>
</tr>
<tr>
<td>4.3.5</td>
<td>Worked example</td>
<td>325</td>
</tr>
<tr>
<td>4.4</td>
<td>Design Example 1: Elastic Design of Braced Steel-Framed Building</td>
<td>340</td>
</tr>
<tr>
<td>4.4.1</td>
<td>Introduction</td>
<td>340</td>
</tr>
<tr>
<td>4.4.2</td>
<td>Description of the structure</td>
<td>342</td>
</tr>
<tr>
<td>4.4.3</td>
<td>General safety criteria, actions and combinations of actions</td>
<td>344</td>
</tr>
<tr>
<td>4.4.3.1</td>
<td>General safety criteria</td>
<td>344</td>
</tr>
<tr>
<td>4.4.3.2</td>
<td>Permanent actions</td>
<td>345</td>
</tr>
<tr>
<td>4.4.3.3</td>
<td>Imposed loads</td>
<td>345</td>
</tr>
<tr>
<td>4.4.3.4</td>
<td>Wind actions</td>
<td>346</td>
</tr>
<tr>
<td>4.4.3.5</td>
<td>Summary of basic actions</td>
<td>353</td>
</tr>
<tr>
<td>4.4.3.6</td>
<td>Frame imperfections</td>
<td>353</td>
</tr>
<tr>
<td>4.4.3.7</td>
<td>Load combinations</td>
<td>356</td>
</tr>
<tr>
<td>4.4.3.8</td>
<td>Load arrangement</td>
<td>358</td>
</tr>
<tr>
<td>4.4.4</td>
<td>Structural analysis</td>
<td>359</td>
</tr>
</tbody>
</table>
Table of Contents

4.4.4.1. Structural model 359
4.4.4.2. Linear elastic analysis 360
4.4.4.3. Susceptibility to 2nd order effects: elastic critical loads 361
4.4.4.4. 2nd order elastic analysis 362

4.4.5. Design checks 363
4.4.5.1. General considerations 363
4.4.5.2. Cross section resistance 365
4.4.5.3. Buckling resistance of beams 366
4.4.5.4. Buckling resistance of columns and beam-columns 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. 2nd order effects 393
5.2.4.1. Introduction 393
5.2.4.2. Elastic critical load 394
5.2.4.3. 2nd order computational analysis 397
5.2.4.4. Simplified methods for analysis 397
5.2.5. Worked example 400

5.3. Member Stability and Buckling Resistance 410
<table>
<thead>
<tr>
<th>Section Number</th>
<th>Section Title</th>
</tr>
</thead>
<tbody>
<tr>
<td>5.3.1</td>
<td>Introduction</td>
</tr>
<tr>
<td>5.3.2</td>
<td>General criteria for the verification of the stability of members with plastic hinges</td>
</tr>
<tr>
<td>5.3.3</td>
<td>Bracings</td>
</tr>
<tr>
<td>5.3.4</td>
<td>Verification of the stability of members with plastic hinges</td>
</tr>
<tr>
<td>5.4.1</td>
<td>Introduction</td>
</tr>
<tr>
<td>5.4.2</td>
<td>General description</td>
</tr>
<tr>
<td>5.4.3.1</td>
<td>General criteria</td>
</tr>
<tr>
<td>5.4.3.2</td>
<td>Permanent actions</td>
</tr>
<tr>
<td>5.4.3.3</td>
<td>Snow loads</td>
</tr>
<tr>
<td>5.4.3.4</td>
<td>Wind loads</td>
</tr>
<tr>
<td>5.4.4.1</td>
<td>Introduction</td>
</tr>
<tr>
<td>5.4.5.1</td>
<td>Structural analysis</td>
</tr>
<tr>
<td>5.4.5.3</td>
<td>Load combinations</td>
</tr>
<tr>
<td>5.4.5.4</td>
<td>Imperfections</td>
</tr>
<tr>
<td>5.4.6</td>
<td>Pre-design</td>
</tr>
<tr>
<td>5.4.7</td>
<td>Design of Industrial Building</td>
</tr>
<tr>
<td>5.4.8</td>
<td>Worked examples</td>
</tr>
<tr>
<td>5.4.9</td>
<td>Modification factors for moment gradients in members laterally restrained along the tension flange</td>
</tr>
<tr>
<td>5.4.10</td>
<td>Design Example 2: Plastic Design</td>
</tr>
</tbody>
</table>

TABLE OF CONTENTS
## Table of Contents

5.4.5.1. Linear elastic analysis 446  
5.4.5.2. 2\textsuperscript{nd} order effects 448  
5.4.5.3. Elastic-plastic analysis 449  
5.4.6. Code checks 451  
5.4.6.1. General considerations 451  
5.4.6.2. Cross section resistance 451  
5.4.6.3. Buckling resistance of the rafters 451  
5.4.6.4. Buckling resistance of the columns 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  
\hspace{1cm} A.2.1 Concentrated torsional moment 467  
\hspace{1cm} A.2.2 Distributed torsional moment 474  

Annex B

ELASTIC CRITICAL MOMENT 483  
B.1. Abacus to Calculate the Coefficients $C_1$, $C_2$ and $C_3$ 483  
\hspace{1cm} B.1.1 Elastic critical moment in beams submitted to end moments simultaneously with transverse loads 483  
\hspace{1cm} B.1.2 Elastic critical moment of unbraced cantilevers 487  
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.

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

Luís Simões da Silva
Rui Simões
Helena Gervásio
Coimbra, 2016
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.

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
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 innumerous “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.

Luís Simões da Silva  
Rui Simões  
Helena Gervásio  
Coimbra, 2010
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 $\gamma_M$ 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 $\gamma_{M0}$, $\gamma_{M1}$ and $\gamma_{M2}$, deemed to guarantee the reliability targets of EN 1990, correspond to each failure mode, respectively. The following values of the partial safety factors $\gamma_M$ are recommended for buildings\textsuperscript{1}: $\gamma_{M0} = 1.00$; $\gamma_{M1} = 1.00$ 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 $\gamma_M$; it is recommended in this case to take the partial factors $\gamma_M$ 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

\textsuperscript{1} These values were adopted by the majority of the National Annexes.
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)).

\[
\left( \frac{\sigma_{x,Ed}}{f_y/y_{MO}} \right)^2 + \left( \frac{\sigma_{f,Ed}}{f_y/y_{MO}} \right)^2 - \left( \frac{\sigma_{x,Ed}}{f_y/y_{MO}} \right) \left( \frac{\sigma_{f,Ed}}{f_y/y_{MO}} \right) + 3 \left( \frac{\tau_{Ed}}{f_y/y_{MO}} \right)^2 \leq 1, \quad (3.1)
\]
where $\sigma_{x,Ed}$ is the design value of the local longitudinal stress, $\sigma_{t,Ed}$ is the design value of the local transverse stress and $\tau_{Ed}$ 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 ($N_{Ed}$, $M_{y,Ed}$, $V_{Ed}$) 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 $N_{Ed}$, $M_{y,Ed}$, $M_{x,Ed}$ this method may be applied by using the following criterion (clause 6.2.1(7)):

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

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
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)):

$$A - n_p t d_0$$  \hspace{2cm} (fracture section 1);  \hspace{2cm} (3.3)

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

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.
3.1. Introduction

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](image1)

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](image2)

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
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.

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, $A_{\text{eff}}$, 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).

![Structures with some members in tension](image)

**Figure 3.6 – Structures with some members in tension**

![Typical cross sections of members in tension](image)

**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.
3. Design of Members

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:

$$\frac{N_{Ed}}{N_{t,Rd}} \leq 1.0,$$  \hspace{1cm} (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,

$$N_{pl,Rd} = A f_y / \gamma_{M0},$$  \hspace{1cm} (3.6)

where:

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

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

$$N_{u,Rd} = 0.9 A_{net} f_u / \gamma_{M2}$$  \hspace{1cm} (3.7)

where:

- $A_{net}$ is the net cross section area,
- $f_u$ is the ultimate strength of steel and
- $\gamma_{M2}$ 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,

$$N_{u,Rd} > N_{pl,Rd} \iff A_{net} > \frac{f_y \gamma_{M2}}{0.9 f_u \gamma_{M0}}.$$  \hspace{1cm} (3.8)
In the case of members with Category C preloaded bolted connections loaded in shear\(^2\), 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)):

\[
N_{net,Rd} = A_{net} f_y / \gamma_{M2}.
\]  

(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:

\[
N_{u,Rd} = \frac{2.0(e_2 - 0.5 d_0) t f_u}{\gamma_{M2}}.
\]  

(1 bolt)  

(3.10)

\[
N_{u,Rd} = \frac{\beta_3 A_{net} f_u}{\gamma_{M2}}.
\]  

(2 bolts)  

(3.11)

\[
N_{u,Rd} = \frac{\beta_3 A_{net} f_u}{\gamma_{M2}}.
\]  

(3 bolts or more)  

(3.12)

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);
- \(\gamma_{M2}\) 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, \( A_{\text{net}} \), is calculated according to sub-section 3.1.2.2 (clause 6.2.2); in angles of unequal legs, connected by the smaller leg, \( A_{\text{net}} \) 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 \( \beta_2 \) and \( \beta_3 \) 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>
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<tr>
<th>Pitch</th>
<th>( p_1 )</th>
<th>( \leq 2.5d_0 )</th>
<th>( \geq 5.0d_0 )</th>
</tr>
</thead>
<tbody>
<tr>
<td>2 bolts</td>
<td>( \beta_2 )</td>
<td>0.4</td>
<td>0.7</td>
</tr>
<tr>
<td>3 bolts or more</td>
<td>( \beta_3 )</td>
<td>0.5</td>
<td>0.7</td>
</tr>
</tbody>
</table>

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).

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 $A_{\text{net}}$ 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](image)

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):

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

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

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

The net area of the plate is given by the minimum value, \( A_{\text{net}} = 195 \, \text{t} \).

**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 \( N_{Ed} = 220 \, \text{kN} \). The cross section consists of two angles of equal legs, in steel grade S235. 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:

\[
N_{Ed} \leq N_{t,\text{Ed}} = \frac{A f_y}{\gamma_{M0}},
\]

where \( \gamma_{M0} = 1.0 \), \( f_y = 235 \, \text{MPa} \) 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 50x50x5mm, 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:

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

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 $\beta_2$ 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 50x50x5 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 $\beta_2$, both evaluated according to clause 3.10.3 of EC3-1-8.

![Figure 3.14 — $A_{net}$ in the bolted connection](image-url)
For $d_0 = 18\text{ mm}$, $2.5d_0 = 45\text{ mm}$ and $5d_0 = 90\text{ mm}$.

As $p_i = 100\text{ mm} > 90\text{ mm}$, then $\beta_2 = 0.70$.

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

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

Thus, the design ultimate resistance is given by:

$$N_u,R_d = 0.7 \times 7.8 \times 10^{-4} \times 360 \times 10^3 \times 1.25 = 157.2\text{ kN}.$$ 

However, $N_{Ed} = 220\text{ kN} > N_u,R_d = 157.2\text{ kN}$; therefore, the chosen cross section is not appropriate. By adopting a cross section with enhanced resistance, for example, two angles $60\times60\times6\text{ mm}$ ($A = 13.82\text{ cm}^2$ and $A_{\text{net}} = 11.66\text{ cm}^2$), then:

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

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

As $N_{pl,Rd} = 324.8\text{ kN} > N_u,R_d = 235.1\text{ kN}$, failure is non-ductile; however, since this is not a design condition, the section defined by two angles $60\times60\times6\text{ mm}$ 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 = $5.75\text{ kN/m}^2$ ($\gamma_G = 1.35$);  
Variable action on the floor = $4.00\text{ kN/m}^2$ ($\gamma_Q = 1.50$).
3.2. TENSION

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 self-weight of the steel truss. Design the tension members of the truss, assuming the following alternatives:

a) Square hollow sections (SHS), and welded connections for the members of the structure.

b) 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**

a) 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 $N_{\text{ed}} = 742.6$ kN, and another for the diagonal tension members, designed for an axial force of $N_{\text{ed}} = 350.2$ 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:

$$N_{\text{ed}} \leq N_{\text{t,rd}} = \frac{A f_y}{\gamma_M},$$

where $\gamma_M = 1.0$, $f_y = 275$ 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 ($A = 28.5$ cm$^2$) cross section for the bottom chord and a SHS 80×80×5 mm ($A = 14.9$ 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 $N_{\text{ed}} = 742.6$ kN, considering the gross cross section; while the diagonals in tension are designed for an axial force $N_{\text{ed}} = 350.2$ 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.

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:

\[ A_{\text{net}} = A - 2td_o = 22.0 - 2 \times 0.6 \times 2.2 = 19.36 \text{ cm}^2. \]

The design ultimate resistance is given by:

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

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 \( N_{\text{pl,Rd}} = 605 \text{ kN} > N_{u,\text{Rd}} = 506.5 \text{ kN} \), the failure of the diagonal members would be non-ductile. This situation often occurs in tension members with bolted connections.
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