Structural Steel Design to Eurocode 3 and AISC Specifications

Claudio Bernuzzi • Benedetto Cordova

WILEY Blackwell

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Contents

Preface

Over the last century, design of steel structures has developed from very simple approaches based on a few elementary properties of steel and essential mathematics to very sophisticated treatments demanding a thorough knowledge of structural and material behaviour. Nowadays, steel design utilizes refined concepts of mechanics of material and of theory of structures combined with probabilistic-based approaches that can be found in design specifications.

This book intends to be a guide to understanding the basic concepts of theory of steel structures as well as to provide practical guidelines for the design of steel structures in accordance with both European (EN 1993) and United States (ANSI/AISC 360-10) specifications. It is primarily intended for use by practicing engineers and engineering students, but it is also relevant to all different parties associated with steel design, fabrication and construction.

The book synthesizes the Authors' experience in teaching Structural Steel Design at the Technical University of Milan-Italy (Claudio Bernuzzi) and in design of steel structures for power plants (Benedetto Cordova), combining their expertise in comparing and contrasting both European and American approaches to the design of steel structures.

The book consists of 16 chapters, each structured independently of the other, in order to facilitate consultation by students and professionals alike. Chapter 1 introduces general aspects such as material properties and products, imperfection and tolerances, also focusing the attention on testing methods and approaches. The fundamentals of steel design are summarized in Chapter 2, where the principles of structural safety are discussed in brief to introduce the different reliability levels of the design. Framed systems and methods of analysis, including simplified methods, are discussed in Chapter 3. Cross-sectional classification is presented in Chapter 4, in which special attention has been paid to components under compression and bending. Design of single members is discussed in depth in Chapter 5 for tension members, in Chapter 6 for compression members, in Chapter 7 for members subjected to bending and shear, in Chapter 8 for members under torsion, and in Chapter 9 for members subjected to bending and compression. Chapter 10 deals with design accounting for the combination of compression, flexure, shear and torsion.

Chapter 11 addresses requirements for the web resistance design and Chapter 12 deals with the design approaches for frame analysis. Chapters 13 and 14 deal with bolted and welded connections, respectively, while the most common type of joints are described in Chapter 15, including a summary of the approach to their design. Finally, built-up members are discussed in Chapter 16. Several design examples provided in this book are directly chosen from real design situations. All examples are presented providing all the input data necessary to develop the design. The different calculations associated with European and United States specifications are provided in two separate text columns in order to allow a direct comparison of the associated procedures.

Last, but not least, the acknowledge of the Authors. A great debt of love and gratitude to our families: their patience was essential to the successful completion of the book.

We would like to express our deepest thanks to Dr. Giammaria Gabbianelli (University of Pavia-I) and Dr. Marco Simoncelli (Politecnico di Milano-I) for the continuous help in preparing

figures and tables and checking text. We are also thankful to prof. Gian Andrea Rassati (University of Cincinnati-U.S.A.) for the great and precious help in preparation of chapters 1 and 13.

Finally, it should be said that, although every care has been taken to avoid errors, it would be sanguine to hope that none had escape detection. Authors will be grateful for any suggestion that readers may make concerning needed corrections.

Claudio Bernuzzi and Benedetto Cordova

CHAPTER 1

The Steel Material

1.1 General Points about the Steel Material

The term *steel* refers to a family of iron–carbon alloys characterized by well-defined percentage ratios of main individual components. Specifically, iron–carbon alloys are identified by the carbon (C) content, as follows:

- *wrought iron*, if the carbon content (i.e. the percentage content in terms of weight) is higher than 1.7% (some literature references have reported a value of 2%);
- *steel*, when the carbon content is lower than the previously mentioned limit. Furthermore, steel can be classified into extra-mild ($C < 0.15\%$), mild ($C = 0.15 \div 0.25\%$), semi-hard ($C = 0.25 \div 0.50\%$), hard ($C = 0.50 \div 0.75\%$) and extra-hard ($C > 0.75\%$) materials.

Structural steel, also called *constructional steel* or sometimes *carpentry steel*, is characterized by a carbon content of between 0.1 and 0.25%. The presence of carbon increases the strength of the material, but at the same time reduces its ductility and weldability; for this reason structural steel is usually characterized by a low carbon content. Besides *iron* and *carbon*, structural steel usually contains small quantities of other elements. Some of them are already present in the iron ore and cannot be entirely eliminated during the production process, and others are purposely added to the alloy in order to obtain certain desired physical or mechanical properties.

Among the elements that cannot be completely eliminated during the production process, it is worth mentioning both *sulfur* (S) and *phosphorous* (P), which are undesirable because they decrease the material ductility and its weldability (their overall content should be limited to approximately 0.06%). Other undesirable elements that can reduce ductility are *nitrogen* (N), *oxygen* (O) and *hydrogen* (H). The first two also affect the strain-ageing properties of the material, increasing its fragility in regions in which permanent deformations have taken place.

The most important alloying elements that may be added to the materials are *manganese* (Mn) and *silica* (Si), which contribute significantly to the improvement of the weldability characteristics of the material, at the same time increasing its strength. In some instances, *chromium* (Cr) and *nickel* (Ni) can also be added to the alloy; the former increases the material strength and, if is present in sufficient quantity, improves the corrosion resistance (it is used for stainless steel), whereas the latter increases the strength while reduces the deformability of the material.

Structural Steel Design to Eurocode 3 and AISC Specifications, First Edition. Claudio Bernuzzi and Benedetto Cordova.

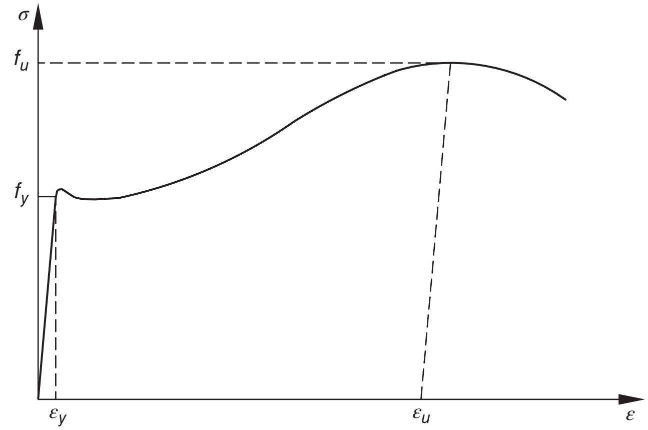


Figure 1.1 Typical constitutive law for structural steel.

Steel is characterized by a symmetric constitutive stress-strain law (σ–ε). Usually, this law is determined experimentally by means of a tensile test performed on coupons (samples) machined from plate material obtained from the sections of interest (Section 1.7). Figure 1.1 shows a typical stress-strain response to a uniaxial tensile force for a structural steel coupon. In particular, it is possible to distinguish the following regions:

- an initial branch that is mostly linear (*elastic phase*), in which the material shows a linear elastic behaviour approximately up to the yielding stress (f_y). The strain corresponding to f_y is usually indicated with ε_y (yielding strain). The slope of this initial branch corresponds to the modulus of elasticity of the material (also known as longitudinal modulus of elasticity or Young's modulus), usually indicated by E, with a value between 190 000 and 210 000 N/mm^2 (from 27 560 to 30 460 ksi, approximately);
- a *plastic phase*, which is characterized by a small or even zero slope in the σ–ε reference system;
- the ensuing branch is the *hardening phase*, in which the slope is considerably smaller when compared to the elastic phase, but still sufficient enough to cause an increase in stress when strain increases, up to the ultimate strength f_u. The hardening modulus has values between 4000 and 6000 N/mm^2 (from 580 to 870 ksi, approximately).

Usually, the uniaxial constitutive law for steel is schematized as a multi-linear relationship, as shown in Figure 1.2a, and for design purposes an elastic-perfectly plastic approximation is generally used; that is the hardening branch is considered to be horizontal, limiting the maximum strength to the yielding strength.

The yielding strength is the most influential parameter for design. Its value is obtained by means of a laboratory uniaxial tensile test, usually performed on coupons cut from the members of interest in suitable locations (see Section 1.7).

In many design situations though, the state of stress is biaxial. In this case, reference is made to the well-known Huber-Hencky–Von Mises criterion (Figure 1.2b) to relate the mono-axial yielding stress (f_y) to the state of plane stress with the following expression:

$$\sigma_1{}^2 - \sigma_1\sigma_2 + \sigma_2{}^2 + 3\sigma_{12}{}^2 = f_y{}^2 \tag{1.1}$$

where σ_1, σ_2 are the normal stresses and σ_{12} is the shear stress.

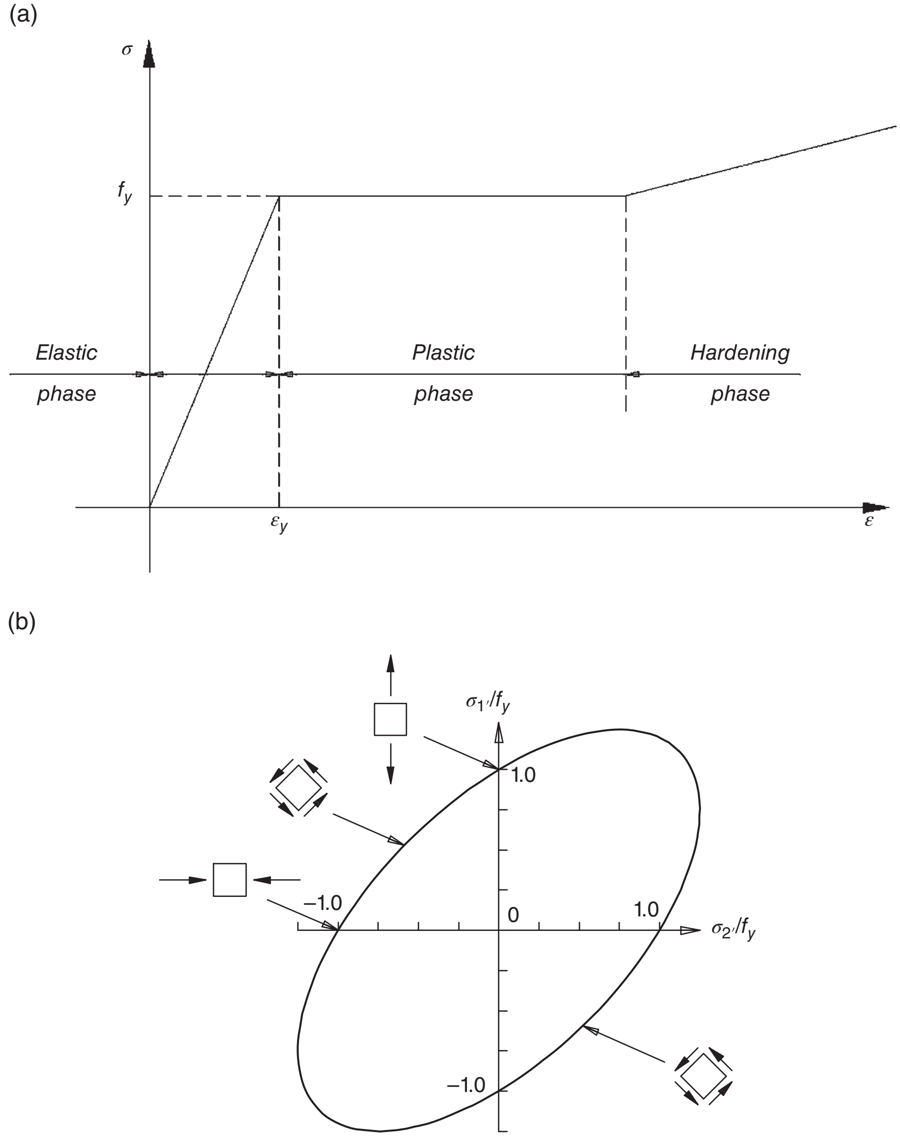


Figure 1.2 Structural steel: (a) schematization of the uniaxial constitutive law and (b) yield surface for biaxial stress states.

In the case of pure shear, the previous equation is reduced to:

$$\sigma_{12} = \tau_{12} = \frac{f_y}{\sqrt{3}} = \tau_y \tag{1.2}$$

With reference to the principal stress directions 1′ and 2′, the yield surface is represented by an ellipse and Eq. (1.1) becomes:

$$(\sigma_{1'})^2 + (\sigma_{2'})^2 - (\sigma_{1'})\cdot(\sigma_{2'}) = f_y{}^2 \tag{1.3}$$

1.1.1 Materials in Accordance with European Provisions

The European provisions prescribe the following values for material properties concerning structural steel design:

Density:	$\rho = 7850\ \text{kg/m}^3$ (= 490 lb/ft^3)
Poisson's coefficient:	$\nu = 0.3$
Longitudinal (Young's) modulus of elasticity:	$E = 210\,000\ \text{N/mm}^2$ (= 30 460 ksi)
Shear modulus:	$G = \dfrac{E}{2(1+\nu)}$
Coefficient of linear thermal expansion:	$\alpha = 12 \times 10^{-6}$ per °C (=6.7×10^{-6} per °F)

The mechanical properties of the steel grades most used for construction are summarized in Tables 1.1a and 1.1b, for hot-rolled and hollow profiles, respectively, in terms of yield strength (f_y) and ultimate strength (f_u). Similarly, Table 1.2 refers to steel used for mechanical fasteners. With respect to the European nomenclature system for steel used in high strength fasteners, the generic tag ($j.k$) can be immediately associated to the mechanical characteristics of the material expressed in International System of units (I.S.), considering that:

- $j \cdot k \cdot 10$ represents the yielding strength expressed in N/mm^2;
- $j \cdot 100$ represents the failure strength expressed in N/mm^2.

Table 1.1a Mechanical characteristics of steels used for hot-rolled profiles.

	Nominal thickness *t*			
	$t \le 40$ mm		**40 mm < $t \le 80$ mm**	
EN norm and steel grade	**f_y (N/mm^2)**	**f_u (N/mm^2)**	**f_y (N/mm^2)**	**f_u (N/mm^2)**
EN 10025-2				
S 235	235	360	215	360
S 275	275	430	255	410
S 355	355	510	335	470
S 450	440	550	410	550
EN 10025-3				
S 275 N/NL	275	390	255	370
S 355 N/NL	355	490	335	470
S 420 N/NL	420	520	390	520
S 460 N/NL	460	540	430	540
EN 10025-3				
S 275 M/ML	275	370	255	360
S 355 M/ML	355	470	335	450
S 420 M/ML	420	520	390	500
S 460 M/ML	460	540	430	530
EN 10025-5				
S 235 W	235	360	215	340
S 355 W	355	510	335	490
EN 10025-6				
S 460 Q/QL/QL1	460	570	440	550

Table 1.1b Mechanical characteristics of steels used for hollow profiles.

	Nominal thickness t			
	$t \leq 40$ mm		$40\text{ mm} < t \leq 65$ mm	
EN norm and steel grade	f_y (N/mm^2)	f_u (N/mm^2)	f_y (N/mm^2)	f_u (N/mm^2)
EN 10210-1				
S 235 H	235	360	215	340
S 275 H	275	430	255	410
S 355 H	355	510	335	490
S 275 NH/NLH	275	390	255	370
S 355 NH/NLH	355	490	335	470
S 420 NH/NLH	420	540	390	520
S 460 NH/NLH	460	560	430	550
EN 10219-1				
S 235 H	235	360		
S 275 H	275	430		
S 355 H	355	510		
S 275 NH/NLH	275	370		
S 355 NH/NLH	355	470		
S 460 NH/NLH	460	550		
S 275 MH/MLH	275	360		
S 355 MH/MLH	355	470		
S420 MH/MLH	420	500		
S 460 NH/NLH	460	530		

Table 1.2 Nominal yielding strength values (f_{yb}) and nominal failure strength (f_{ub}) for bolts.

Bolt class	**4.6**	**4.8**	**5.6**	**5.8**	**6.8**	**8.8**	**10.9**
f_{yb} (N/mm^2)	240	320	300	400	480	640	900
f_{ub} (N/mm^2)	400	400	500	500	600	800	1000

The details concerning the designation of steels are covered in EN 10027 Part 1 (*Designation systems for steels – Steel names*) and Part 2 (*Numerical system*), which distinguish the following groups:

- *group 1*, in which the designation is based on the usage and on the mechanical or physical characteristics of the material;
- *group 2*, in which the designation is based on the chemical content: the first symbol may be a letter (e.g. C for non-alloy carbon steels or X for alloy steel, including stainless steel) or a number.

With reference to the group 1 designations, the first symbol is always a letter. For example:

- *B* for steels to be used in reinforced concrete;
- *D* for steel sheets for cold forming;
- *E* for mechanical construction steels;
- *H* for high strength steels;
- *S* for structural steels;
- *Y* for steels to be used in prestressing applications.

Focusing attention on the structural steels (starting with an *S*), there are then three digits *XXX* that provide the value of the minimum yielding strength. The following term is related to the technical conditions of delivery, defined in EN 10025 ('Hot rolled products of structural steel') that proposes the following five abbreviations, each associated to a different production process:

- the *AR* (*As Rolled*) term identifies rolled and otherwise unfinished steels;
- the *N* (*Normalized*) term identifies steels obtained through normalized rolling, that is a rolling process in which the final rolling pass is performed within a well-controlled temperature range, developing a material with mechanical characteristics similar to those obtained through a normalization heat treatment process (see Section 1.2);
- the *M* (*Mechanical*) term identifies steels obtained through a thermo-mechanical rolling process, that is a process in which the final rolling pass is performed within a well-controlled temperature range resulting in final material characteristics that cannot be obtained through heat treating alone;
- the *Q* (*Quenched and tempered*) term identifies high yield strength steels that are quenched and tempered after rolling;
- the *W* (*Weathering*) term identifies weathering steels that are characterized by a considerably improved resistance to atmospheric corrosion.

The YY code identifies various classes concerning material toughness as discussed in the following. Non-alloyed steels for structural use (EN 10025-2) are identified with a code after the yielding strength (XXX), for example:

- YY: alphanumeric code concerning toughness: S235 and S275 steels are provided in groups JR, J0 and J2. S355 steels are provided in groups JR, J0, J2 and K2. S450 steels are provided in group J0 only. The first part of the code is a letter, J or K, indicating a minimum value of toughness provided (27 and 40 J, respectively). The next symbol identifies the temperature at which such toughness must be guaranteed. Specifically, R indicates ambient temperature, 0 indicates a temperature not higher than 0°C and 2 indicates a temperature not higher than −20°C;
- C: an additional symbol indicating special uses for the steel;
- N, AR or M: indicates the production process.

Weldable fine grain structural steels that are normalized or subject to normalized rolling (EN 10025-3); that is, steels characterized by a granular structure with an equivalent ferriting grain size index greater than 6, determined in accordance with EN ISO 643 ('Micrographic determination of the apparent grain size'), are defined by the following codes:

- N: for the production process;
- YY: for the toughness class. The L letter identifies toughness temperatures not lower than −50°C; in the absence of the letter L, the reference temperature must be taken as −20°C.

Fine grain steels obtained through thermo-mechanical rolling processes (EN 10025-4) are identified by the following code:

- M: for the production process;
- YY: for the toughness class. The letter L, as discussed previously, identifies toughness temperatures no lower than −50°C; in the absence of the letter L, the reference temperature must be taken to be −20°C.

Weathering steels for structural use (EN 10025-5) are identified by the following code:

- the YY code indicates the toughness class: these steels are provided in classes J0, J2 and K2, indicating different toughness requirements at different temperatures.
- the W code indicates the weathering properties of the steel;
- P indicates an increased content of phosphorous;
- N or AR indicates the production process.

Quenched and tempered high-yield strength plate materials for structural use (EN 10025-6) are identified by the following codes:

- Q code indicates the production process;
- YY: identifies the toughness class. The letter L indicates a specified minimum toughness temperature of −40°C, while code L1 refers to temperatures not lower than −60°C. In the absence of these codes, the minimum toughness values refer to temperatures no lower than −20°C.

In Europe, it is mandatory to use steels bearing the CE marks, in accordance with the requirements reported in the Construction Products Regulation (CPR) No. 305/2011 of the European Community. The usage of different steels is allowed as long as the degree of safety (not lower than the one provided by the current specifications) can be guaranteed, accompanied by adequate theoretical and experimental documentation.

1.1.2 Materials in Accordance with United States Provisions

The properties of structural steel materials are standardized by ASTM International (formerly known as the *American Society for Testing and Materials*). Numerous standards are available for structural applications, generally dedicated to the most common product families. In the following, some details are reported.

1.1.2.1 General Standards

ASTM A6 (*Standard Specification for General Requirements for Rolled Structural Steel Bars, Plates, Shapes and Sheet Piling*) is the standard that covers the general requirements for rolled structural steel bars, plates, shapes and sheet piling.

1.1.2.2 Hot-Rolled Structural Steel Shapes

Table 1.3 summarizes key data for the most commonly used hot-rolled structural shapes.

- *W-Shapes*
  ASTM A992 is the most commonly used steel grade for all hot-rolled W-Shape members. This material has a minimum yield stress of 50 ksi (356 MPa) and a minimum tensile strength of 65 ksi (463 MPa). Higher values of the yield and tensile strength can be guarantee by ASTM A572 Grades 60 or 65 (Grades 42 and 50 are also available) or ASTM A913 Grades 60, 65 or 70 (Grace 50 is also available). If W-Shapes with atmospheric corrosion resistance characteristics are required, reference can be made to ASTM A588 or ASTM A242 selecting 42, 46 or 50 steel Grades. Finally, W-Shapes according to ASTM A36 are also available.
- *M-Shapes and S-Shapes*
  These shapes have been produced up to now in ASTM A36 steel grade. From some steel producers they are now available in ASTM A572 Grade 50. M-Shapes with atmospheric corrosion resistance characteristics can be obtained by using ASTM A588 or ASTM A242 Grade 50.

Table 1.3 ASTM specifications for various structural shapes (from Table 2-3 of the *AISC Manual*).

Steel type	ASTM designation		F_y minimum yield stress (ksi)	F_u tensile stress (ksi)	W	M	S	HP	C	MC	L	HSS rectangular	HSS round	Pipe
Carbon	A36		36	58–80	▒	■	■	▒	■	■	■			
	A53 Gr. B		35	60										■
	A500	Gr. B	42	58									■	
			46	58								■		
		Gr. C	46	62									▒	
			50	62								▒		
	A501		36	58								▒	▒	
	A529	Gr. 50	50	65–100	▒	▒	▒	▒	▒	▒	▒			
		Gr. 55	55	70–100	▒	▒	▒	▒	▒	▒	▒			
High strength low alloy	A572	Gr. 42	42	60	▒	▒	▒	▒	▒	▒	▒			
		Gr. 50	50	65	▒	▒	▒	■	▒	▒	▒			
		Gr. 55	55	70	▒	▒	▒	▒	▒	▒	▒			
		Gr. 60	60	75	▒	▒	▒	▒	▒	▒	▒			
		Gr. 65	65	80	▒	▒	▒	▒	▒	▒	▒			
	A618	Gr. I and II	50	70								▒	▒	
		Gr. III	50	65								▒	▒	
	A913	50	50	60	▒	▒	▒	▒	▒	▒	▒			
		60	60	75	▒	▒	▒	▒	▒	▒	▒			
		65	65	80	▒	▒	▒	▒	▒	▒	▒			
		70	70	90	▒	▒	▒	▒	▒	▒	▒			
	A992		50–65	65	■	▒	▒	▒	▒	▒	▒			
Corrosion resistant high strength low-alloy	A242		42	63	▒									
			46	67	▒			▒			▒			
			50	70	▒	▒	▒	▒	▒	▒	▒			
	A588		50	70	▒	▒	▒	▒	▒	▒	▒			
	A847		50	70								▒	▒	

■ = Preferred material specification.
▒ = Other applicable material specification.
(blank) = Material specification does not apply.

- *Channels*
  See what is stated about M- and S-Shapes.
- *HP-Shapes*
  ASTM A572 Grade 50 is the most commonly used steel grade for these cross-section shapes. If atmospheric corrosion resistance characteristics are required for HP-Shapes, ASTM A588 or ASTM A242 Grade 46 or 50 can be used. Other materials are available, such as ASTM A36, ASTM A529 Grades 50 or 55, ASTM A572 Grades 42, 55, 60 and 65, ASTM A913 Grades 50, 60, 65, 70 and ASTM A992.
- *Angles*
  ASTM A36 is the most commonly used steel grade for these cross-sections shapes. Atmospheric corrosion resistance characteristics of the angles can be guaranteed by using ASTM A588 or ASTM A242 Grades 46 or 50. Other available materials: ASTM A36, ASTM A529 Grades 50 or 55, ASTM A572 Grades 42, 50, 55 and 60, ASTM A913 Grades 50, 60, 65 and 70 and ASTM A992.
- *Structural Tees*
  Structural tees are produced cutting W-, M- and S-Shapes, to make WT-, MT- and ST-Shapes. Therefore, the same specifications for W-, M- and S-Shapes maintain their validity.

Table 1.4 Applicable ASTM specifications for plates and bars (from Table 2-4 of the *AISC Manual*).

Steel type	ASTM designation		F_y minimum yield stress (ksi)	F_u tensile stress (ksi)	Plates and bars: To 0.75 inclusive	Over 0.75–1.25	Over 1.25–1.5	Over 1.5–2 incl.	Over 2–2.5 incl.	Over 2.5–4 incl.	Over 4–5 incl.	Over 5–6 incl.	Over 6–8 incl.	Over 8
Carbon	A36		32	58–80	░	░	░	░	░	░	░	░	░	■
			36	58–80	■	■	■	■	■	■	■	■	■	░
	A529	Gr. 50	50	70–100	▒	▒	▒	▒	▒	░	░	░	░	░
		Gr. 55	55	70–100	▒	▒	▒	░	░	░	░	░	░	░
High strength low alloy	A572	Gr. 42	42	60	▒	▒	▒	▒	▒	▒	▒	▒	░	░
		Gr. 50	50	65	▒	▒	▒	▒	▒	▒	░	░	░	░
		Gr. 55	55	70	▒	▒	▒	▒	░	░	░	░	░	░
		Gr. 60	60	75	▒	▒	░	░	░	░	░	░	░	░
		Gr. 65	65	80	▒	▒	░	░	░	░	░	░	░	░
Corrosion resistant high strength low alloy	A242		42	63	░	░	░	▒	▒	▒	░	░	░	░
			46	67	░	▒	▒	░	░	░	░	░	░	░
			50	70	▒	░	░	░	░	░	░	░	░	░
	A588		42	63	░	░	░	░	░	░	░	▒	▒	░
			46	67	░	░	░	░	░	░	▒	░	░	░
			50	70	▒	▒	▒	▒	▒	▒	░	░	░	░
Quenched and tempered alloy	A514		90	100–130	░	░	░	░	░	▒	▒	▒	░	░
			100	110–130	▒	▒	▒	▒	▒	▒	▒	▒	░	░
Quenched and tempered low alloy	A852		70	90–110	▒	▒	▒	▒	▒	▒	░	░	░	░

■ = Preferred material specification.
▒ = Other applicable material specification.
░ = Material specification does not apply.

- *Square, Rectangular and Round HSS*
  ASTM A500 Grade B ($F_y = 46$ ksi and $F_u = 58$ ksi) is the most commonly used steel grade for these shapes. ASTM A550 Grade C ($F_y = 50$ ksi and $F_u = 62$ ksi) is also used. Rectangular HSS with atmospheric corrosion resistance characteristics can be obtained by using ASTM A847. Other available materials are ASTM A501 and ASTM A618.
- *Steel Pipes*
  ASTM A53 Grade B ($F_y = 35$ ksi and $F_u = 60$ ksi) is the only steel grade available for these shapes.

1.1.2.3 Plate Products

As to plate products, reference can be made to Table 1.4.

- *Structural plates*
  ASTM A36 $F_y = 36$ ksi (256 MPa) for plate thickness equal to or less then 8 in. (203 mm), $F_y = 32$ ksi (228 MPa) for higher thickness and $F_u = 58$ ksi (413 MPa) is the most commonly used steel grade for structural plates. For other materials, reference can be made to Table 1.4.
- *Structural bars*
  Data related to structural plates are valid also for bars with the exception that ASTM A514 and A852 are not admitted.

1.1.2.4 Sheets

ASTM A606 and ASTM A1011 are the two main standards for metal sheets. The former deals with weathering steel, the latter standardizes steels with improved formability that are typically used for the production of cold-formed profiles.

1.1.2.5 High-Strength Fasteners

ASTM A325 and A490 are the standards dealing with high-strength bolts used in structural steel connections. The nominal failure strength of A325 bolts is 120 ksi (854 MPa), without an upper limit, while the nominal failure strength of A490 bolts is 150 ksi (1034 MPa), with an upper limit of 172 ksi (1224 MPa) per ASTM, limited to 170 ksi (1210 MPa) by the structural steel provisions. ASTM F1852 and F2280 are standards for tension-control bolts, characterized by a splined end that shears off when the desired pretension is reached. Loosely, A325 (and F1852) bolts correspond to 8.8 bolts in European standards and A490 (and F2280) bolts correspond to 10.9 bolts.

ASTM F436 standardizes hardened steel washers for fastening applications. ASTM F959 is the standard for direct tension indicator washers, which are a special category of hardened washers with raised dimples that flatten upon reaching the minimum pretension force in the fastener.

ASTM A563 standardizes carbon and alloy steel nuts.

ASTM A307 is the standard for steel anchor rods; it is also used for large-diameter fasteners (above 1½-in.). ASTM F1554 is the preferred standard for anchor rods.

ASTM 354 standardizes quenched and tempered alloy steel bolts.

ASTM A502 is the standard of reference for structural rivets.

1.2 Production Processes

Steel can be obtained by converting wrought iron or directly by means of fusion of metal scrap and iron ore. Ingots are obtained from these processes, which then can be subject to hot- or cold-mechanical processes, eventually becoming final products (plates, bars, profiles, sheets, rods, bolts, etc.). These products, examined in detail in Section 1.5, can be obtained in various ways that can be practically summarized into the following techniques:

- *forming process by compression or tension* (e.g. forging, rolling, extrusion);
- *forming process by flexure and shear*.

Among these processes, the most important is the rolling process in both its hot- and cold-variations, by which most products used in structural applications (referred to as rolled products) are obtained. In the *hot-rolling* process, steel ingots are brought to a temperature sufficient to soften the material (approximately 1200°C or 2192°F), they first travel through a series of juxtaposed counter-rotating rollers (*primary rolling* – Figure 1.3) and are roughed into square or rectangular cross-section bars.

These semi-worked products are produced in different shapes that can be then further rolled to obtain plates, large- or medium-sized profiles or small-sized profiles, bars and rounds. This additional process is called *secondary rolling*, resulting in the final products.

For example, in order to obtain the typical I-shaped profiles, the semi-worked products, at a temperature slightly above 1200°C (or 2192°F), are sent to the rolling train and its initially rectangular cross-section is worked until the desired shape is obtained. Figure 1.4 shows some of the intermediate cross-sections during the rolling process, until the final I-shape product is obtained.

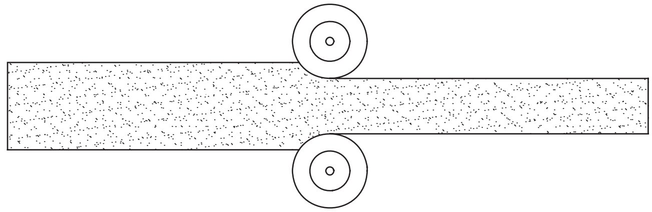


Figure 1.3 Rolling process.

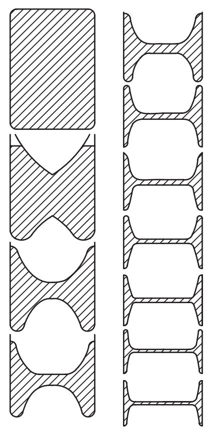


Figure 1.4 Intermediate steps of the rolling process for an I-shape profile.

The rolling process improves the mechanical characteristics of the final product, thanks to the compressive forces applied by the rollers and the simultaneous thinning of the cross-section that favours the elimination of gases and air pockets that might be initially present. At the same time, the considerable deformations imposed by the rolling process contribute to refine the grain structure of the material, with remarkable advantages regarding homogeneity and strength. In such processes, in addition to the amount of deformations, also the rate of deformations is a very important factor in determining the final characteristics of the product.

Cold rolling is performed at the ambient temperature and it is frequently used for non-ferrous materials to obtain higher strengths through hardening at the price of an often non-negligible loss of ductility. When cold-rolling requires excessive strains, the metal can start showing cracks before the desired shape is attained, in which case additional cycles of heat treatments and cold forming are needed (Section 1.3).

The forming processes by *bending and shear* consist of bending thin sheets until the desired cross-section shape is obtained. Typical products obtained by these processes are cold-formed profiles, for which the thickness must be limited to a few millimetres in order to attain the desired deformations. Figure 1.5 shows the intermediate steps to obtain hollow circular cold-formed profiles by means of continuous formation processes.

It can be seen that the coil is pulled and gradually shaped until the desired final product is obtained. Figure 1.6 instead shows the main intermediate steps of the punch-and-die process to obtain some typical profiles currently used in structural applications. With this second working technique, thicker sheets can be shaped into profiles with thicknesses up to 12–15 mm (0.472–0.591 in.), while the limit value of the coil thickness for continuous formation processes is approximately 5 mm (0.197 in.). As an example, Figure 1.7 shows some intermediate steps of the

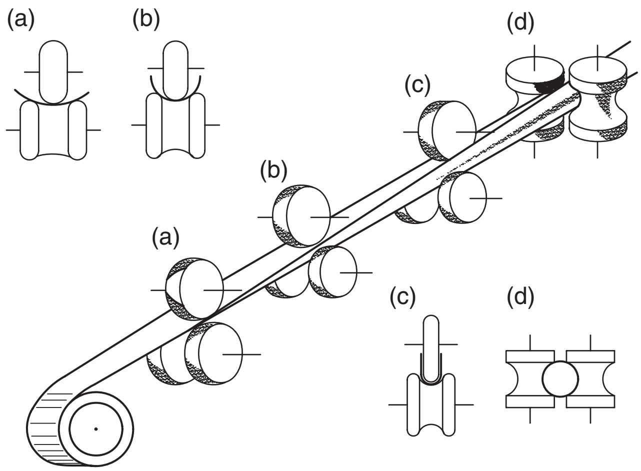


Figure 1.5 Continuous formation of circular hollow cold-formed profiles.

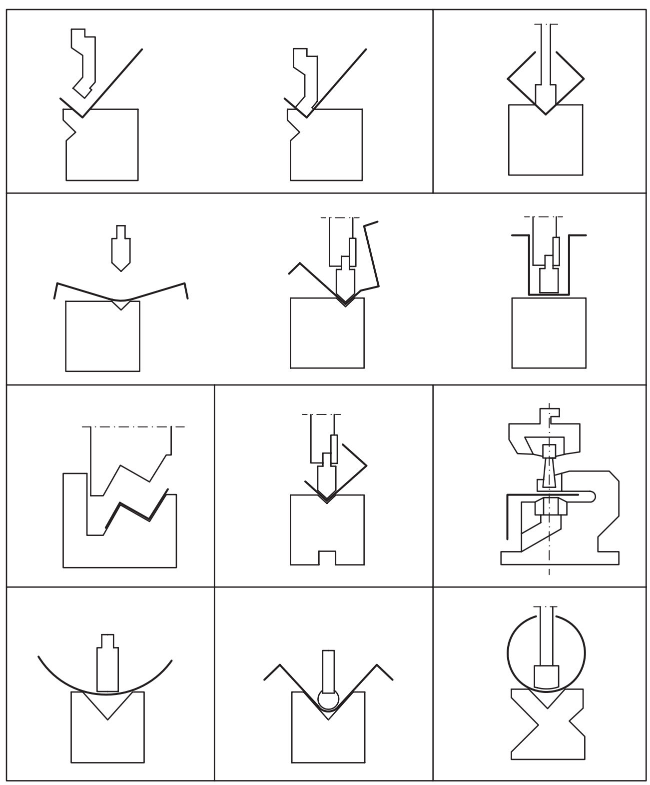


Figure 1.6 Punch-and-die process for cold-formed profiles.

cold-formation process of a stiffened channel profile, with regular perforations, typically used for steel storage pallet racks and shelving structures.

Another important category of steel products obtained with punch-and-die processes is represented by metal decking, currently used for slabs, roofs and cladding.

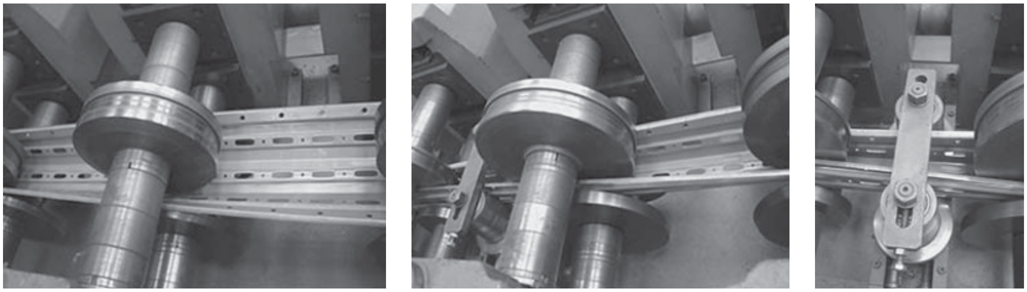


Figure 1.7 Cold-formation images of a stiffened channel profile.

1.3 Thermal Treatments

Steel products, just like other metal products, can be subject to special *thermal treatments* in order to modify their molecular structure, thus changing their mechanical properties. The basic molecular structures are *cementite*, *austenite* and *ferrite*. Transition from one structure to another depends on temperature and carbon content. The main thermal treatments commonly used, which are briefly described in the following, are *annealing*, *normalization*, *tempering*, *quenching*, *pack-hardening* and *quenching and tempering*:

- *annealing* is the thermal cycle that begins with the heating to a temperature close to or slightly above the critical temperature (corresponding to the temperature at which the ferrite-austenite transition is complete); afterwards the temperature is maintained for a predetermined amount of time and then the material is slowly cooled to ambient temperature. Generally, annealing leads to a more homogenous base material, eliminating most defects due to solidifying process. Annealing is applied to either ingots, semi-worked products or final products. Annealing of worked products is useful to increase ductility, which might be reduced by hardening during the mechanical processes of production, or to release some residual stresses related to non-uniform cooling or production processes. In particular, annealing can be used on welded parts that are likely to be mired by large residual stresses due to differential cooling;
- *normalization* consists of heating the steel to a temperature between 900 and 925°C (approximately between 1652 and 1697°F), followed by very slow cooling. Normalization eliminates the effects of any previous thermal treatment;
- *tempering* is a thermal process that, similar to annealing, consists of heating the material slightly above the critical temperature followed by a sudden cooling, aimed at preventing any readjustment of the molecular matrix. The main advantage of the tempering process is represented by an increase of hardness that is, however, typically accompanied by a loss of ductility of the material;
- *quenching* consists of heating the tempered part up to a moderate temperature for an extended amount of time, improving the ductility of the material;
- *pack-hardening* is a process that consists of heating of a part when in contact with solid, liquid or gaseous materials that can release carbon. It is a surface treatment that is employed to form a harder layer of material on the outside surface (up to a depth of several millimetres), in order to improve the wearing resistance;

Quenching and tempering can be applied sequentially, resulting in a remarkable strength improvement of ordinary carbon steels, without appreciably affecting the ductility of the product. High strength bolts used in steel structures are typically quenched and tempered.

1.4 Brief Historical Note

Iron refinement has taken place for millennia in partially buried furnaces, fuelled by bellows resulted in a spongy iron mass, riddled of impurities that could only be eliminated by repeated hammering, resulting in *wrought iron*. That product had modest mechanical properties and could be welded by *forging*; that is, by heating the parts to join to a cherry red colour (750–850°C or 1382–1562°F) and then pressing them together, typically by hammering. Wrought iron products could be superficially hardened by *tempering* them in a bath of cold water or oil and the final product was called *steel*. Note that these terms have different implications nowadays.

In thirteenth century Prussia, thanks to an increase in the height of the interred furnaces and the consequent increase in the amount of air forced in the oven by hydraulically actuated bellows, the maximum attainable temperatures were increased. Consequently, a considerably different material from steel was obtained, namely *cast iron*. Cast iron was a brittle material that, once cooled, could not be wrought. On the other hand, cast iron in its liquid state could be poured into moulds, assuming whatever shape was desired. A further heating in an open oven, resulting in a carbon-impoverished alloy, allowed for *malleable iron* to be obtained.

In the past, the difficulties associated with the refinement of *iron ore* have limited the applications of this material to specific fields that required special performance in terms of strength or hardness. Applications in construction were limited to ties for arches and masonry structures, or connection elements for timber construction. The *industrial revolution* brought a new impulse in metal construction, starting in the last decades of the eighteenth century. The invention of the steam engine allowed hydraulically actuated bellows to be replaced, resulting in a further increase of the air intake and the other significant advantage of locating the furnaces near iron mines, instead of forcing them to be close to rivers. In 1784, in England, Henry Cort introduced a new type of furnace, the *puddling furnace*, in which the process of eliminating excess carbon by oxidation took place thanks to a continuous stirring of the molten material. The product obtained (*puddled iron*) was then hammered to eliminate the impurities. An early rolling process, using creased rollers, further improved the quality of the products, which was worked into plates and square cross-section members. Starting in the second half of the nineteenth century, several other significant improvements were introduced. In 1856, at the Congress of the British Society for the Scientific Progress, Henry Bessemer announced his patented process to rapidly convert cast iron into steel. Bessemer's innovative idea consisted of the insufflation of the air directly into the molten cast iron, so that most of the oxygen in the air could directly combine with the carbon in the molten material, eliminating it in the form of carbon oxide and dioxide in gaseous form.

The first significant applications of cast iron in buildings and bridges date back to the last decades of the eighteenth century. An important example is the cast iron bridge on the Severn River at Ironbridge Gorge, Shropshire, approximately 30 km (18.6 miles) from Birmingham in the UK. It is an arched bridge and it was erected between 1775 and 1779. The structure consisted of five arches, placed side by side, over a span of approximately 30 m (98 ft), each made of two parts representing half of an arch, connected at the key without nails or rivets.

The expansion of the railway industry, with the specific need for stiff and strong structures capable of supporting the large weights of a train without large deformations, provided a further spur to the development of bridge engineering. Between 1844 and 1850 the Britannia Bridge (Pont Britannia) on the Menai River (UK) was built; this bridge represents a remarkable example of a continuously supported structure over five supports, with two 146 m (479 ft) long central spans and two 70 m (230 ft) long side spans. The bridge had a closed tubular cross-section, inside which the train would travel, and it was made of puddled iron connected by nails. Robert Stephenson, William Fairbairn and Eaton Hodgkinson were the main designers, who had to tackle a series of problems that had not been resolved yet at the time of the design. Being a statically indeterminate structure, in order to evaluate the internal forces, B. Clapeyron studied the structure applying the three-moment equation that he had recently developed. For the static behaviour of the cross-section, based on experimental tests on scaled models of the bridge, N. Jourawsky suggested some stiffening details to prevent plate instability. The Britannia Bridge also served as a stimulus to study riveted and nailed connections, wind action and the effects of temperature changes.

With respect to buildings, the more widespread use of metals contributed to the development of framed structures. Around the end of the 1700s, cast iron columns were made with square, hollow circular or a cross-shaped cross-section. The casting process allowed reproduction of the classical shapes of the column or capital, often inspired by the architectural styles of the ancient Greeks or Romans, as can be seen in the catalogues of column manufacturers of the age. The first applications of cast iron to bending elements date back to the last years of the 1700s and deal mostly with floor systems made by thin barrel vaults supported by cast iron beams with an inverted T cross-section. During the first decades of the nineteenth century studies were commissioned to identify the most appropriate shape for these cast iron beams. Hodgkinson, in particular, reached the conclusion that the optimal cross-section was an unsymmetrical I-shape with the compression flange up to six times smaller than the tension flange, due to the difference in tensile and compressive strengths of the material. Following this criterion, spans up to 15 m could be accommodated.

The first significant example of a structure with linear cast iron elements (beams and columns) is a seven-storey industrial building in Manchester (UK), built in 1801. Nearing halfway through the century, the use of cast iron slowed to a stop, to be replaced by the use of steel. Plates and corner pieces made of puddle iron had been already available since 1820 and in 1836 I-shape profiles started to be mass produced.

More recent examples of the potential for performance and freedom of expression allowed by steel are represented by tall buildings and skyscrapers. The prototype of these, the *Home Insurance Building*, was built in 1885 in Chicago (USA) with a 12 storey steel frame with rigid connections and masonry infills providing additional stiffness for lateral forces. In the same city, in 1889, the *Rand–McNally* building was erected, with a nine-storey structural frame entirely made of steel.

Early in the twentieth century, the first skyscrapers were built in Chicago and New York (USA), characterized by unprecedented heights. In New York in 1913, the *Woolworth Building* was built, a 60-storey building reaching a height of 241 m (791 ft); in 1929 the *Chrysler Building* (318 m or 1043 ft) was built and in 1930 the *Empire State Building* (381 m or 1250 ft) was built. Other majestic examples are the steel bridges built around the world: in 1890, near Edinburgh (UK) the *Firth of Forth* Bridge was built, possessing central spans of 521 m (1709 ft), while in 1932 the *George Washington Bridge* was built in New York; a suspension bridge over a span of 1067 m (3501 ft).

Many more references can be found in specialized literature, both with respect to the development of iron working and the history of metal structures.

1.5 The Products

A first distinction among steel products for the construction industry can be made between *linear* and *plane products*. The formers are mono-dimensional elements (i.e. elements in which the length is considerably greater than the cross-sectional dimensions).

Plane products, namely sheet metal, which are obtained from plate by an appropriate working process, have two dimensions that are substantially larger than their thickness. Plane products are used in the construction industry to realize floor systems, roof systems and cladding systems. In particular, these products are most typical:

- *ribbed metal decking for bare steel applications*, furnished with or without insulating material, used for roofing and cladding applications. These products are typically used to span lengths up to 12 m or 39 ft (ribbed decking up to 200 mm/7.87 in. depth are available nowadays). In the case of roofing systems for sheds, awnings and other relatively unimportant buildings, non-insulated ribbed decking is usually employed. The extremely light weight of these systems makes them very sensitive to vibrations. These products are also commercialized with added insulation (Figure 1.8), installed between two outer layers of metal decking (as a sandwich panel). For special applications, innovative products have been manufactured, such as the ribbed arched element shown in Figure 1.9, meant for long-span applications
- *ribbed decking products for concrete decks*: these products are usually available in thicknesses from 0.6 to 1.5 mm (0.029–0.059 in.) and with depths from 55 mm (2.165 in.) to approximately 200 mm (7.87 in.). A typical application of these products is the construction of composite or non-composite floor systems: typically, the ribbed decking is never less than 50 mm (2 in., approximately) deep and the thickness of the concrete above the top of the ribs is never less than 40 mm (1.58 in.) thick. The ribbed decking element functions as a stay-in-place form and may or may not be accounted for as a composite element to provide strength to the floor system (Figure 1.10). If composite action is desired, the ribbed decking may have additional ridges and other protrusions in order to guarantee shear transfer between steel and concrete. When composite action is not required, the ribbed decking can be smooth and it just functions as a stay-in-place form. In either case, welded wire meshes or bi-directional reinforcing bars should be placed at the top fibre of the slab to prevent cracking due to creep and shrinkage or due to concentrated vertical loads on the floor.

The choice of cladding and the detailing of ribbed decking elements for roofing and flooring systems (both bare steel and composite) are usually based on tables provided by the manufacturers. For instance, in manufacturers' catalogues tables are generally provided in which the main

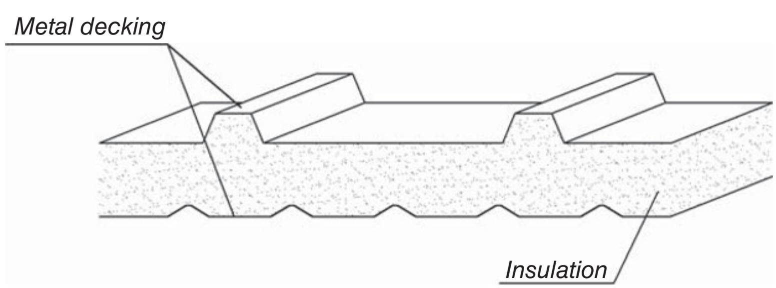


Figure 1.8 Typical insulated element.

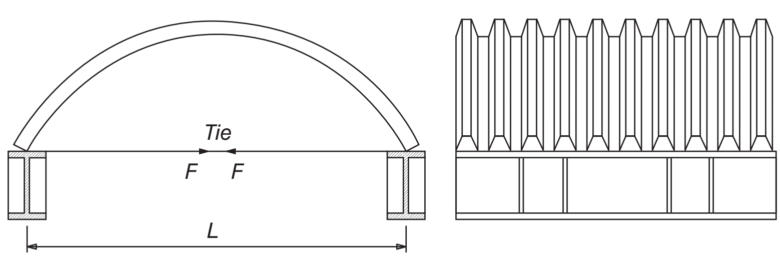


Figure 1.9 Example of a special ribbed decking product.

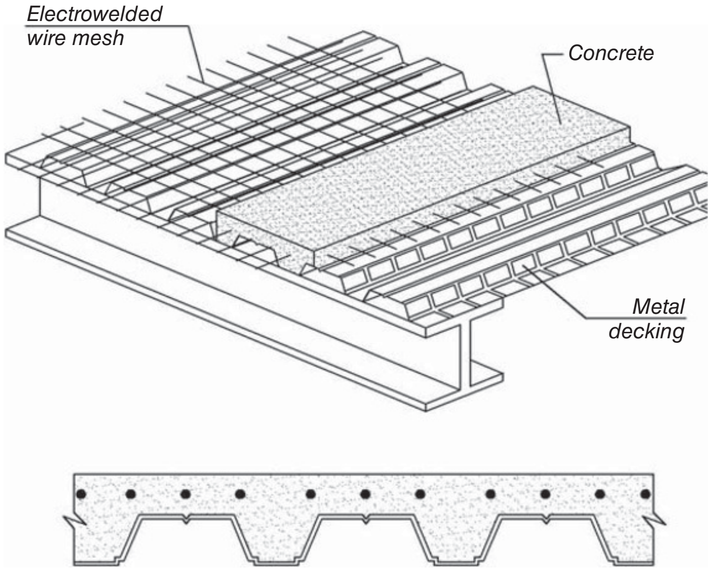


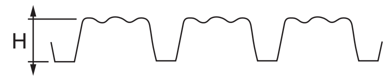
Figure 1.10 Typical steel-concrete composite floor system.

utility data from the commercial and structural points of view are presented: the weight per unit area, the maximum span as a function of dead and live loads and the maximum deflection as a function of the support configuration. Figure 1.11 schematically shows an example of the typical tables developed by manufactures for a bare steel deck: the product is provided with different thicknesses (from 0.6 to 1.5 mm or 0.029 to 0.059 in.): for each thickness, the maximum load is shown as a function of the span.

An aspect that is sometimes overlooked in the design phase is the fastening system of the cladding or roofing panels to the supporting elements, which has to transfer the forces mainly associated with snow, wind and thermal loads. Depending on the configuration of a cladding or a roofing panel with respect to the direction of wind, it can be subject to either a positive or a negative pressure. In the case of cladding, negative (upward) pressures are typically less demanding than positive (downward) pressures. Similarly, negative pressures on roofing systems are typically less controlling than snow or roof live loads. This said, the fastening details between cladding or roofing panels and their supporting elements must be appropriately sized, also taking into account the fact that in the corner regions of a building, or in correspondence to discontinuities such as windows or ceiling openings, local effects might arise causing large values of positive or negative pressures, even when wind speeds are not particularly elevated (Figure 1.12). Concerning thermal variations, it is necessary to make sure that the panels and the fastening systems are capable of sustaining increases or decreases of temperature, mostly due to sun/UV exposure. A rule of thumb that can be followed for maximum ranges of temperature variation, applicable to panels of different colours, in hypothetical summer month and a south-west exposure, is as follows:

- ±18°C (64.4°F) for reflecting surfaces;
- ±30°C (86°F) for light coloured surfaces;
- ±42°C (107.6°F) for dark coloured surfaces.

The fastening systems usually comprise screws with washers to distribute loads more evenly. In some instances, local deformations of thin decks can occur at the fastening locations, causing a potential for leaks.



Product: XYZ H = 75 mm						
Thickness	0.7 mm		0.8 mm			1.5 mm
Weight [kg/m^2]			11.02			
Weight [kg/m]			6.28			
Second moment of area [cm^4/m]			94.71			
Section modulus [cm^3/m]			31.79			

	Distance between supports: span length [m]									
Thickness	1.50	1.75	2.00	2.25	2.75	3.00	3.25	3.50	3.75	5
0.6 *mm*			443							
0.7 *mm*			550							
0.8 *mm*			660							
1.0 *mm*			922							
1.2 *mm*			1151							
1.5 *mm*			1147							
	Distance between supports: span length [m]									
Thickness	1.50	1.75	2.00	2.25	2.75	3.00	3.25	3.50	3.75	5
0.6 *mm*			554							
0.7 *mm*			688							
0.8 *mm*			832							
1.0 *mm*			1152							
1.2 *mm*			1438							
1.5 *mm*			1846							

Figure 1.11 Example of a design table for a bare steel ribbed decking product.

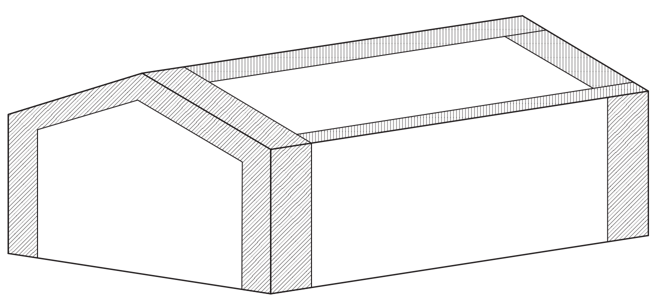


Figure 1.12 Regions that are typically subject to local effects of wind loads.

1.6 Imperfections

The behaviour of steel structures, and thus the load carrying capacity of their elements, depends, sometimes very significantly, on the presence of imperfections. Depending on their nature, imperfections can be classified as follows:

- mechanical or structural imperfections;
- geometric imperfections.

1.6.1 Mechanical Imperfections

The term *mechanical* or *structural imperfections* indicates the presence of *residual stresses* and/or the lack of homogeneity of the mechanical properties of the material across the cross-section of the element (e.g. yielding strength or failure strength varying across the thickness of flanges and web). Residual stresses are a self-equilibrating state of stress that is locked into the element as a consequence of the production processes, mostly due to non-uniform plastic deformations and to non-uniform cooling. If reference is made, for example, to a hot-rolled prismatic member at the end of the rolling process, the temperature is approximately around 600°C (1112°F); the cross-sectional elements with a larger exposed surface and a smaller thermal mass, will cool down faster than other more protected or thicker elements. The cooler regions tend to shrink more than the warmer regions, and this shrinkage is restrained by the connected warmer regions. As a consequence, a stress distribution similar to that shown in Figure 1.13b takes place, with tensile stresses that oppose the shrinkage of the perimeter regions and compressive stresses that equilibrate them in the inner regions. When the warmer regions finally cool down, plastic phenomena contribute to somewhat reduce the residual stresses (Figure 1.13c). Once again, the perimeter regions that have reached the ambient temperature restrain the shrinkage of the inner regions during their cooling process and as a consequence, once cooling has completed, the outside regions are subject to compressive stresses, while the inside regions show tensile stresses (Figure 1.13d).

Figure 1.14 shows the distributions of residual stresses during the cooling phase after the hot-rolling process for a typical I-beam profile and in particular, the phases span from (a), end of the hot-rolling process, to (d), the instant at which the whole profile is at ambient temperature. The magnitude and the distribution of residual stresses depend on the geometric characteristics of the cross-section and, in particular, on the width to thickness ratio of its elements (flanges and webs).

For I-shaped elements, Figure 1.15 shows the distribution of residual stresses (σ_r) as a function of the width/thickness ratio of the cross-sectional elements: terms h and b refer to the height of the profile and to the width of the flange, respectively, while t_w and t_f indicate web and flange thickness, respectively. Stocky profiles; that is, those that have a height/width ratio not greater than 1.2, show tensile residual stresses in the middle of the flanges and compressive residual stresses at the extremes of the flanges, while in the web there can be either tensile or compressive residual stresses, depending on the geometry. For slenderer profiles with $h/b \geq 1.7$, the middle part of the flanges show prevalently tensile residual stresses, while compressive residual stresses can be found in the middle region of the web.

Residual stresses can affect the load carrying capacity of member, especially when they are subject to compressive forces. For larger cross-sections, the maximum values of the residual stresses can easily reach the yielding strength of the material.

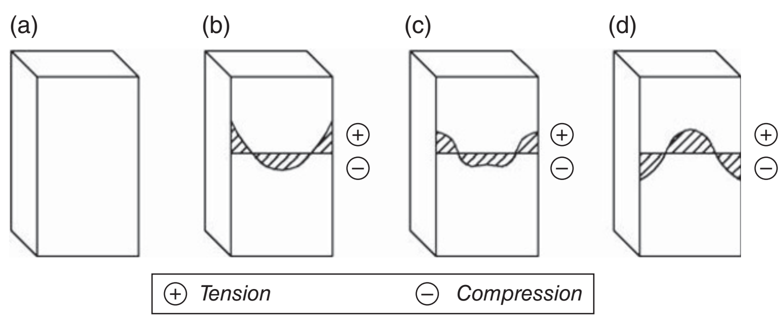


Figure 1.13 Residual stress distribution in a hot-rolled rectangular profile during the cooling phase (temporary from a to d).

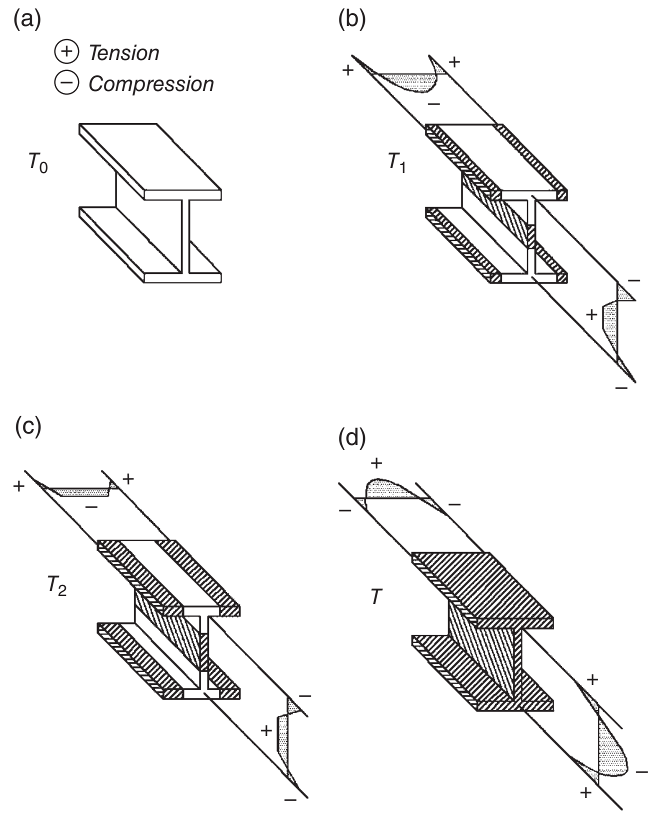


Figure 1.14 Distribution of residual stresses during the cooling phase of an I-shape.

In the case of cold-formed profiles and plates, the raw product is a hot- or cold-rolled sheet. If the rolling process is performed at ambient temperature, the outermost fibres, in contact with the rollers, tend to stretch, while the central fibres remain undeformed. As a consequence, a self-equilibrated residual state of stress arises, such as the one shown in Figure 1.16, due to the differential elongation of the fibres in the cross-section.

In the case of hot-rolling of a plate, the residual stresses develop similarly to those presented for the rectangular (Figure 1.13) and for the I-shaped (Figure 1.14) sections.

In the case of cold-formed profiles or metal decks, an additional source of imperfections is the cold-formation process. The bending processes in fact alter the mechanical properties of the material in the vicinity of the corners. In order to permanently deform the material, the process brings it beyond its yielding point so that the desired shape can be attained. As an example, Figure 1.17 shows the values of the yielding strength (f_y) and of the ultimate strength (f_u) for the virgin material compared to the same values for the cold-formed profile at different locations. It is apparent how the cold-formation process increases both yielding and failure strengths, with a larger impact on the yielding strength.

From the design standpoint, recent provisions on cold-formed profiles, among which part 1–3 of Eurocode 3 (EN-1993-1-3) allows account for a higher yielding strength of the material, due to the cold-formation process, when performing the following design checks:

h/b	*Cross-section*		σ_r *(web)*	σ_r *(flange)*	t_w/h	t_w/b	t_f/h	t_f/b
≤ 1.2	*b*, *h*, t_w, t_f	*a*	*T*, *C*, *T*	*T*, *C*, *C*	0.032 ÷ 0.040	0.032 ÷ 0.040	0.045 ÷ 0.061	0.045 ÷ 0.060
≤ 1.2		*b*	*T*	*T*, *C*, *C*	0.075 ÷ 0.100	0.078 ÷ 0.112	0.091 ÷ 0.162	0.093 ÷ 0.182
≤ 1.2		*c*	*C*, *T*, *C*	*C*	0.062 ÷ 0.068	0.068 ÷ 0.073	0.104 ÷ 0.114	0.113 ÷ 0.121
>1.2 <1.7		*c*			0.031 ÷ 0.032	0.042 ÷ 0.048	0.048 ÷ 0.051	0.062 ÷ 0.080
>1.2 <1.7		*d*	*T*, *C*, *T*	*T*, *C*, *C*	0.030	0.046	0.051	0.077
≥1.7		*e*	*T*, *C*, *T*	*T*	0.018 ÷ 0.028	0.039 ÷ 0.056	0.025 ÷ 0.043	0.063 ÷ 0.085

Figure 1.15 Distribution of residual stresses in hot-rolled I-shapes.

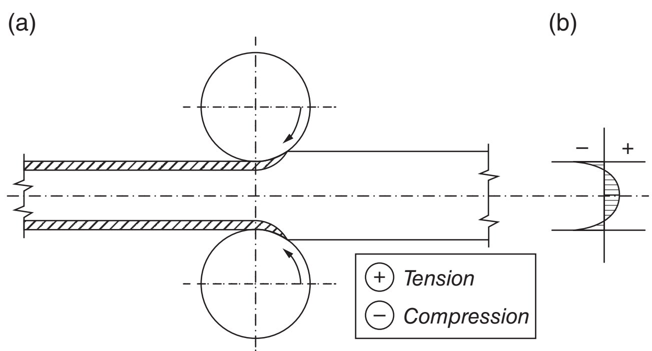


Figure 1.16 Residual stresses in a cold-rolled plate.

- design of tension members;
- design of compression members of class 1, 2 and 3, in accordance with the criteria described in Chapter 4 (Cross-Section Classifications), that is fully engaged cross-sections, in the absence of local buckling;

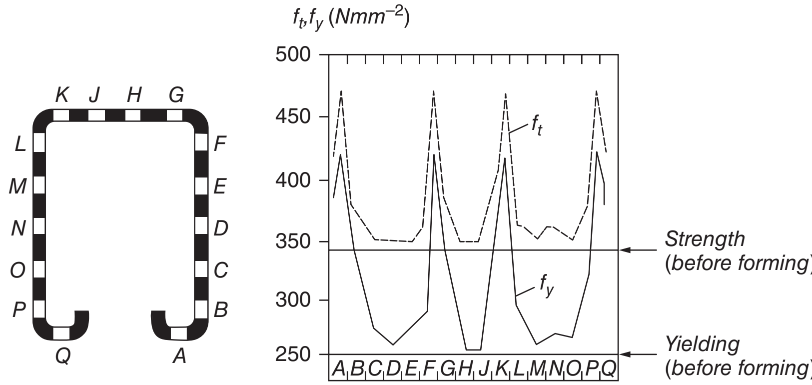


Figure 1.17 Variation of the mechanical properties of the material after cold-formation.

- design of flexural members with compression elements of class 1, 2 and 3 (i.e. with fully engaged compression elements, in the absence of local buckling).

The stub column test (Section 1.7.2) can be used to experimentally evaluate the increase of strength of a cold-formed member; alternatively, the post-forming average yielding strength f_{ya} can be evaluated based on the virgin material's yielding and ultimate strength (f_{yb} and f_u, respectively) as follows:

$$f_{ya} = f_{yb} + \frac{(f_u - f_{yb}) \cdot k \cdot n \cdot t^2}{A_g} \tag{1.4a}$$

$$f_{ya} \leq \frac{f_{yb} + f_u}{2} \tag{1.4b}$$

in which coefficient k accounts for the type of process ($k = 5$ in all the cases except for the continuous formation with rollers for which $k = 7$ has to be adopted), A_g is the gross area of the cross-section, n is the number of 90° bends with an inner radius $r \leq 5\,t$ (bends at angles different than 90° are taken into account with fractions of n) and t is the thickness of the plate or coil before forming.

The average value of the increased yielding strength f_{yb} cannot be used when calculating the effective cross-section area, or when designing members that, after the cold forming process, have been subject to heat treatments such as annealing, which reduce the residual stresses due to cold forming.

1.6.2 Geometric Imperfections

The term *geometric imperfections* refers to those differences that can be found between the theoretical shape and real size of the members, or of the structural systems as a whole, and the actual members or as-built structure. In particular, geometric imperfections can be subdivided into:

- cross-sectional imperfections;
- member imperfections;
- structural system imperfections.

Cross-sectional imperfections are related to the dimensional variation of the cross-sectional elements with respect to the nominal dimensions and can be ascribed essentially to the production process. Different values of area, moments of inertia and section moduli can influence the performance of the cross-section (e.g. in terms of load-carrying capacity or bending moment resistance). Tolerances are established by standards for the final products, not only in terms of maximum difference between actual and nominal linear dimensions, but also with reference to:

- perpendicularity tolerance between cross-sectional elements;
- tolerances with respect to axes of symmetry;
- straightness tolerance.

Figure 1.18 shows few examples of parameters to be measured for the tolerance checks for an I-shaped section.

Among *member imperfections*, the *longitudinal* (*bow*) imperfection is certainly the most important. It consists essentially of a deviation of the axis of the element from the ideal straight line and is caused by the production process. This out-of-straightness defect can cause load eccentricity, as well as an increased susceptibility to buckling phenomena.

Structural system imperfections can be ascribed to various causes, such as variability in the lengths of framing members, lack of verticality of columns and of horizontality of beams, errors in the location of foundations, errors in the placement of the connections and so on. These imperfections must be carefully accounted for during the global analysis phase. In a very simplified but efficient way, additional fictitious forces (notional loads) can be applied to the structure to reproduce the effects of imperfections. For example, the lack of verticality of columns in sway frames is accounted for by adding horizontal forces to the perfectly vertical columns (Figure 1.19), proportional to the resultant vertical force F_i acting on each floor.

This design simplification can be explained directly with reference to a cantilever column of height h with an out-of-plumb imperfection and subject to a vertical force N at the top. The additional bending moment M due to the lack of verticality, expressed by angle φ (Figure 1.20), can be approximated at the fixed end as:

$$M = N[h \cdot \tan(\varphi)] \tag{1.5}$$

Within the small displacement hypothesis (thus approximating *tan* (φ) with the angle φ itself), the effect of this imperfection can be assimilated to that of a fictitious horizontal force F acting at the top of the column and causing the same bending moment at the base of the column. The magnitude of F is thus given by:

$$F = \frac{M}{h} = N\phi \tag{1.6}$$

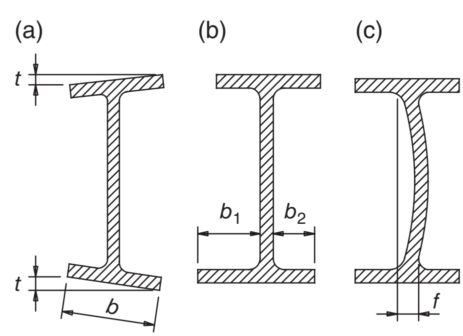


Figure 1.18 Additional tolerance checks for I-shapes: (a) perpendicularity tolerance, (b) symmetry tolerance and (c) straightness tolerance.

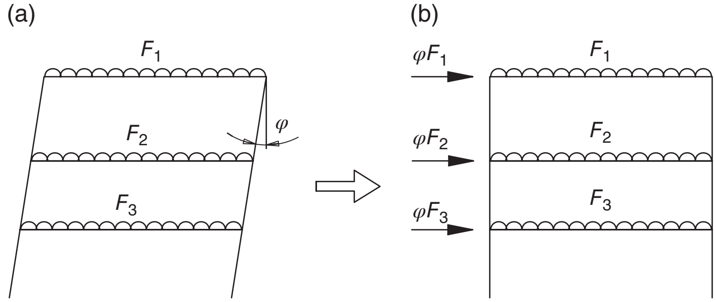


Figure 1.19 Horizontal notional loads equivalent to the imperfections for a sway frame.

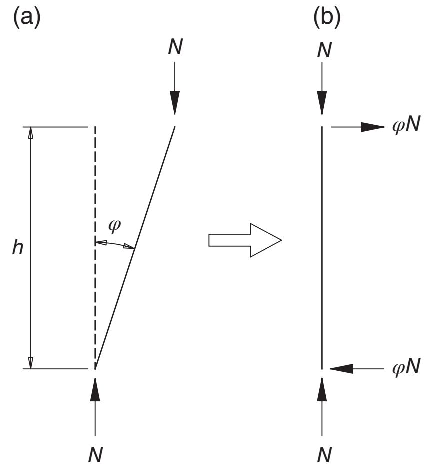


Figure 1.20 Imperfect column (a) and horizontal equivalent force (b).

1.7 Mechanical Tests for the Characterization of the Material

An in-depth knowledge of the mechanical characteristics of steel, as well as of any other structural material, is of paramount importance for design verification checks. Additionally, besides the mandatory tests performed at the factory on base materials and worked products, it is often important to perform laboratory tests on coupons cut from plane and linear *in-situ* products in order to validate the design hypotheses with actual material characteristics.

For each laboratory test there are very specific standardization requirements. Globally *ISO* (*International Organization for Standardization*) and in Europe *CEN* (*European Committee for Standardization*) standardization requirements are provided, whereas in the US, the *ASTM* is the governing body, emanating standards that contain detailed instructions on the geometry of the coupons, on the testing requirements, on the equipment to be used and on the presentation and use of the test results.

Among the most important tests for the characterization of steel there are: chemical analysis, macro- and micro-graphic testing. In particular, chemical analysis is very important to determine the main properties of steel, among which are weldability, ductility and resistance to corrosion, and to determine the percentage of carbon and other desired and undesired alloying elements.

Some alloying elements have no direct impact on the material strength, but play a key role in the determination of other properties, such as weldability and corrosion resistance. As discussed in the introductory section, in addition to carbon and iron, impurities can be present that can have a detrimental effect on the behaviour of the material, such as favouring brittleness. Since it is virtually impossible and uneconomical to completely eliminate such impurities, it is important to verify that their content is within acceptable limits. Due to these considerations, based on the grade of steel considered, the standards specifying material characteristics (EN 10025, ASTM A992, ASTM A36, ASTM A490 are some examples) contain tables defining the maximum percent content of some alloying elements (typically, carbon – C, silica – Si, phosphorous – P, sulfur – S and nitrogen – N) or a range of acceptability for other alloying elements (such as manganese – Mn, chromium – Cr, molybdenum – Mo and copper – Cu).

Chemical analyses can be performed either on the molten material (ladle analysis) or on the final product (product analysis), even after it has been erected, by means of a sample site extraction. It is possible that the limits prescribed for the chemical makeup of the material can be different, based on whether the analysis has been performed on the ladle material or on the final product (in general, the values prescribed for the analysis on molten material are more stringent than the ones on the final product).

The weldability property is directly related to a carbon equivalent value (CEV), based on the results of the analysis on the ladle material, defined as follows:

$$\mathrm{CEV} = \mathrm{C} + \frac{\mathrm{Mn}}{6} + \frac{\mathrm{Cr} + \mathrm{Mo} + \mathrm{V}}{5} + \frac{\mathrm{Ni} + \mathrm{Cu}}{15} \tag{1.7}$$

in which C indicates the percentage content of carbonium, Mn for manganese, Cr for chromium, Cu for copper, Mo for molybdenum and Ni for nickel.

In order to ensure good weldability characteristics, the material should have as low a CEV as possible, with maximum values prescribed by the various standards.

The macrographic test is performed to establish the de-oxidation and the de-carbonation indices of steel, related to weldability. The micrographic test allows analysis of the crystalline structure of steel and its grain size and the ability to relate some mechanical characteristics of the material to its micro-structure as well as to investigate the effects that thermal treatments have on the material.

In the following, a brief description of some of the most important mechanical laboratory tests performed on structural steel is presented.

1.7.1 Tensile Testing

The most important and well-known mechanical test is the *uniaxial tensile test*. This test allows measurement of some important mechanical characteristics of steel (yield strength, ultimate strength, percentage elongation at failure and the complete stress-strain curve, as discussed in Section 1.1). The test consists of the application of a tensile axial force to a sample obtained according to specific standards (EN ISO 6892-1 and ASTM 370-10). The tensile force is applied with an intensity that increases with an established rate, recording the extension Δ over a gauge length L_0 in the middle of the sample (Figure 1.21).

The stress σ is calculated dividing the measured applied force by the nominal cross-sectional area of the coupon (A_{nom}), while the strain ε is calculated by means of change of the gauge length:

$$\varepsilon = \frac{\Delta}{L_0} = \frac{L_d - L_0}{L_0} \tag{1.8}$$

in which L_d is the distance between the gauge marks during loading.

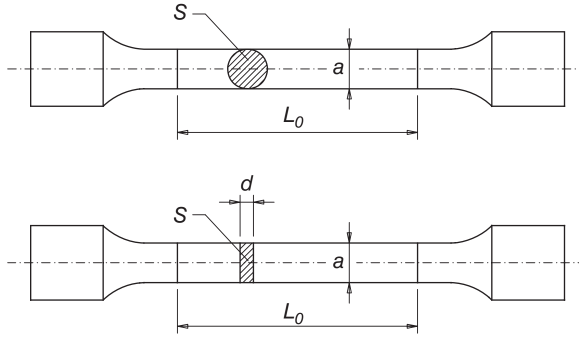


Figure 1.21 Typical sample for rolled products.

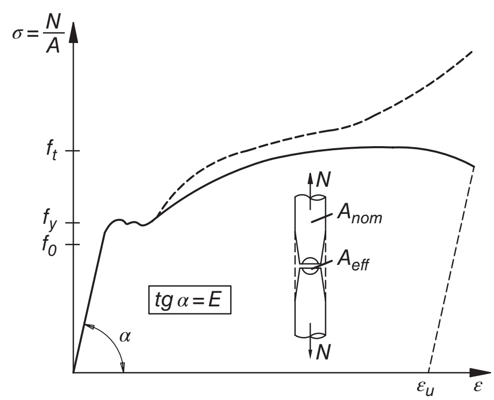


Figure 1.22 Typical stress-strain (σ–ε) relationship for structural steels.

For steel materials with a carbon percentage of up to 0.25%, that is for structural steels, the typical stress-strain relationship is shown in Figure 1.22. The initial branch of the curve is very close to linear elastic.

From the slope of the initial branch of the σ–ε curve, the longitudinal elastic modulus or Young's modulus, can be calculated as $E = \tan(\alpha)$. Once the value of the stress indicated with f_0 in the figure is reached, which can be defined as the limit of proportionality, there is no more direct proportionality between stress and strain, but the material still behaves elastically. Corresponding to a stress f_y, yielding occurs and the stress-strain response is characterized by a slightly undulating response that is substantially horizontal due to the onset plastic deformations (Figure 1.23).

It is worth noting that low-carbon steels usually show two distinct values of the yielding stress: an upper yielding point, R_{eH}, after which the strains increase with a local decrease of the stress, and a lower yielding point, R_{eL}, at which there are no appreciable reductions in the stress associated with an increase in strain. The upper yielding point R_{eH} is significantly affected by the load rate, unlike the lower yielding point, which is substantially independent of the rate and is thus usually taken as the yielding strength to be used for design, that is $f_y = R_{eL}$.

Until the yielding stress is reached, the transverse deformations of the coupon due to Poisson's effect are very small. The effective cross-sectional area of the coupon (A_{eff}) is considered, with a small approximation, to be equal to the nominal cross-sectional area ($A_{\text{eff}} = A_{\text{nom}}$). For higher levels of the applied force, the transverse deformations are not negligible anymore, but for the sake of practicality the stress is always calculated making reference to the nominal area of the

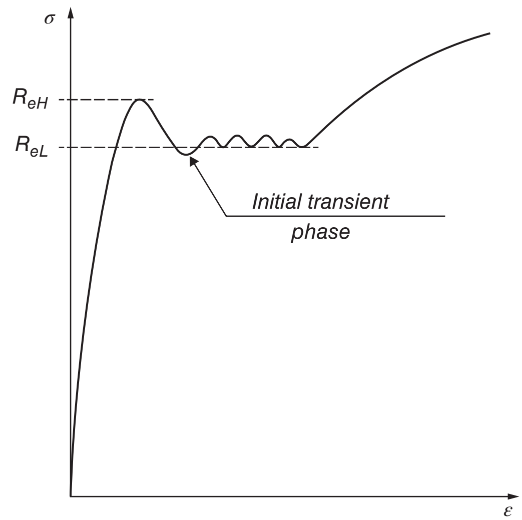


Figure 1.23 Upper and lower yielding points for structural steel.

undeformed cross-section (A_{nom}). As a consequence, the resulting stress-strain diagram results in the solid-line curve in Figure 1.22, which is characterized by a *softening* branch with increasing stresses corresponding to increasing strains, which is the *hardening* branch. This branch ends when the transverse deformations of the coupons stop being uniform along the length of the coupon, and start focusing in a small region towards the middle of the coupon itself. This phenomenon is identified as *necking* (reduction of area) and one of the immediate consequences is that an increase in strain now corresponds to a decrease in stress, until the coupon fails. If the effective cross-sectional area is used (A_{eff}), the resulting stresses would be always increasing until failure, because even if the carried force decreases, so does the cross-sectional area (dashed curve in Figure 1.22), showing hardening all the way up to failure.

The failure strength f_u is based on the maximum value of the applied load during the test, whereas the failure strain ε_u, more commonly measured as the percent elongation at failure, is evaluated according to Eq. (1.8), putting the two parts of the broken coupon back together so that a ultimate length L_u between the gauge points can be measured.

Usually, structural steels are required to have a sufficient elongation at failure so that an adequate ductility can be expected, allowing for large plastic deformations without failure. In the absence of ductility, a considerable amount of design simplifications provided in all specifications could not be used, significantly complicating all design tasks.

The constitutive law, and consequently the material mechanical characteristics, depends on the loading rate and on the temperature at which the tensile test is performed (usually ambient temperature). With an increase in temperature, the performance parameters of steel decrease sensibly, including a reduction of the modulus of elasticity, yielding strength and of the failure strength. Above approximately 200°C (392°F), the yielding phenomenon tends to disappear in favour of a basically monotonic stress-strain curve (Figure 1.24).

1.7.2 Stub Column Test

The stub column test, also known as the global compression test, is performed on stubs cut from steel profiles (Figure 1.25) sufficiently short so that global buckling phenomena will not affect the results. This test, used in the past mainly in the US, is of great interest, because it allows

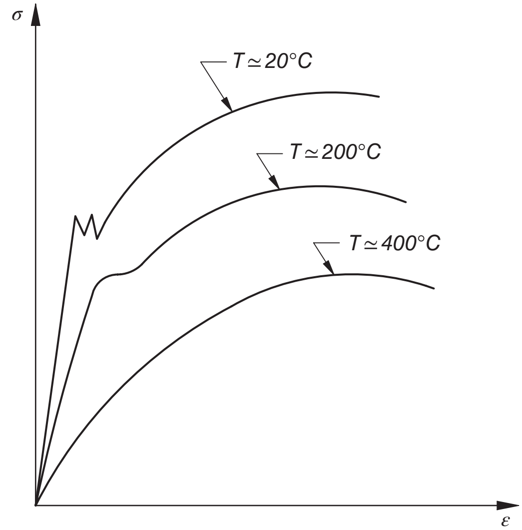


Figure 1.24 Influence of temperature on the constitutive law of steel.

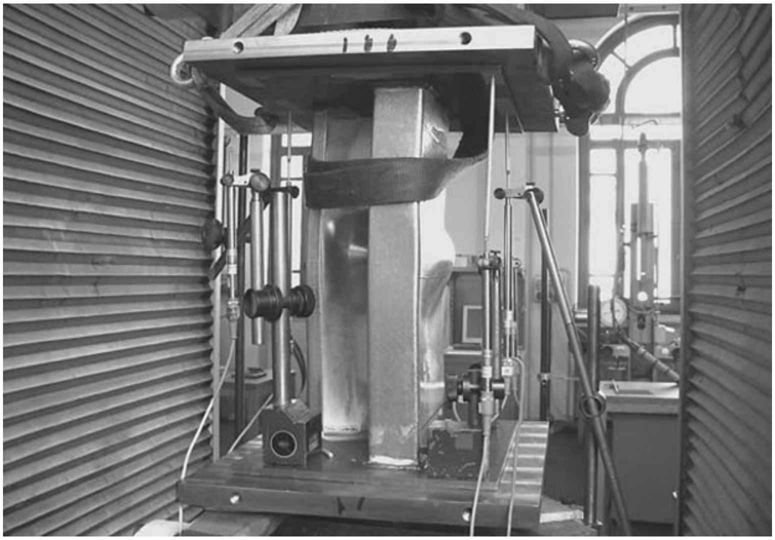


Figure 1.25 Testing of a specimen in a stub column test.

measurement of a stress-strain curve for the whole cross-section of a member, not just for a coupon cut from it.

The stub column test, in fact, provides the mechanical properties of the materials averaging out the structural imperfections of the profile due, for instance, to the presence of residual stresses or to different yielding of failure strengths in various parts of the profile (web, flanges, etc.). Some profiles, in fact, due to the production process, may show a variation of mechanical properties across the thickness and also have a non-uniform distribution of residual stresses. An equivalent yielding strength ($f_{y,\mathrm{eq}}$) can be evaluated as a function of the experimental load that causes yielding of the specimen ($P_{y,\mathrm{exp}}$) and of the cross-sectional area (A) as follows:

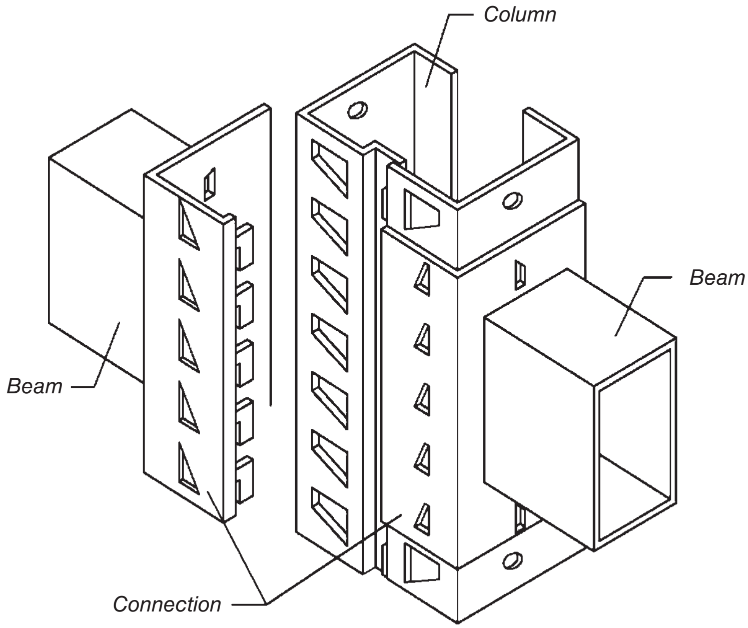


Figure 1.26 Typical components of adjustable storage pallet racks.

$$f_{y,eq} = \frac{P_{y,\exp}}{A} \tag{1.9}$$

The stub column test of stocky elements is very important to determine the performance characteristics, especially when the cross-sectional geometry is particularly complex. As typical examples, industrial storage rack systems can be considered, in which the column, typically a thin-walled cold-formed member, has a regular pattern of holes to facilitate modular connections (Figure 1.26) and thus does not have uniform cross-sectional area over its length.

For such elements, the load carrying capacity is affected by local and distortional buckling phenomena, due to the small thickness of the profiles and to the use of open cross-sections. Often, due to the non-uniform cross-section of these elements, there are no theoretical approaches to evaluate their behaviour. In these circumstances, the experimental ratio of the failure load to the yielding load can be used to equate the element in question with an equivalent uniform cross-section member and then use the theoretical equations available for that case. In the case of profiles with regular perforation systems, based on the experimental axial load capacity ($P_{\exp}$) and on the material yielding strength (f_y), an equivalent cross-sectional area can be determined as:

$$A_{eq} = \frac{P_{\exp}}{f_y} \tag{1.10}$$

1.7.3 Toughness Test

The toughness test measures the amount of energy required to break a specially machined specimen, evaluating the toughness of the material, that is its ability to resist impact and in general to avoid brittle behaviour. The standardized test utilizes a gravity-based pendulum device (Charpy's

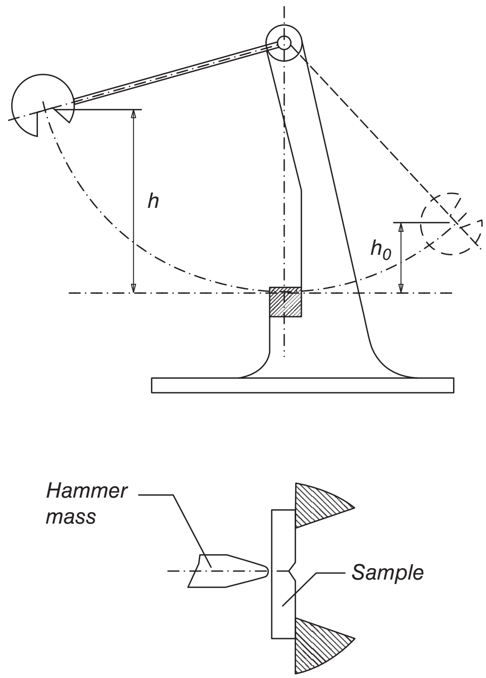


Figure 1.27 Charpy V-notch test.

pendulum) and the specimen is a rectangular bar with a suitable notch having a standardized shape (Figure 1.27). The impact is provided by a hammer suspended above the specimen that is released starting at a relative height h. Upon impacting the specimen, which is restrained by two supports at its ends, the hammer continues its swing climbing on the opposite side to a new relative height h_0 (with $h_0 < h$). The difference between h and h_0 is proportional to the energy absorbed by the specimen, E_p, that is:

$$E_p = G(h - h_0) \tag{1.11}$$

in which G is the weight of the hammer.

Toughness is measured by the ratio between the energy E_p and the area of the notched cross-section of the specimen. The tougher is the metal, the smaller the height h_0.

Toughness values depend on the shape of the specimen and in particular on the details of the notch. Among standardized notch types, it is worth mentioning the types: type *KV*, type K_{cu}, type *Keyhole*, type *Messenger* and type *DVM*. Usually toughness decreases as the mechanical strength increases and it is greatly influenced by the testing temperature, which affects the crack formation and propagation.

A temperature value can be identified, referred to as *transition temperature*, below which toughness is reduced so much to be unacceptable, due to the excessive brittleness of the material. For special applications (structures in extremely cold climates, freezing plants, etc.), metals with a very low transition temperature must be used. Toughness is expressed in energy units, usually Joules, at a specified temperature. Sometimes, the code used to identify toughness (e.g. JR, J0 or J2) follows the identification of the steel type. For structural steel, the minimum toughness required is usually 27 J, as already briefly discussed in Section 1.1. Table 1.5 contains an example of required toughness values for various European designations.

Table 1.5 Codes used for toughness requirement (Charpy V-notch).

Test temperature (°C)	Minimum value of energy 27 J	40 J	60 J
20	JR	LR	KR
0	J0	L0	K0
−20	J2	L2	K2
−30	J3	L3	K3
−40	J4	L4	K4
−50	J5	L5	K5
−60	J6	L6	K6

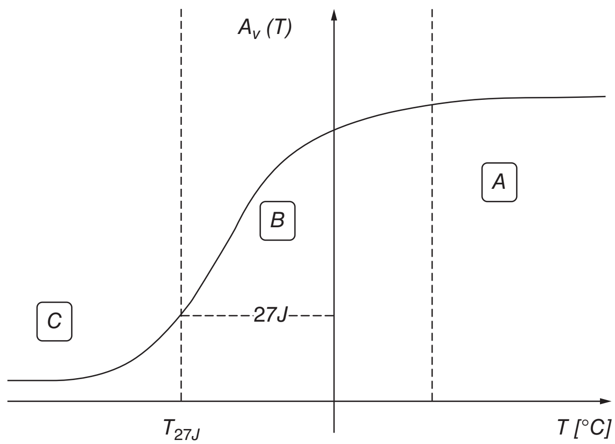


Figure 1.28 Energy associated with the toughness test as a function of the testing temperature.

For welded steel construction, and especially for those structures subject to low temperatures, it is advisable to choose steels with good toughness at low temperatures. Thermo-mechanical rolling typically produces these kinds of steel. It is also worth keeping in mind that good toughness also corresponds to good weldability.

Despite the fact that the ductility of a particular class of steel can be evaluated by means of laboratory tests, the same material in special conditions could show a fragile behaviour associated with a sudden failure at low stresses, even below yielding.

Fragile behaviour depends on several factors. Among these, the temperature at which the element is subject to in-service can cause this type of failure. With reference to the Charpy V-notch test, indicating with $A_v(T)$ the work performed by the hammer as a function of the test temperature (T), a diagram similar to the one in Figure 1.28 can be obtained, characterized by the following three regions:

- region A, corresponding to higher temperatures, with higher toughness values, indicating a material capable of undergoing large plastic deformations;
- region C, corresponding to lower temperatures, with very small toughness values and thus elevated brittleness;
- region B, between regions A and C, is the transition zone and is characterized by a very variable behaviour, with a rapid decrease in toughness as the temperature decreases.

Brittle failure can also be influenced by the rate of increase of stresses, as there is the possibility of localized overstresses that could practically prevent the onset of plastic deformations, causing sudden failures. The width of the three regions in Figure 1.28 is a function of the chemical composition of the steel. In particular, the transition temperature can be lowered by acting on the content of carbon, manganese and nickel, and/or with annealing or quenching and tempering heat treatments.

1.7.4 Bending Test

The bending test is used to evaluate the capacity of the material to withstand large plastic deformations at ambient temperature without cracking. The specimen, usually with a solid rectangular cross-section (but circular or rectangular solid specimens can also be used) is subject to a plastic deformation by means of a continuous bending action without load reversal. In detail, as shown in Figure 1.29, the specimen is placed over two roller bearings with radius R and then a force is applied by means of another roller with diameter D until the ends of the specimen form an angle α with respect to each other.

The values of R and D depend on the size of the specimen. At the end of the test, the specimen's bottom face is examined to ascertain that no cracks have formed.

1.7.5 Hardness Test

Hardness, for metals, represents the resistance that the material opposes to the penetration of another body and thus allows gathering of information on the resistance to scratching, to abrasion, to friction wear and to localized pressure.

The hardness test measures the capacity of the material to absorb energy and can also provide an estimate of the material strength. The test itself consists in the measurement of the indentation left on the specimen surface by a steel sphere that is pressed onto the specimen with a predetermined amount of force for a predetermined amount of time (Figure 1.30).

Depending on the shape of the tip penetrating device, there are various hardness tests that are chosen based on the material to be tested. Among these, the *Brinell Hardness Test*, the *Vickers Hardness Test* and the *Rockwell Hardness Test* are the most important.

The ISO 18265 norm, 'Metallic Materials Conversion of Hardness Values', has been specifically written to provide conversion values among the various types of hardness tests.

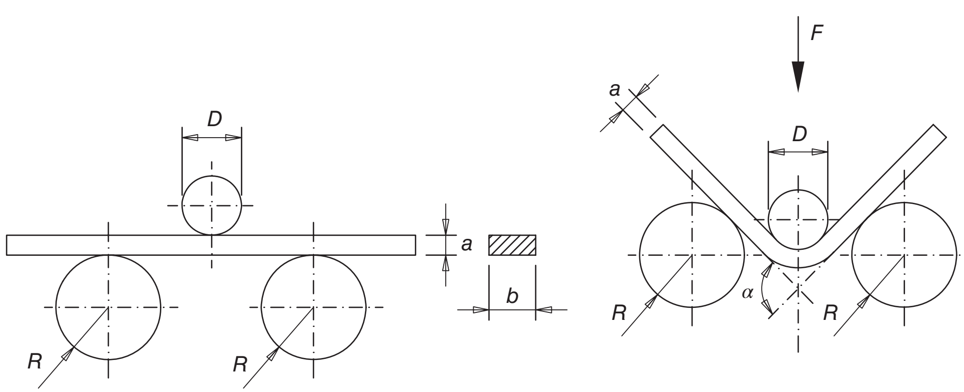


Figure 1.29 Bending test.

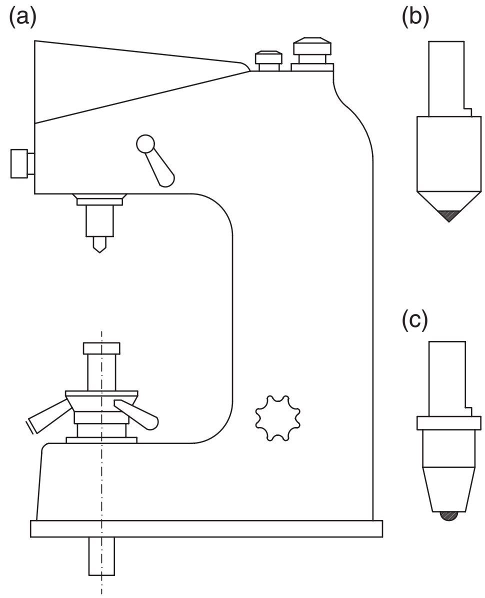


Figure 1.30 Hardness test: (a) durometer, (b) conical tip and (c) spherical tip.

Thanks to the somewhat direct relationship between hardness and strength, hardness testing is sometimes used to evaluate the tensile strength of metal elements in the field when a destructive test is not an option. In the past, several research projects have been conducted to establish a correlation between hardness and tensile strength in some materials. It is worth mentioning that, in 1989, the Technical Report ISO/TR 10108 '*Steel-Conversion of Hardness Values to Tensile Strength Values*' was published, reporting the range of tensile strength values corresponding to experimentally measured hardness.

CHAPTER 2

References for the Design of Steel Structures

2.1 Introduction

A structure has to be designed and executed in such a way that, during its intended life, it will support all load applied, with an appropriate degree of reliability and in an economical way. A very important task of each designer is to size the skeleton frame to be safe for the entire life of the structure. This condition is guaranteed if, from the construction stage until decommissioning due to old age, this condition is satisfied for each component of the structure:

$$\text{Effects of actions} < \text{Resistance} \tag{2.1}$$

With regard to internal forces and moments, on the basis of the structural model, designers must select the loads and individuate the load combinations of interest. It is essential to estimate the loads acting on the structure defining their values appropriately, that is without exaggeration (otherwise, the resulting system could be too heavy and un-economical) but at the same time avoiding load values that are too low, which would lead to unsafe design.

As far as the resistance is concerned, suitable design limits are fixed by codes, which refer to the performance of the cross-section as well as of the parts of whole structural systems. Designers have to evaluate them correctly.

A fundamental requisite of design is that the structure must be safe throughout its use, otherwise all the subjects contributing to the safety of building are responsible, that is designer, project manager, builders, acceptance test engineers and so on.

The concept of building safety is very old. Babylonian king, Hammurabi, about 4000 years ago imposed the law of retaliation to builders, with a penalty proportional to the social class of membership of the parties involved. In this code, which is the first significant example of treaty law, it was prescribed that:

- *If a builder builds a house for someone and completes it, the owner should give him a fee of two shekels in money for each sar of surface (rule 228).*
- *If a builder builds a house for someone, and does not construct it properly, and the house which he built falls and kills its owner, then the builder should be put to death (rule 229).*
- *If it kills the son of the owner the son of that builder should be put to death (rule 230).*
- *If it kills a slave of the owner, then he should pay slave for slave to the owner of the house (rule 231).*

Structural Steel Design to Eurocode 3 and AISC Specifications, First Edition. Claudio Bernuzzi and Benedetto Cordova.

It is important to note the attention given to the concept of durability by this very old code: even then it was assumed that a fundamental requirement of the construction was no damage should occur during its entire life.

About two centuries ago, Napoleon Bonaparte introduced the concept of responsibility extended to the first decade of age of the construction. In addition to the builder, the presence of a technician (e.g. the designer) was required too, sharing all the responsibilities and, eventually prison, in case of collapse or damage within a decade from putting the structure into service.

2.1.1 European Provisions for Steel Design

In 1975, the Commission of the European Community decided on an action programme in the field of construction, based on article 95 of the Treaty, with the objective of eliminating all the technical obstacles to trade and the harmonization of technical specifications. Within this action programme, the Commission took the initiative to establish a set of harmonized technical rules for the design of construction works that, in the first stage, would serve as an alternative to the national rules in force in the Member States and, ultimately, would replace them. For 15 years, the Commission, with the help of a Steering Committee with Representatives of Member States, conducted the development of the Eurocodes programme, which led to the first generation of European codes in the 1980s.

The Structural Eurocode programme comprises the following standards consisting of 10 parts:

EN 1990 – Eurocode 0: Basis of structural design;
EN 1991 – Eurocode 1: Actions on structures;
EN 1992 – Eurocode 2: Design of concrete structures;
EN 1993 – Eurocode 3: Design of steel structures;
EN 1994 – Eurocode 4: Design of composite steel and concrete structures;
EN 1995 – Eurocode 5: Design of timber structures;
EN 1996 – Eurocode 6: Design of masonry structures;
EN 1997 – Eurocode 7: Geotechnical design;
EN 1998 – Eurocode 8: Design of structures for earthquake resistance;
EN 1999 – Eurocode 9: Design of aluminium structures.

The Eurocode standards provide common structural design rules for everyday use for the design of whole structures and component products of both traditional and innovative nature. Unusual forms of construction or design conditions are not specifically covered and additional expert consideration will be required by the designer in such cases.

The National Standards implementing Eurocodes comprise the full text of the Eurocode (including any annexes), as published by CEN, which may be preceded by a National Title Page and National Foreword, and may be followed by a National Annex. This may only contain information on those parameters that are left open in the Eurocode for national choice, known as Nationally Determined Parameters, to be used for the design of buildings and civil engineering works to be constructed in the country concerned, that is:

- values and/or classes where alternatives are given in Eurocodes,
- values to be used where a symbol only is given in Eurocodes,
- country specific data (geographical, climatic, etc.), for example, a snow map,
- the procedure to be used where alternative procedures are given in Eurocodes,
- references to non-contradictory complementary information to assist the user to apply Eurocodes.

There is a need for consistency between the harmonized technical specifications for construction products and the technical rules for works.

The EN 1993 (in the following identified as EC3 or Eurocode 3) is intended to be used with Eurocodes EN 1990 (Basis of Structural Design), EN 1991 (Actions on structures) and EN 1992 to EN 1999, when steel structures or steel components are referred to.

EN 1993-1 is the first of six parts of EN 1993 (Design of Steel Structures), to which this book refers to. It gives generic design rules intended to be used with the other parts EN 1993-2 to EN 1993-6. It also gives supplementary rules applicable only to buildings. EN 1993-1 comprises 12 subparts EN 1993-1-1 to EN 1993-1-12 each addressing specific steel components, limit states or materials. In the following, the list of all the EN 1993 documents is presented:

EN 1993-1: Eurocode 3: Design of steel structures – Part 1, which is composed by:
- EN 1993-1-1: Eurocode 3: Design of steel structures – Part 1-1: General rules and rules for buildings;
- EN 1993-1-2: Eurocode 3: Design of steel structures – Part 1-2: General rules – Structural fire design;
- EN 1993-1-3: Eurocode 3 – Design of steel structures – Part 1-3: General rules – Supplementary rules for cold-formed members and sheeting;
- EN 1993-1-4: Eurocode 3 – Design of steel structures – Part 1-4: General rules – Supplementary rules for stainless steels;
- EN 1993-1-5: Eurocode 3 – Design of steel structures – Part 1-5: Plated structural elements;
- EN 1993-1-6: Eurocode 3 – Design of steel structures – Part 1-6: Strength and Stability of Shell Structures;
- EN 1993-1-7: Eurocode 3 – Design of steel structures – Part 1-7: Plated structures subject to out of plane loading;
- EN 1993-1-8: Eurocode 3: Design of steel structures – Part 1-8: Design of joints;
- EN 1993-1-9: Eurocode 3: Design of steel structures – Part 1-9: Fatigue;
- EN 1993-1-10: Eurocode 3: Design of steel structures – Part 1-10: Material toughness and through-thickness properties;
- EN 1993-1-11: Eurocode 3 – Design of steel structures – Part 1-11: Design of structures with tension components;
- EN 1993-1-12: Eurocode 3 – Design of steel structures – Part 1-12: Additional rules for the extension of EN 1993 up to steel grades S 700;

EN 1993-2: Eurocode 3 – Design of steel structures – Part 2: Steel Bridges;
EN 1993-3-1: Eurocode 3 – Design of steel structures – Part 3-1: Towers, masts and chimneys – Towers and masts;
EN 1993-3-2: Eurocode 3 – Design of steel structures – Part 3-2: Towers, masts and chimneys – Chimneys;
EN 1993-4-1: Eurocode 3 – Design of steel structures – Part 4-1: Silos;
EN 1993-4-2: Eurocode 3 – Design of steel structures – Part 4-2: Tanks;
EN 1993-4-3: Eurocode 3 – Design of steel structures – Part 4-3: Pipelines;
EN 1993-5: Eurocode 3 – Design of steel structures – Part 5: Piling;
EN 1993-6: Eurocode 3 – Design of steel structures – Part 6: Crane supporting structures.

Numerical values for partial factors and other reliability parameters are recommended as basic values that provide an acceptable level of reliability. They have been selected assuming that an appropriate level of workmanship and quality management applies.

2.1.2 United States Provisions for Steel Design

The main specification to apply for the design of steel structures in United States is ANSI/AISC 360-10 'Specification for Structural Steel Buildings' that addresses steel constructions as well as composite constructions: steel acting compositely with reinforced concrete. This specification states design requirements (stability and strength) for steel members and composite constructions, design of connections, fabrication and erection, Quality Control and Quality Assurance.

AISC 360-10 does not address minimum loads to be used: this topic is covered by ASCE/SEI 7-10 'Minimum Design Loads for Buildings and Other Structures' in the absence of an applicable specific local, regional or national building code.

AISC 360-10 does not cover seismic design: for seismic resistant structures the specifications to be applied are: ANSI/AISC 341-10 'Seismic Provisions for Structural Steel Buildings' and ANSI/AISC 358-10 'Prequalified Connections for Seismic Applications'. The first one gives additional rules for design and fabrication of steel structure to be used in seismic areas, the second one gives design methods for designing connections to be used in seismic resistant structures.

Finally, AISC 303-10 'Code of Standard Practice for Steel Buildings and Bridges' addresses design, purchase, fabrication and erection of structural steel.

Very useful tools for the designer are the AISC manuals: mainly the *AISC 325 Steel Construction Manual* and *AISC 327 Seismic Design Manual*, which discuss very interesting design examples to help in design activity.

2.2 Brief Introduction to Random Variables

All the variables involved in the design phase, both for determining resistance and stress distribution in cross-sections and members, are random in nature and not deterministic. As an example, with reference to the strength of materials, the imperfect homogeneity always present in samples for laboratory tests, as well as in structural *in-situ* elements, prevents association of an univocal value to resistance properties (such as, for example the yielding stress or the ultimate strength). Similarly, the set of internal forces and moments on structural members due to acting loads cannot be determined exactly because of the major sources of uncertainty and approximation involved in the parameters used for their definition.

Random variables are characterized by a number that expresses the probability (indicated as *prob* or p_r) of their occurrence. In the following, Y indicates the considered random variable (e.g. the measurement associated to a length, to a force, to weight or to the value of the force acting on a structure) and y represents the generic value assumed; the probability is identified by the term *prob* or p_r. The analytical treatment is based principally on the following two functions:

Relative Probability Density Function (PDF) (Figure 2.1), $f_Y(y)$, defined as:

$$f_Y(y)\mathrm{d}y = prob\{y < Y \le y + \mathrm{d}y\} = p_r\{y < Y \le y + \mathrm{d}y\} \tag{2.2}$$

Cumulate Density Function (CDF), $F_Y(y)$, defined as:

$$F_Y(y) = prob\{Y \le y\} = p_r\{Y \le y\} \tag{2.3}$$

The PDF describes the relative likelihood for this random variable to take on a given value. The probability of the random variable falling within a particular range of values is given by the integral of this variable density over that range that is given by the area under the density function but above the horizontal axis and between the lowest and greatest values of the range. The PDF is non-negative everywhere, and its integral over the entire space is equal to one.

$$\int_{-\infty}^{\infty} f_Y(y)\mathrm{d}y = 1 \tag{2.4}$$

The cumulative PDF (CDF) represents the probability that the random variable in question takes a value not exceeding y and is linked to the PDF from the integral relationship:

$$F_Y(y) = \int_{-\infty}^{y} f_Y(\mathrm{c})\mathrm{dc} \tag{2.5}$$

To better understand the correspondence between the functions PDF and CDF, it can be considered Figure 2.2. The area under the PDF function in the range between $-\infty$ and y_1, or equally between $-\infty$ and y_2, finds a corresponding value of the abscissa of the CDF, $F_Y(y_1)$ and $F_Y(y_2)$, respectively.

As an example of distribution of random variable, the density probability function of the weight per unit volume of both the concrete and the steel material are presented in Figure 2.3. Note that the curve of the concrete is extended on a portion of the abscissa axis appreciably wider than that

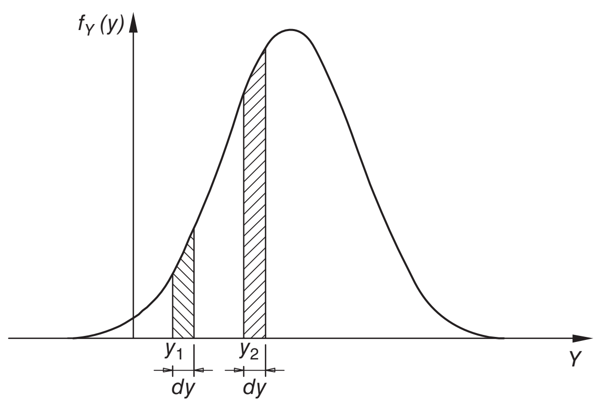


Figure 2.1 Example of the probability density function (PDF).

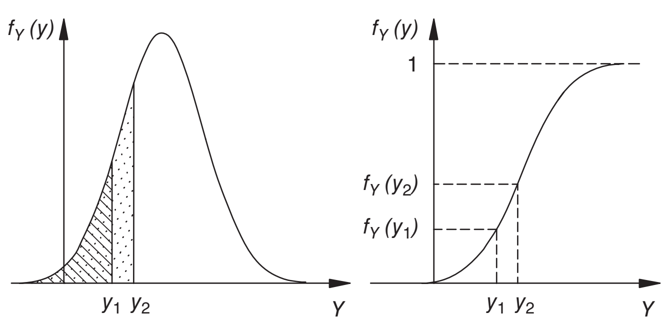


Figure 2.2 Probability density function and cumulate density function.

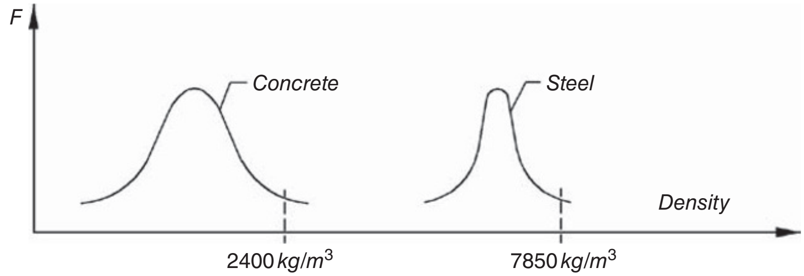


Figure 2.3 Probability density function for the weight per unit volume of the concrete and the steel.

corresponding to the steel, due to the relevant heterogeneity of the first material with respect to the latter.

As already mentioned, all the quantities interested in the check of Eq. (2.1) are random variables, and the data to be used in the design phase should be chosen based on reasonable probability values (or equally, acceptable risk levels) in relation to what they express. Actually, some of them (for example geometrical data or the eccentricity of loads) are, in most cases, taken as deterministic in order to simplify design. As discussed in the following, for the resistance, reference is made to low probabilistic values; that is to values with a high probability of being exceeded (95%). Otherwise, if actions are considered, high values (i.e. values with a reduced probability of being exceeded: 5%) are assumed for design.

In Figure 2.3 the values currently adopted for concrete and steel weight of the structural elements are indicated. (2400 kg/m^3 (149.83 lb/ft^3) to 7850 kg/m^3 (490.06 lb/ft^3) for concrete and steel, respectively). These values, like all the weights per unit of volume of both structural and non-structural elements usually correspond to the value that has a 95% probability of not being exceeded, or the 5% probability of being exceeded, and are defined 95% fractile or 95% characteristic values.

2.3 Measure of the Structural Reliability and Design Approaches

In the last century there has been a significant evolution of the philosophy of the structural reliability and, as a consequence, of the design methods. Nowadays, very sophisticated approaches are available, able to account for the variability of the main parameters governing design. It should be noted that these calculation methods currently in use are characterized by precision awareness. Because of the uncertainties of different type and nature that intervene in the design, the structure is always characterized by a well-defined level of risk. As a consequence, it is not possible to design a structure characterized by a zero level of failure probability: every structure has a probability of failure strictly depending by design, erection phase and maintenance during its use. To better understand this concept, reference can be made to Figure 2.4, where probability of failure and costs are measured by the abscissa and ordinate axis, respectively, of the considered reference systems.

If the initial cost of construction is considered (curve for the erection phase), also including design, a 100% safe structure is associated with infinite costs. The decrease in the cost of construction corresponds to an increase in failure probability. Furthermore, costs associated with repairing phases during the construction life have to be considered, when both moderate (curve b) and severe (curve c) damages could occur. It can be noted that costs associated with these damages increase with increasing failure probability. By adding the initial construction cost with the one of moderate or severe damages, the resulting curves (*d* and *e*, respectively) are characterized