Design of aluminium structures

Introduction to Eurocode 9 with worked examples

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Chapter 1 - Introduction

Aluminium is a material of choice for structural application, i.e. for parts contributing to the mechanical resistance and stability of buildings, constructions, engineering works and transport applications. Roofs for sport arenas, industrial halls, silos, bridges, trains, ships and oil platforms are just a few examples of where aluminium structures can be found.

To help consumers in getting reliable, consistent and improved product experience, European Aluminium is heavily involved in the development of standards at both international and European level. On top of more than 120 European standards for aluminium and its alloys in various forms, which have already been published by the European Committee for Standardization (CEN) with the support of the aluminium industry, many other standards offer solutions for the use of aluminium in various sectors.

When looking at structural design for aluminium, Eurocode 9: Design of aluminium structures (often abbreviated as EN 1999 or EC 9) describes the principles and requirements for the proper design of aluminium alloy structures. While in general standardisation supports customer's choices, Eurocode 9 is an excellent tool not only to facilitate customer's choices for aluminium products, but also serves as a promotion tool for using aluminium in structures.

This publication is a compendium of basic information on such aspects of aluminium including:

- The reasons to use aluminium for structural purposes
- The description of main aluminium alloys available for structural use
- The design of joints
- Worked examples on the application of Eurocode 9
- Sustainability of aluminium

Developed by European Aluminium with the contributions of Prof. Dr. Ing. Federico Mazzolani, Chairman of *CEN Technical Committee (TC) 250 Sub-Committee (SC) 9 on Eurocode 9: Design of aluminium structures*, Prof. Dr. Ing. Torsten Höglund (Convenor of the *TC 250 SC 9 Working Groups* for all Parts of Eurocode 9), Dipl. Ing. Reinhold Gitter and Dipl. Ing. Werner Mader (German representatives in *CEN TC 250 SC 9*), this document will be of particular interest to structural engineers designing infrastructures, means of transport, offshore constructions and, more generally, to anyone with an interest in the applications and development of aluminium for structural uses.

Chapter 2 - History of Aluminium Structures

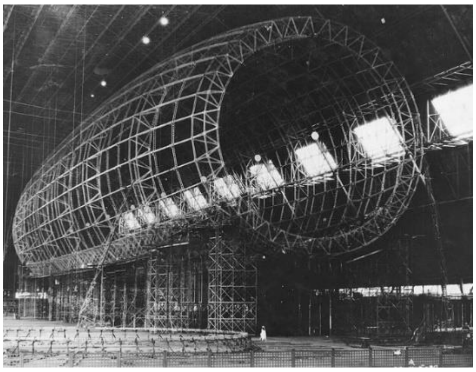
Almost three centuries have passed since 1722, when Friedrich Hoffman, professor at the University of Halle (Germany), announced the base of *alum* to be an individual substance. At the end of the eighteenth century the French chemist Guyton De Morveau suggested to call this substance *alumina,* which is derived from the Latin word *aluminis*, used in Egypt in the sixteenth century B.C. for indicating a material of dubious composition.

Following its isolation as an element and owing to the interest for the lightness and the brightness of such new metal, efforts were therefore made to produce it industrially. A first step in this direction was done during the 1850s by the French chemist Henry Sainte-Claire Deville, professor at the Sorbonne University in Paris. But it was only later, when Paul Louis Toussaint Héroult (1863-1914) in France and Charles Martin Hall (1863-1914) in the USA set-up at the same time the electrolytic process that paved the way for the industrial production of aluminium. As a result of the so-called Hall-Héroult process a 200 times cheaper price for Aluminium was achieved. That was fundamental to launch large-scale production of such material. In 1900 the industrial production of aluminium reached 8000 tons.

The properties of this new material impressed not only technicians, but also literates. Charles Dickens (1812-1870) wrote: *"Within the course of the last two years a treasure has been divined, unearthed and brought to light. What do you think of a metal as white as silver, as unalterable as gold, as easily melted as copper, as tough as iron, which is malleable, ductile, and with the singular quality of being lighter that glass? Such a metal does exist and that in considerable quantities on the surface of the globe. The advantages to be derived from a metal endowed with such qualities are easy to be understood. Its future place as a raw material in all sorts of industrial applications is undoubted, and we may expect soon to see it, in some shape or other, in the hands of the civilised world at large".*

Charles Dickens' prediction was correct: today aluminium is the second largest metal used worldwide and its production is higher than the one of all non-ferrous metals together. The idea of Jules Verne may be associated to the further applications in the modern aeronautic and aerospace industry.

The beginning of twentieth century assisted to the construction of extraordinary structures: the airships (Fig. 2.1 and 2.2). They consisted of a huge internal structure framework made of aluminium in the shape of a cigar, surrounded by a series of cells filled with helium gas. The most famous was the Zeppelin, however the first example was the Schwarz, built in 1897. The dirigible aluminium structure is composed by transversal rings connected by longitudinal beams; both have a reticulated structure: a modern design concept now applied for example large span roofing constructions.



Figures 2.1 and 2.2 - The most famous dirigibles

Some other striking examples are the statue of Eros in London's Piccadilly, 1893 (Figure 2.3), the aluminium sheets installed to clad the dome of the San Gioacchino church in Rome, 1911 (Figure 2.4), and the aluminium components installed in New York's Empire State Building, 1931, the first building to use anodised aluminium.

More recently in 1958, an extraordinary structure was built for the Universal Exhibition of Brussels and clad with aluminium: the Atomium, a 102 meter tall structure, which is half way between sculpture and architecture, symbolising a ferrite crystal made of 9 iron atoms, magnified 165 billion times.

Figure 2.3 - The Eros statue in Piccadilly circus, London

The aluminium cladding - initially conceived to last six months – has served its purpose for almost 50 years. After the Atomium was undergoing renovation (2004-2007): the original aluminium skin was used for new purposes. A thousand aluminium triangular panels are available for sale with a certificate of authenticity for collectors and Atomium fans. The remaining 30 tonnes of aluminium have been made available for recycling.

Today aluminium is the material of choice in many fields, ranging from packaging, furniture, window, cladding of the buildings, to automotive, airplane and offshore industry as well as to bridges and civil engineering structures. In many of these fields, aluminium is out of competition: one cannot fly, enjoy high speed trains, high performance cars or fast ferries without it.

Figure 2.4 - San Gioacchino’s church, in Rome

Figure 2.5 - The world's largest aluminium boat, 130m long and 32m wide

Figures 2.6 and 2.7 - Production of aluminium train wagons

Chapter 3 - Industrial production Process

Engineers responsible for the design of an aluminium structure are faced with two peculiarities. The first is the large number of aluminium *alloys* combined with the different available *tempers*. The second is the fact that either as sheet or as a standard section, only a limited range of alloys are available from stock. The reason for this is, while steel structural sections are usually manufactured through rolling processes, aluminium sections for structural purposes are often produced by means of hot extrusion. Rolling is characterised by high roll die costs in combination with considerable changeover times and therefore needs large production quantities to keep costs as low as possible. When looking at aluminium extrusions, die costs are low for small sections and increase only moderately for larger shapes. The quantities to produce aluminium sections in a cost-effective way are relatively small and lie between 200 kg and 3000 kg depending on the size of the section. Consequently, many engineers and companies design specific sections for their own projects in order to ensure high functionality. Ninety percent of all sections produced by aluminium extruders are individually designed and are therefore only available for the use by the designer/purchaser of the specific section.

This may explain why designing aluminium structures requires a deep knowledge of the material, especially when compared to the design of steel structures. This chapter will investigate the properties of the most important alloys, the system of designations of alloys and tempers and will dig into the availability of semi-finished products and their relative costs.

3.1 Hardening of aluminium

Pure aluminium itself is a metal with relatively low strength. Aluminium in its purest form has a *tensile strength* of around 40 N/mm² and a *yield strength* of about 10 N/mm². Aluminium alloys however have been developed with mechanical properties beyond those of the base material.

A very efficient means of producing material with greater resistance is to introduce suitable foreign elements into the aluminium matrix (alloying). By introducing suitable foreign metals into the aluminium matrix it is possible to produce lattice imperfections that allow better mechanical performance of the material. One of the elements which best suits the requirement to improve strength is magnesium (see Figure 3.1). Aluminium-magnesium alloys were indeed the predominant choice for structural aluminium applications 100 years ago and many years later.

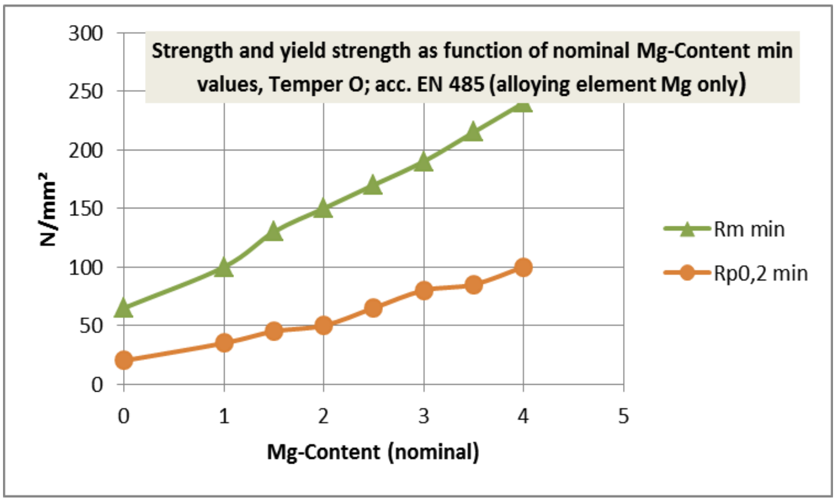


Figure 3.1: Hardening effect as a function of the content of alloying element, here Mg

3.2 Strain hardening

Plastic deformation produces imperfections in the lattice by markedly increasing the number of so-called *dislocations* particularly along the slip-planes. With increasing load and deformation, additional slip planes continuously develop so that, with the resulting increase in dislocation density, the material increases its mechanical strength. Parallel to this increase in strength, ductility decreases until ultimately the deformation process has to be stopped. When cold rolling, this so called "*work hardening*" or "strain hardening" continues until the material begins to develop cracks. However, the hardening process can be reversed using heat. Depending on temperature and time, the gain in material strength can be reversed and brought back to its starting level before cold working. The material also recovers its original ductility. This thermal process is known as "annealing". From this soft state the cold working processes can be restarted. Fig. 3.2 shows the effect of cold working and annealing on strength, here as a function of time at constant temperature.

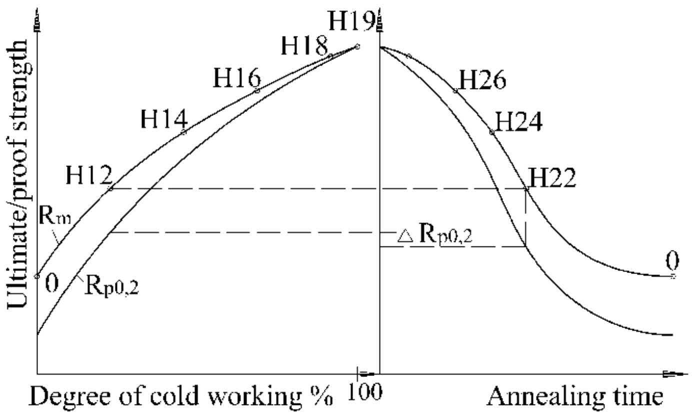


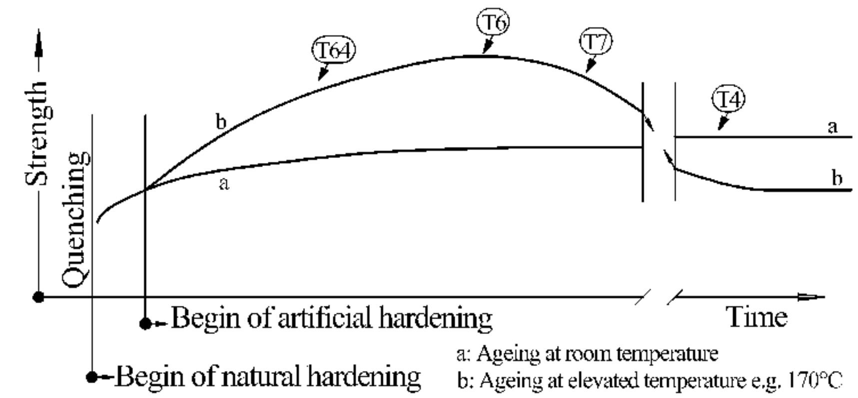
Figure 3.2: Work hardening and annealing

3.3 Precipitation hardening

One or more suitable elements can form intermetallic compounds, i.e. particles bonded together with the aluminium matrix. As seen before, such elements also constitute lattice imperfections and, depending on the size of these particles as well as on their distribution, are responsible for considerable increase in strength. The whole precipitation hardening process begins with the alloy being heated above a reference temperature and saturated there until a homogeneous (solid) solution is produced. Then, *quenching* is necessary to get a uniform distribution of all elements also at ambient temperature. After that, *ageing* ensures that the elements involved begin to diffuse in the aluminium matrix, while numerous nucleation sites and precipitates grow and form intermetallic compounds.

3.4 Artificial ageing

Natural ageing begins immediately after quenching at a relatively high speed but degressively and then asymptotically approaches an upper limit (T4 in Figure 3.3). Depending on the alloy, the ageing process might take weeks, but for most alloys this can be considered to be already concluded after one week. Artificial ageing can start hours (but also days, depending on manufacturing needs) later. The material to be artificially aged is placed in a furnace, which allows the ageing to be carried out under different temperature conditions. Typical for all temperatures is a quick hardening that progressively reaches a maximum (T6). If the material is exposed to high temperatures for a longer time, the effect of the precipitations on strength decreases and we get an over-aged temper (T7). In general, over-aged tempers are characterised by better ductility, corrosion resistance (some copper and/or zinc containing alloys) and better electrical conductivity.

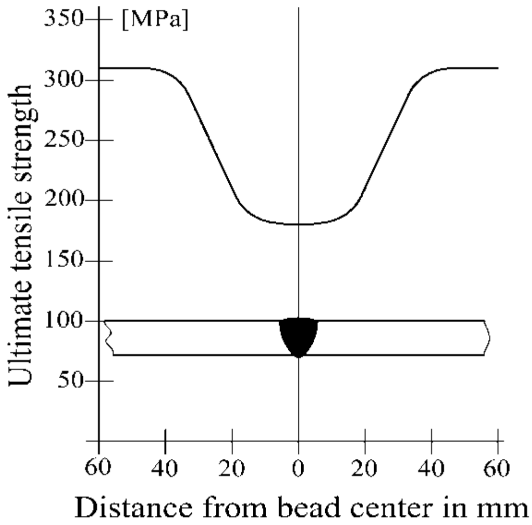


It must be emphasised that today precipitation hardening alloys are dominating in many areas (e.g. extruded sections). They present a significant lower deformation resistance during the (warm) working processes, while gaining their often-remarkable strength on a later stage through precipitation hardening.

Figure 3.3: Strength in function of time at ambient and elevated temperature

3.5 Influence of Heat

The strength of aluminium, similarly to other materials, decreases when temperature increases. Up to certain temperatures this phenomenon is still reversible, i.e. after cooling down the material has the same properties as before. With temperatures up to 80 degrees Celsius, the drop in strength is negligible for all alloys and tempers. Over 80 °C some design situations could require creep effects to be considered. Heat-treatable alloys begin to lose strength with temperatures over 110 °C depending on time. Non-heat-treatable alloys in work hardened tempers begin to lose strength with temperatures over 150 °C - also depending on time. In 'O temper' non-heat-treatable alloys, no permanent loss in strength occurs.



Welding causes much more severe losses in the strength of the material. In case of welding, the temperatures are so high that the effects of a decrease in strength in the vicinity of the weld (the so called Heat Affected Zone HAZ) must be taken into account, as this often constitutes an important aspect of the verification of the design of a structure. The heat-treatable alloys in temper T6 (Fig 3.3) have a loss of approximately 40% of their strength with the single exception of the alloy EN AW-7020, which loses only 20% of its initial strength. To facilitate design calculations, the area of strength losses is replaced by a rectangular area with the width b_{haz}, which is standardised and may depend on the material thickness in a range of some ten millimetres. The strength value in this zone is also standardised and depends on alloy and temper.

Figure 3.4: Reduction of strength in the heat affected zone (HAZ) (typical for EN AW-6082)

3.6 Alloys

In practice, only a few elements have proven to be suitable as alloying additions in aluminium wrought and cast materials for structural applications. These are:

Copper (Cu); Manganese (Mn); Silicon (Si); Magnesium (Mg); Zinc (Zn)

They can be used as single elements as well as in combination. When working with aluminium alloys, it is necessary to know the nomenclature used with this material. This refers to the designation of the alloys in use and to the temper states in which they are supplied to the market.

3.6.1 Designation of wrought alloys

The numerical system developed by The Aluminium Association in the USA is today the most recognised system to design wrought alloys worldwide. European Aluminium is among the signatories to the Declaration of Accord on an International Alloy Designation System for Wrought Aluminium and Wrought Aluminium Alloys[1].

Consequently, European standards also follow this nomenclature which makes use of a 4-digit number to further designate the alloy. Wrought alloys are designated with the prefix "EN AW-". The first digit gives basic information about the principal alloying element(s):

2xxx= Cu; 3xxx=Mn; 4xxx=Si; 5xxx=Mg; 6xxx=Mg+Silicon; 7xxx=Zn

The designation system also characterizes the hardening of the alloys belonging to a family. The 1xxx, 3xxx and 5xxx series are so called non-*heat-treatable* alloys; they gain their strength by alloying (e.g. increasing content of Mg) and work hardening. The 2xxx, 6xxx and 7xxx series are heat-treatable alloys, which gain their strength by alloying but make use of precipitation hardening as the main mechanism.

In the past, several national standards used to designate alloys based on chemical symbols. For the engineer who was not familiar with aluminium alloys, this was an advantage as it made possible to identify the characteristics of a given alloy more easily (see Table 3.1 below).

Non heat-treatable alloys					**Heat treatable alloys** (Precipitation hardening alloys)				
Numerical Desig nation	**Chemical Desig nation**	**Form of Products**			**Numerical desig nation**	**Chemical Desig nation**	**Form of products**		
EN AW-	**EN AW-**	**sheet**	**extru sions**	**forging**	**EN AW-**	**EN AW-**	**sheet**	**extru sions**	**forging**
3004	AlMn1Mg1	**X**			6060	AlMgSi		**X**	
3005	AlMn1Mg0,5	**X**			6061	AlMg1SiCu	**X**	**X**	
3103	AlMn1	**X**			6063	AlMg0,7Si		**X**	

[1] https://www.aluminum.org/sites/default/files/Teal%20Sheet.pdf

5005/5005A	AlMg1(B)/(C)	X			6005A	AlSiMg(A)		X	
5049	AlMg2Mn0,8	X			6082	AlSi1MgMn	X	X	X
5052	AlMg2,5	X			6106	AlMgSiMn		X	
5083	AlMg4,5Mn0,7	X	X	X	7020	AlZn4,5Mg1	X	X	
5454	AlMg3Mn	X	X						
5754	AlMg3	X	X	X					
8011A	AlFeSi	X							

Table 3.1. Wrought alloys listed in EN 1999-1-1

This is the reason why the European aluminium standards still use two principles for aluminium alloy designation: numerical system specified in EN 573-1 and chemical symbols system specified in EN 573-2. Both systems are used in EN 573-3 that provides the detailed chemical composition and form of products of wrought aluminium alloys.

3.6.2 Designation of casting alloys

The elements used for casting alloys are basically the same as for wrought alloys. However, for castings different compositions are preferred. Casters prefer type 4xxxx alloys with high silicon content, since it ensures a good quality production. Casting alloy designations have the prefix "EN AC-" to distinguish them from wrought alloys and have 5 digits in total. This system is European and not used in the USA.

Non heat treatable alloys		**Heat treatable alloys** (Precipitation hardening alloys)	
Numerical designation	Chemical designation	Numerical designation	Chemical designation
EN AC-	**EN AC-**	**EN AW-**	**EN AW-**
51300	AlMg5	42100	AlSi7Mg0,3
44200	AlSi12(a)	42200	AlSi7Mg0,6
		43000	AlSi10Mg(a)
		43300	AlSi9Mg

Table 3.2. Casting alloys listed in EN 1999-1-1

The most frequently used alloys are EN AC-42100, -43000 and -44200. The alloys preferred by the foundries are EN AC-43000, -43300 and -44200 due to their good castability. The alloy EN AC-51000 (AlMg5) is difficult to cast and therefore not popular at foundries and is therefore used quite occasionally despite the fact that engineers like to make use of it due to its bright surface and anodisability (other alloys are more or less greyish, especially when anodised).

For more details, please refer to EN 1780-1, EN 1780-2 and EN 1780-3.

3.7. Tempers and designation of tempers

3.7.1 Tempers of non-heat-treatable alloys

Figure 3.2 explains the system in use for the definition of tempers for non-heat-treatable alloys. By increasing the degree of cold working, ultimate strength and proof strength also increase. Nevertheless, there is a limit. For example for temper H18 - fully hardened or 4/4 hard. The cold working process can be stopped earlier to get the tempers strain hardened between O and H18, e.g. H14 or ½ hard. However, it is also possible to produce tempers with lower strength than 4/4 hard, by annealing fully hardened material, i.e. H18. In this case, the material will be characterised by lower strength values with a modified designation e.g. H24, i.e. "Strain hardened and partially annealed, 1/2 hard". In the system is so that e.g. H14 and H24 have the same strength value, but the proof strength of H24 is a little lower (see Figure 3.2), but its *formability* is better. Some more differentiations for the designation of tempers exist.

For the most relevant for non-heat-treatable alloys, see tables 3.1 and 3.2. For other temper please refer to the standard EN 515.

Symbol	**Description**
O	Annealed (soft)
H 111	Annealed and slightly strain-hardened (less than H11) during subsequent operations such as stretching or levelling
H12	Strain-hardened, 1/4 hard
H22	Strain-hardened and partially annealed, 1/4 hard
H32	Strain-hardened and stabilized, 1/4 hard
H42	Strain-hardened and painted or lacquered -1/4 hard
H14	Strain-hardened, 1/2 hard
H18	Strain-hardened, 4/4 hard (fully hardened)

Table 3.3: Tempers in use for structural application of work hardened semi-products (typical examples to explain the system)

3.7.2 Tempers of heat-treatable alloys

The complete heat-treatment consists of a solution heat-treatment, a quenching process and subsequent ageing, where the actual hardening occurs. It must be said that, unlike steel, aluminium alloys are not hard immediately after quenching.

To get the highest strength values it is important to keep the material at the correct solution heat temperature for enough time and to follow the correct quenching procedure (see Figure 3.3 above). Depending on the alloy, this may be carried out using water or air. Heat-treatable alloys are produced in many tempers. For structural engineering only a limited number is important and listed in Table 3.4.

Symbol	Description
T4	Solution heat-treated and then naturally aged
T5	Cooled from an elevated temperature shaping process and then artificially aged
T6	Solution heat-treated and then artificially aged
T61 T64	Solution heat-treated and then artificially aged in under ageing conditions in order to improve formability (T64 between T61 and T6)
T66	Solution heat-treated and then artificially aged – mechanical property level higher than T6 achieved through special control of the process 6000 series alloys
T7	Solution heat-treated and artificially over-aged
Tx51 Tx510 Tx511	These suffixes stand for a controlled stretching to relieve internal stresses coming from manufacturing (the fourth digit characterises only variants – no influence on characteristic values!)

Table 3.4: The main tempers in use for structural application of precipitation hardened semi-products (T7 only listed to explain the system)

3.8 Alloys and tempers of alloys listed

Considering their applicability for structural application and availability on the market, only a limited number of alloys are included in the Eurocode 9. This is also true for the tempers of the alloys. Therefore, only the tempers which were most frequently used in the past are listed, i.e. tempers H12, H14 and their corresponding partially annealed tempers for work hardened materials. Higher strength tempers such as H16 and H18 are less common, since good forming behaviour is often desired.

In EN 1999-1-1 (Part -1-1 of Eurocode 9), 17 wrought alloys are listed. They are used for products that have been circulating for a long time on the European market and that have been approved by several national authorities.

EN 1999-1-1 offers a wide range of alloys and tempers to be used for structural applications. The range of strength, in the sense of proof strength, varies from EN AW-5005 O with 35 N/mm² up to EN AW-7020 T6 with 290 N/mm².

For structural engineering the most commonly used alloys are:

- EN AW-6082, EN AW-6061 and EN AW-7020 (less frequently) for structures and components from sheet and extrusions of the same alloy
- EN AW-5083 and EN AW-5754 for structures and components from sheet (also in structures mixed with

sections of other alloys)
- EN AW-6060 and EN AW-6063 for structures and components from extrusions (also in structures mixed with sheet of other alloys)

3.9 Practical viewpoints for the selection of materials

3.9.1 Sheet and plate

When designing aluminium structures, one must consider that sheets in small and mid-sized formats up to 1500x3000 mm are easy to obtain but the availability of more complex alloys and tempers is limited. EN AW-5083, -5754 and -6082 are commonly available. Some specialised shipyard suppliers may also have larger dimensions on stock, but in this case delivery time must be carefully taken into account. Sheets of non-commonly used aluminium alloys need significant amount of orders with quantities of around 30-50 tons.

For structural engineering projects, it is often very important to know what the geometric limits are for the existing production facilities of semi-products. Sheets and plates can be produced with widths of more than 3 m and lengths of up to 22 m. The exact limits may depend on thickness and alloy. Most of the manufacturers deal with lengths under 10 m and widths up to approximately 2 m. When designing in sheets, it is also important to know that folding presses with working widths up to 16m are not very common although facilities with more than 20 m do exist.

3.9.2 Extrusions

For aluminium extrusions, die costs are modest. Die changing needs only short times and therefore production batches can be ordered for smaller quantities, normally between 200 and 3000 kg. This makes it possible for engineers to design special sections, optimally adapting their requirements to the needs of each structural application. The advantages are remarkable: reduced costs, low weight, transport facility, functional structural sections.

These specificities give tremendous advantages to extruded aluminium. In any case, it is recommended to check the stock availability of semi-finished products whenever starting with the design of aluminium for structural applications.

In principle, extrusion works like squeezing paste out of a tube (Figure 3.5), a process the aluminium industry is accustomed to performing on a daily basis all over the world by means of an extrusion press (Figure 3.6). Here, a preheated aluminium *billet* (400 to 550 °C, depending on the alloy) is positioned in a preheated container. Under the forces of the stem the material begins to flow through the die and so acquires the form defined by the die. While aluminium is an extremely good material for extrusion, steel cannot be extruded.

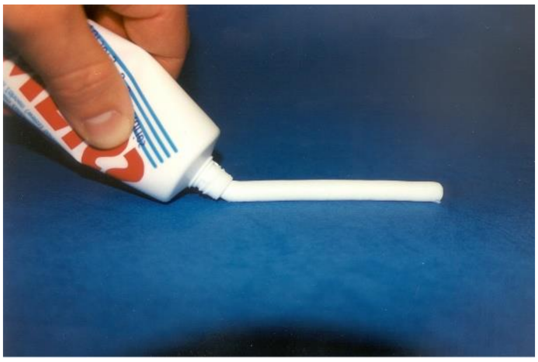


Figure 3.5: Extruding toothpaste

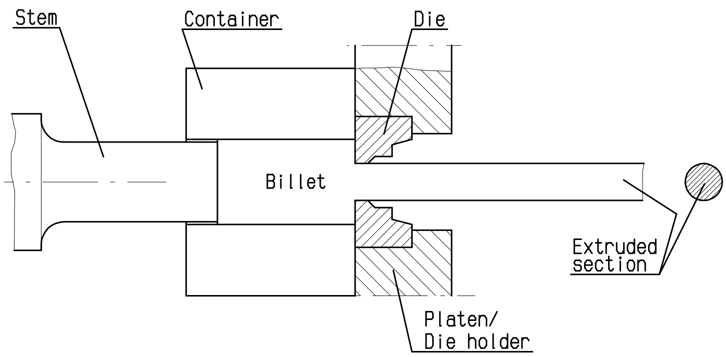


Figure 3.6: Extruding an aluminium bar

3.9.3 Castings and forgings

Cast and forged parts are always individually designed parts and ordered directly from the manufacturer (if they are not part of a system distributed via stockists). If no special experience exists, it is recommended that the engineer contacts and collaborates closely with the manufacturer to determine the best design in combination with the alloy.

Depending upon alloy, foundry and size of the casting, the usual minimum quantity is in the order of 500-1000 pieces but in special circumstances it may be possible to obtain somewhat smaller amounts. Sand cast parts are possible in much lower quantities, depending on size and alloy. Production lots of 10 or 50 are not unusual. The procurement of the casting alloys listed in EN 1999-1-1 is no problem for the foundry, and small quantities can be supplied.

A similar situation applies to forged parts. In general, a batch of 1000 workpieces is required for an economically viable manufacturing process Some manufacturers may produce smaller quantities, in case the customer accepts the typically higher costs associated. However, it may be a problem for the forging shop to get the pre-material from the semi-product manufacturer in the form or quantity needed. The alloys listed in EN 1999-1-1 are all commonly used.

Figure 3.7 gives an idea of extruded aluminium sections used for structural purposes. For the engineer not familiar with the design, it is recommended to get in contact with a manufacturer and get advice about suitable alloys to be chosen, possible tolerances of the cross section, straightness etc. (e.g. based on EN 755, EN 12020)

For structural engineering, it may also be important to know what the geometric limits are for the existing production facilities for the semi-products. Figure 3.8 shows the limits for production in Europe. Depending on cross section and alloy, profiles can be produced with lengths up to 30 m. Normal stock length is 6 m.

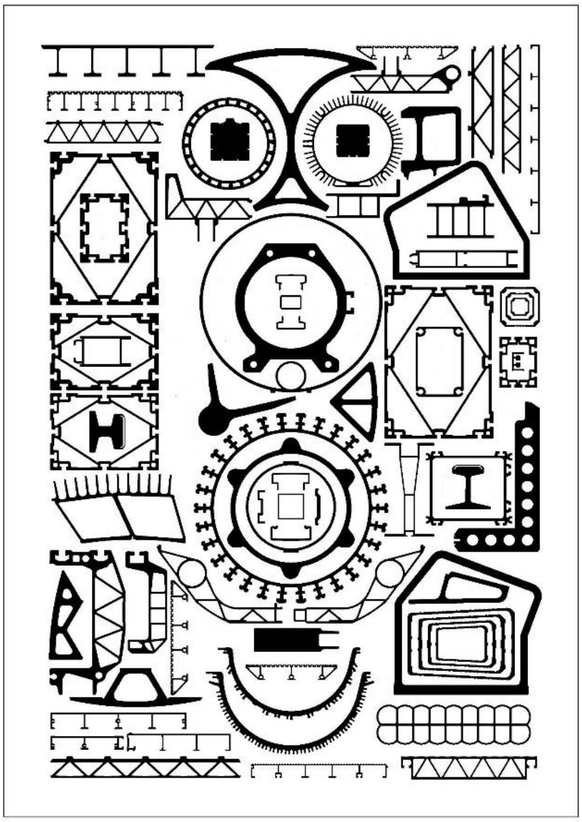


Figure 3.7: Multiple possibility for sections

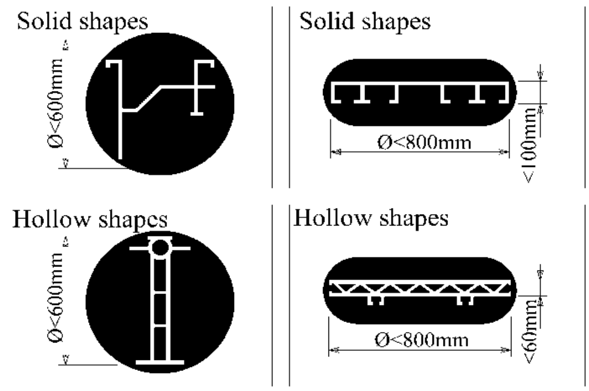
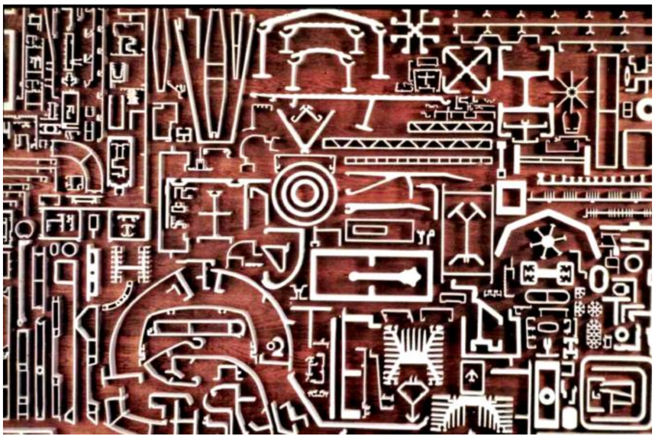


Figure 3.8: Upper dimensional limits for the design of extrusions

Chapter 4 - Why Aluminium in Structural Engineering

4.1 Basic prerequisites

The main difference between aluminium and steel is clearly emphasized by the comparison between the typical products which characterize these materials (Figures 4.1 and 4.2). Steel hot rolled sections look strong, heavy and rough; contrary aluminium extruded profiles which are various, light and elegant.



Figures 4.1 and 4.2: Typical steel and aluminium profiles

The success of aluminium alloys as constructional material and thus being a possible competitor to steel are based on some prerequisites, connected to the physical properties, the production process and the technological features. It is commonly recognised that aluminium alloys can be economical, and therefore competitive, in those applications where full advantage is taken of the following properties:

A. *Lightness*: due to the low specific weight of aluminium alloys, which is one third of steel, it is possible to:
   - simplify the assembly phases;
   - transport fully prefabricated components;
   - reduce the loads transmitted to foundations;
   - economize energy either during assembly and/or in service;
   - reduce the physical labour.

Significant examples of the advantages of this property are given in Figures 4.3 and 4.4, which show a floating dock and a bridge span respectively moved by a crane and by a truck.

Figures 4.3 and 4.4: Examples of transportation of fully prefabricated structures.

B. *Corrosion resistance:* due to the formation of a protective oxide film on the surface, it is possible to:
- reduce the maintenance costs;
- provide good performance in corrosive environments.

C. *Functionality* of structural shapes, due to the extrusion process, makes it possible to:

- optimise the geometrical properties of the cross-section by designing a shape which simultaneously gives the minimum weight and the highest structural efficiency;
- obtain stiffened shapes without using built-up sections, thus avoiding welding or bolting;
- simplify connecting systems among different component, thus improving joint details;
- combine different functions of the structural component, thus achieving a more economical and rational profile.

Examples of extruded aluminium profiles are shown in Figure 4.5. It can be observed that the double sections can be adapted introducing bulbs and stiffeners to reduce local buckling effects. However, also by introducing a rail in the middle of flanges for functional reasons. The T sections can also be improved by bulbs and stiffeners for strengthening purpose, but the web can be doubled for improving the connecting system. Other sections, like L, Y and C are designed with bulbs and stiffeners, which avoid local buckling.

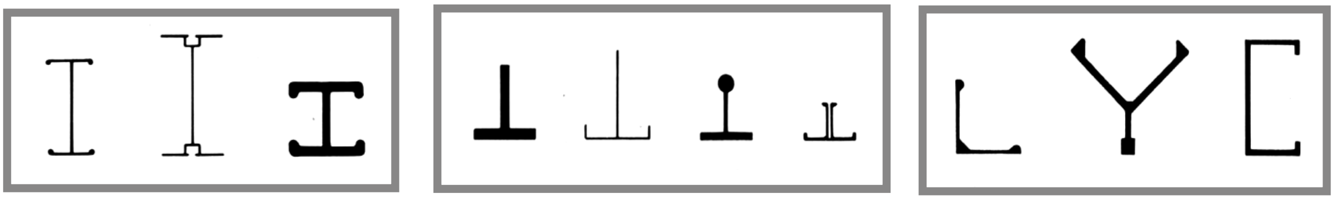


Figure 4.5: Examples of extruded shapes

Figure 4.6 shows the nodal details of a crane bridge made of aluminium; each node is designed in such a way to improve the connection with the bars. Node 1 is an upside-down T section which web is bifurcated to accommodate the diagonal bars. The flange supports the two rails Node 2 is the tubular section of the upper chord with two expansions for connecting the transversal bars. Node 3 is a stiffened C section with three expansions to connect both, the bottom and the transversal bars, while the rail runs on the top flange.

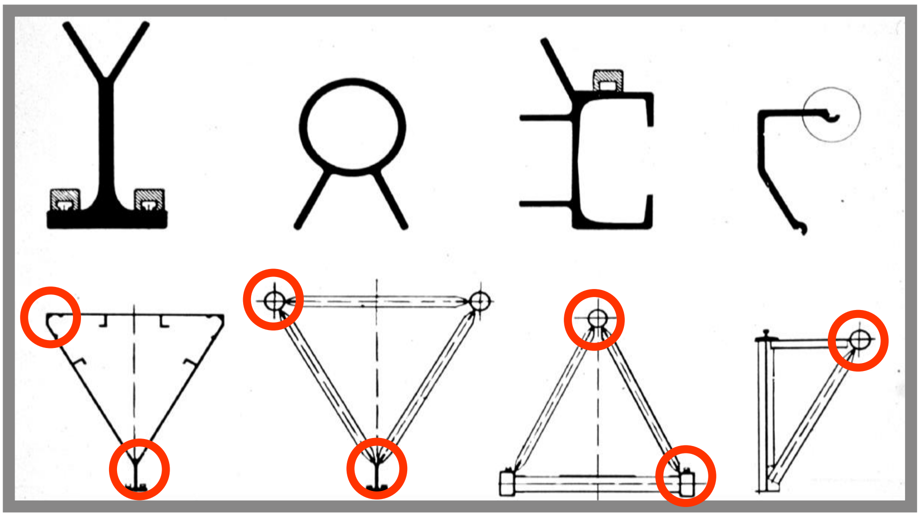


Figure 4.6: Nodal details of a crane bridge

4.2 Fields of application

Some typical cases where aluminium can compete with shall be mentioned. They highlight the benefits of using aluminium compared to steel, especially because of its main basic properties: lightness, corrosion resistance and functionality. Used in civil engineering, aluminium can be successfully used for:

a) Long-span roof systems in which live loads are small compared with dead loads, as in the case of lattice space structures and geodetic domes (Figures 4.7 and 4.8), covering large span areas, like halls, auditoriums.
b) Structures located in inaccessible places far from the fabrication shop, for which transport costs and ease of erection are of extreme importance, such as electrical transmission towers, which can be carried by helicopter already assembled (Figure 4.9).
c) Structures situated in corrosive or humid environments such as swimming pool roofs, river bridges, hydraulic structures and offshore super-structures (Figure 4.10).
d) Structures having moving parts, such as sewage plant crane bridges and moving bridges, where lightness means economy of power under service (Figure 4.11).
e) Structures for special purposes, for which maintenance operations are particularly difficult and must be limited, as in case of masts, lighting towers, antennas, tower signs, motorway signs, etc.

In general, the same main pre-requisites are fruitfully exploited in all kinds of applications in “Structural Engineering”. Table 4.1 illustrates a series of structural applications which are grouped according to the three basic properties. **L** for lightness; **C** for corrosion resistance; **F** for functionality. The seven lists correspond to the main influence of one property (**L, C, F** separate) or to the combination of two (**C+F, C+L, F+L**) or three (**C+L+F**).

All together, they represent the choices in which the use of aluminium is potentially advantageous and, therefore, competitive with steel.

Figure 4.7: ENEL aluminium dome, Rome (Italy)

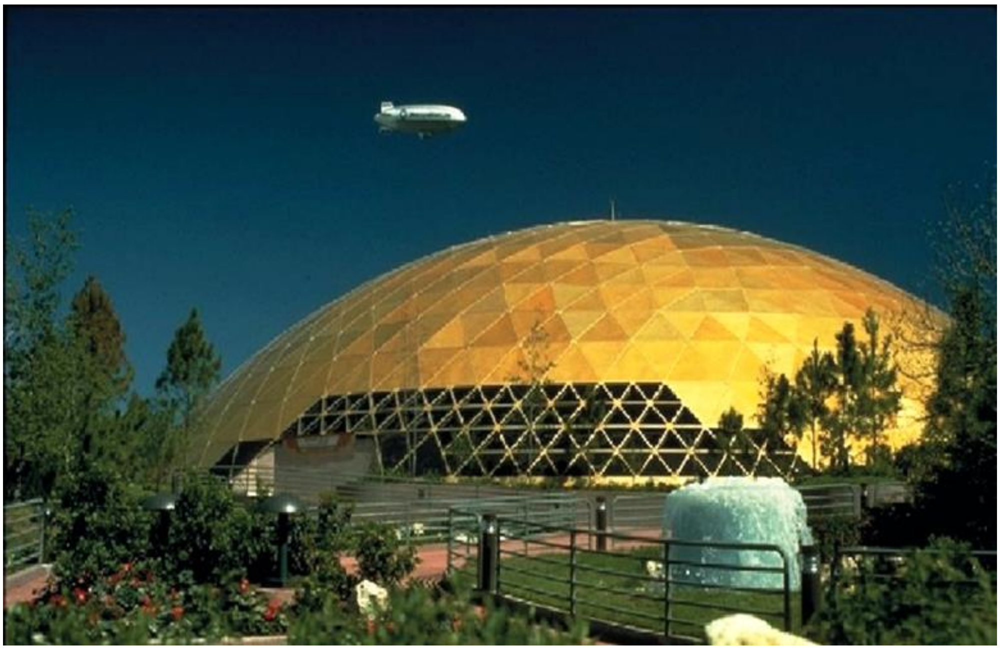


Figure 4.8: A typical geodetic dome (Epcot Centre, Florida)

Figure 4.9: A prefabricated bridge transported by helicopter

Figure 4.10: The superstructures of an offshore platform

Figure 4.11: Aluminium bridge with maintenance-free movable deck, Amsterdam (the Netherlands)

<table>
<tr>
<td>C
• Storage vessels
• Lamp columns
• Profiled roof and wall cladding
• Support for railway overhead electrification
• Enclosure structues for sewage works
• Sound barriers
• Vehicle restraint systems
• Sewage plant bridges
• Silos
• Traffic signal gantries
• Traffic signal poles*</td>
<td>C+L
• Lighting control towers
• Flag poles
• Aircraft access bridges
• Transmission towers
• Bridge inspection gantries
• Offshore structures (living quarters. bridges)
• Tank flotation covers</td>
<td>L
• Crane booms
• Lorry mounted cranes
• Pit props
• Bridges
• Mobile bridge inspection gantries
• Scaffolding systems
• Ladders
• Cherry pickers
• Telescopic platforms
• Masts for tents</td>
</tr>
<tr>
<td rowspan="2">C+F
• Domes over sewage tanks
• Marina landing stages
• Roof access staging
• Dam logs
• Curtain walling
• Overcladding support systems
• Pedestrian parapets
• Chicken house structures
• Wood drying kilns
• Space structures (domes, reticular, lattice, etc.)
• Exhibition stands
• Swimming pool fossa
• Canopies
• Bus shelters
• Green houses/Glass houses</td>
<td>C+F+L
• Grating planks
• Helideck</td>
<td rowspan="2">F+L
• Access ramps
• Support for shuttering
• Track ways (temporary)
• Elevators for building materials
• Scaffold planks
• Trench supports
• Grave digging supports
• Loading ramps
• Landing mats for aircraft
• Access gangways
• Shuttering support beams
• Military bridges
• Radio masts
• Shuttering
• Telescopic conveyor belt structures
• Grandstand structures (temporary)
• Building maintenance gantries
• Fabric structure frames</td>
</tr>
<tr>
<td>F
• Prefabricated balconies
• Conveyor belt structures
• Monorails
• Robot support structures
• Shuttering form work
• Tunnel shuttering</td>
</tr>
</table>

Table 4.1: Structural applications grouped according to the prevalence of the three basic properties lightness (L), corrosion resistance (C), functionality (F)

Chapter 5 - Design of aluminium structures according to Eurocode

5.1 Introduction

The current version (2007, as amended in 2009) of Eurocode 9: Design of aluminium structures is composed of five documents:

- *Part 1-1: General structural rules*

- *Part 1-2: Structural fire design*

- *Part 1-3: Structures susceptible to fatigue*

- *Part 1-4: Cold-formed structures*

- *Part 1-5: Shell structures*

Part 1-1 General structural rules

Contrary to the other Eurocodes, EC9 consists of just one part, which is divided into one basic document "General structural rules" and four specific documents, which are related to the basic one. There are no separate documents dealing with specific types of structures, like in steel (i.e. bridges, towers, tanks …). Eurocode 9 looks instead at general items which are applicable not only to the range of the so-called "Civil Engineering", but more widely to any kind of structural applications in the wider field of "Structural Engineering", including also the transportation and offshore industries. Some rules for bridges and lattice spatial roof structures are given in annexes to Part 1-1.

The main features of the calculation methods in Part 1-1 are given in the following sections.

Part 1-2: Structural fire design

As in all Eurocodes, Part 1-2 is devoted to fire design. For aluminium structures, fire design has been codified for the first time according to general rules, which assess the fire resistance based on three criteria: Resistance (R), Insulation (I) and Integrity (E).

Aluminium alloys are generally less resistant to high temperatures than steel and reinforced concrete. Nevertheless, by introducing rational risk assessment methods, the analysis of a fire scenario may in some cases result in a more beneficial time-temperature relationship and thus make aluminium more competitive and the thermal properties of aluminium alloys (e.g. high thermal conductivity) may have a beneficial effect on the temperature development in the structural component.

Part 1-3: Structures susceptible to fatigue

The knowledge on the fatigue behaviour of aluminium joints has been consolidated during the last 30 years. In 1992, the European Recommendations on Fatigue Design of Aluminium Alloy Structures were published, representing a fundamental basis for the development of EC9. In its Part 1-3: Structures susceptible to fatigue, it is possible to find general rules applicable to all kind of structures under fatigue loading conditions with respect to the limit state of fatigue induced fracture. Three design methods have been introduced:

– Safe life design;
– Damage tolerant design;
– Design assisted by testing.

Eight basic groups of detail categories have been considered, including:

- non-welded details;
- members with transverse and longitudinal welded joints and attachments;
- bolted joints;
- special aluminium mechanically joints such as bolt channel joints and screw groove joints;
- adhesively bonded joints.

The use of finite elements and the guidance on assessment by fracture mechanism have been suggested for stress analysis. The importance of quality control on welding has been particularly emphasised in general, and specific reference to EN 1090 "Execution of steel and aluminium structures" has been taken into consideration (see also Chapter 7).

Part 1-4: Cold-formed structures

Part 1-4 is mainly referred to the use of trapezoidal sheeting. It is similar as the corresponding steel part EC3-1-3, but the effective thickness method is used instead of the effective width one.

Part 1-5: Shell structures

Part 1-5 has been built-up by following the same format of the similar document EC3-1-6, but the calculation methods are based on appropriate buckling curves which are obtained on the bases of the experimental evidence on aluminium shells.

5.2. Main aspects to consider when designing aluminium structures

This is a summary of the main aspects to be considered when designing aluminium structures, as they are very different from steel.

Weight

Low weight (g = 2 700 kg/m^3) is particularly important where the weight of the structure is a dominant factor and where the low weight permits a vehicle to carry a greater load. Low weight is also important during the transport and assembly of a structure. Examples of weight savings are given in Tables 5.1 and 5.2.

Deflection

As the modulus of elasticity of aluminium is low (E = 70 000 MPa), the deflection check is a priority issue. Deflection requirements are often critical in aluminium structures, for example when the deflection should not be greater than four-hundredths of the span of a beam. Often the strength is not fully exploited and simple approximations are adequate for the check of the resistance. A simplified method to allow for local buckling and softening in the *Heat Affected Zone (HAZ)* is given in EN 1999-1-1, 6.4.

If the deflection of a beam in bending is the critical factor, the stiffness EI must be the same as in steel. Because the modulus of elasticity E of aluminium is one third that of steel, the moment of inertia I_{al} for the aluminium beam must be $3I_{\text{steel}}$. If the height of the beam is not increased, the flange area must be increased by a factor of three and, because the density of aluminium is one third of that of steel, this means that the aluminium beam will have the same weight as the steel beam. If, on the other hand, it is acceptable to increase the height of the beam, a considerable weight saving can be made by choosing aluminium. This is illustrated in Table 5.2, where an IPE 240 steel beam is compared with some alternative aluminium beams.

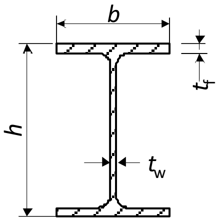
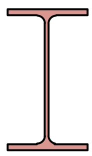
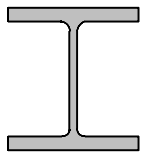
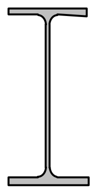
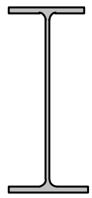
				
Material	steel	aluminium	aluminium	aluminium
Moment of inertia I/mm^4	$38.9 \cdot 10^6$	$116.6 \cdot 10^6$	$116.7 \cdot 10^6$	$117.3 \cdot 10^6$
EI/MNm^2	8,17	8,17	8,17	8,17
h/mm	240	240	300	330
b/mm	120	240	200	200
t_w/mm	6.2	12	6	6
t_f/mm	9.8	18.3	12,9	10
Weight $/\mathrm{kg/m}$	30,7	30,3	18,4	15,8
Weight in % of steel beam	100%	99%	60%	51%

Table 5.1: Beams with same stiffness

Strength-to-elastic modulus ratio

Most of the structural aluminium alloys have relatively high “strength-to-elastic modulus” ratio. This effect is especially clear when the aluminium alloy is strain-hardened or heat-treated. Structural aluminium alloys have roughly twice the strength-to-elastic modulus ratio than standard steels. However, when compared with high strength steels, structural aluminium alloys have about the same ratio.

This large strength-to-elastic modulus ratio means that, if an aluminium structure is designed according to deflection criteria, the stress is very often low. It also means that if the design of the structure is based on strength it is often possible to save weight by using aluminium. This is illustrated in Table 7, where it can be seen that the weight is not very much depending of the height of the section in these examples.

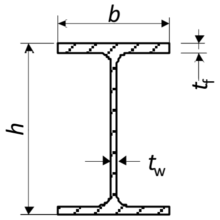
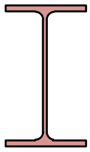
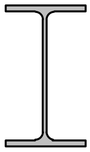
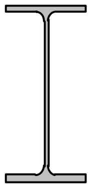
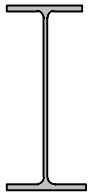
				
Material	steel	aluminium	aluminium	aluminium
f_y or f_o /MPa	355	260	260	260
Moment resistance /kNm	108	108	108	108
h/mm	240	240	300	330
b/mm	120	120	120	120
t_w/mm	6.2	8.0	7.0	6.0
t_f/mm	9.8	14.2	11.5	11.0
Weight /kg/m	30.7	13.8	12.9	12.3
Weight in % of steel beam	100%	45%	42%	40%

Table 5.2 – Beams with same moment resistance

Strength reduction after welding

Aluminium alloys used in structural applications are strengthened by heat-treating or cold working. This means that the strength of the alloy is reduced locally by welding, hot-straightening or hot-forming.

6xxx alloys in T6 temper lose roughly half of their strength in the heat affected zone when welded. Alloys in T4 temper generally retain their strength. See Chapter 3.6

The strength of alloys can be restored by artificial ageing after welding.

To avoid reduction of the load-bearing resistance, welds should - if possible - be situated in areas where stresses are low, such as the neutral axis of a beam in bending.

Fatigue

When fatigue may be a key design factor, it must be considered that the fatigue strength of a welded part is roughly 40% of that of steel. No problems arise for not welded structural members.

Relatively low hardness

It is suggestable to avoid unnecessary transport and components that, because of their size and shape, are prone to deformation or surface damage. For hardness values, see Chapter 3.6.

Not prone to *brittle fracture* at low temperatures

Contrary to steel, aluminium does not become brittle at low temperatures, as it does not have a brittle transition temperature. Aluminium tends to become tougher, stiffer and stronger at low temperatures.

Low *damping factor*

Where oscillation is induced by varying imposed frequencies, such as gusts of wind, the structure should be made stiff enough to ensure that the natural frequency is considerably higher than the largest imposed frequency.

High thermal expansion

The coefficient of thermal expansion is twice that of steel. i.e. 0.000023/°C. Changes in ambient temperature and other temperature variations must be considered, if the corresponding movement could result in significant forces. However, because of the low modulus of elasticity, the stresses caused by restrained expansion are moderate. For instance, if a beam is rigidly fixed at the ends, a temperature change produces a stress state in an aluminium beam, which is 2/3 of that in a steel beam.

High thermal conductivity

The high thermal conductivity means for instance that the temperature differences on both sides of a profile equalize quickly.

High corrosion resistance

Corrosion resistance is often a key reason for choosing aluminium. See Chapter 4. Aluminium normally does not require corrosion protection. Attention should however be given to the risks of standing water and crevice corrosion.

Extrudability

Extrusion technology offers many possibilities for creating tailor-made profiles. It is often profitable to produce a special profile even for moderate quantities. See Chapter 3.9.2.

Formability and machining

Aluminium is easily worked even in the cold state, especially in low states of temper. Aluminium can be formed in press brakes and roll presses and can be deep-drawn, deep-pressed, hydroformed, etc.

Tight cross-sectional tolerances

Extruded profiles with open cross-sections are produced to tight tolerances and can be produced in very straight lengths by stretching after extrusion. Cross-sectional tolerances for hollow profiles can be slightly larger.

Low residual stress

In contrast to hot-rolled steel profiles, extruded aluminium profiles have low residual stresses in the longitudinal direction. This advantage respect to steel can be found also in built-up aluminium sections, despite of heat-affected zones, due to the low elastic modulus and the high thermal conductivity.

5.3. Limit state design

Limit state design and partial safety factor are the methods which the design standards are based upon. In Europe the EN 19xx standards (Eurocodes) are the basis for the structural design of all structural materials in civil engineering. For the design of aluminium structures, the complete standard package to be followed is the following:

- *EN 1990 Eurocode 0 – Basis for structural design*, which gives the partial safety factor on loads and rules for combination of loads to give the different action effects.
- *EN 1991 Eurocode 1 – Actions on structures,* which gives the characteristic loads for structures and buildings such as self-weight, live loads (Part 1-1), wind loads (Part 1-4), snow loads (Part 1-5), action during execution (Part 1-6), traffic loads on bridges (Part 2) etc.
- *N 1999 Eurocode 9 – Design of aluminium structures*, which gives the design rules for aluminium structures.

National Annex

National standards implementing the European standards have a National Annex containing Nationally Determined

Parameters to be used for the design of building and civil engineering structures in the relevant country. These National Annexes can give other values of relevant parameters (e.g. of partial factors), than the ones which are recommended in the Eurocodes. Notes in the Eurocodes indicate where national choices are allowed.

Limit states

According to the Eurocodes, calculations should be carried out for the two limit states:

– *Serviceability limit state;*

– *Ultimate limit state.*

The *serviceability limit state* sets the requirements for normal use. For aluminium structures it imposes requirements on deflection and, in some cases, vibration. As mentioned above, this limit state is often the critical design situation for aluminium structures because of the aluminium's low modulus of elasticity. Normal loads (i.e. without partial safety factors and also low combination factors) are used in this check.

The *ultimate limit state* is used to check that the structure has adequate strength regarding material failure, instability (torsional, lateral torsional and flexural-torsional buckling) and collapse. Fatigue and strength when exposed to fire are further ultimate limit states.

5.4. Serviceability limit state

As already mentioned, it is important to check deflection and other deformations in aluminium structures. It is often required to check that the calculated deflection for a beam is less than a value which depends on the span. EC 9 does not give any limits for deflection. According to *EN 1990 – Basis of structural design*, limits for deflections should be specified for each project and agreed with the owner of the construction work. The National Annex to the code can specify limits for deflections and limits for vibration of floors.

Examples of limits for deflections are given in Table 5.3. A deflection > $L/200$ can usually be seen with the naked eye.

Design situation	Deflection limit, L = span
Floor beams in buildings	$L/400$ + vibration requirements
Road bridge	$L/600$
Railway bridge	$L/800$
Spectator stands	$L/400$
Beams carrying plaster or other brittle finish	$L/400$
Purlins and sheeting rails on roofs	Typically, $L/200$ and 25 mm to avoid ponding (damming up water to form a small pool)
Glass facades and roofs	$L/200$

Table 5.3: Examples of vertical deflection limits

Calculation of deflections and deformations in the serviceability limit state are elastic calculations and normally based on the moment of inertia for the gross cross-section of the member. However, for members in cross-section class 4 (cf. Chapter 5.6.2) the moment of inertia should be reduced according to 7.2.4 in EN 1999-1-1.

5.5. Ultimate limit state method

The ultimate limit state is the situation where the safety of the structure is checked. A structure shall not collapse and design in accordance with the ultimate limit state shall avoid structural failure. The partial safety factor for the resistance (γ_M) shall take care of the scattering of the strength properties and the geometry of the cross-section.

The partial safety factor for the load effects (γ_Q and γ_G) shall take care of the scattering in the determination of the loads and of the probability in the combination of different loads. The partial safety factor is different for the different types of loads, their uncertainty and how they are combined. Dead loads (i.e. self-weight of the structure) have a low partial safety factor, while the live loads (i.e. all forces that are variable during operation, e.g. snow loads, wind load, imposed loads in buildings, traffic loads …) have a higher partial safety factor.

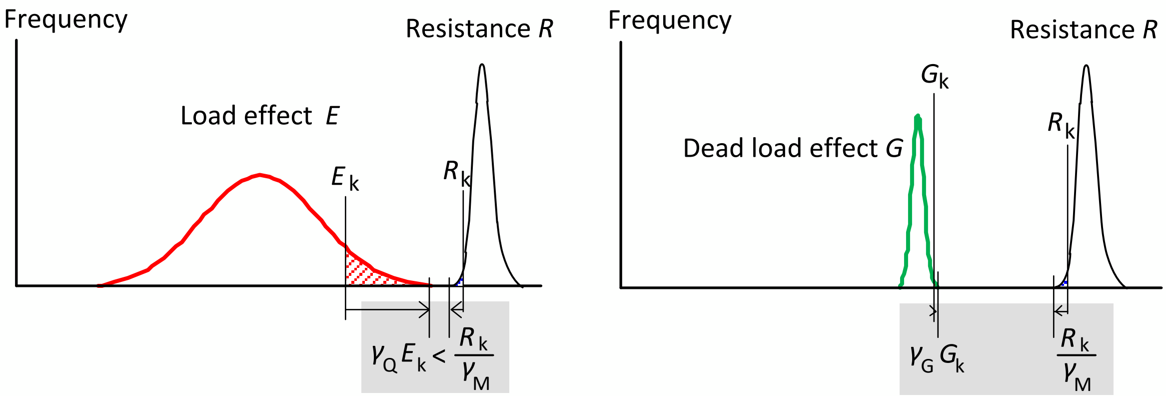


Figure 5.1 – Illustration of the partial factor method

The condition to be fulfilled is (see Figure 5.1):

$$\gamma_Q E_k \leq \frac{R_k}{\gamma_M}$$

where:

E_k is the characteristic value of the load effects; it may be axial tension or compression, bending moment, shear or a combined load effect on a cross-section or on a connection.

R_k is the characteristic value of the resistance; it may be axial tension or compression, bending moment, shear or a combined resistance.

γ_M is the partial safety factor for the resistance, also called material factor.

γ_Q is the partial safety factor for the load effects, also called load factor.

Typical values for the partial safety factor for the resistance are 1.10 (γ_{M1}) for members and 1.25 (γ_{M2} and γ_{Mw}) for bolt and rivet connections and welded connections.

These are the material factors for building and civil engineering structures and may also be used in all structural design because the material, the geometrical dimensions and the fabrication of connections are almost similar in all aluminium structures.

Typical values for the load effect factors in buildings and civil engineering are 1.2 – 1.3 for dead loads and 1.5 for live loads. These factors may also be used for the design of components for other structures, like commercial vehicles.

5.6. Ultimate limit state for members

5.6.1 General

Buckling class for materials, cross-section class, effective thickness to allow for local buckling and HAZ as well as methods for members in compression and bending are specific to Eurocode 9. These are therefore presented in more details in the following paragraphs. Further rules and methods are listed, and references are given in Tables 5.4 and 5.5.

The resistance of a member in compression depends on material properties, cross-section type and dimensions and member properties. Material properties like strength and Buckling Class (BC) (see 5.6.2), together with cross-section properties constitutes the cross-section class which depends on the slenderness b/t of the cross-section parts, see Figure 5.2.

The cross-section class defines the stress distribution at the ultimate limit state and thus how the cross-section resistance is calculated.

Material Buckling Class and HAZ define which buckling curve should be used and, together with member length, support conditions and cross-section resistance, the resistance of the member can be calculated.

5.6.2 Cross-section class

Cross-sections are classified in 4 classes. In Figure 5.2, the stress distribution for the different classes are shown which identify how the cross-section behaves during bending. This is directly linked to the resistance of the cross-section. Limits for b/t for the different classes are given Table 6.2 of EC 9. For Buckling Class, slenderness b/t and cross-section class, see also items 3, 4 and 5 in the overview in 5.6.6 A).

Class 1, "ductile": The resistance may be calculated based on plastic behaviour taking the hardening of the material into account. Plastic hinges can develop in a continuous beam or frame.

Class 2, "compact": The resistance may be calculated based on plastic behaviour without hardening. Elastic theory is used for continuous beams or frames.

Class 3, "semi-compact": The cross-section can develop elastic or partly plastic resistance. Elastic theory is used for continuous beams or frames.

Class 4, "slender": Thin parts of a cross-section may buckle before attainment of the proof stress. The resistance is based on an effective cross-section. Elastic theory is used for continuous beams or frames. Usually the reduction of the stiffness due to local buckling in the calculation of the moment distribution is omitted.

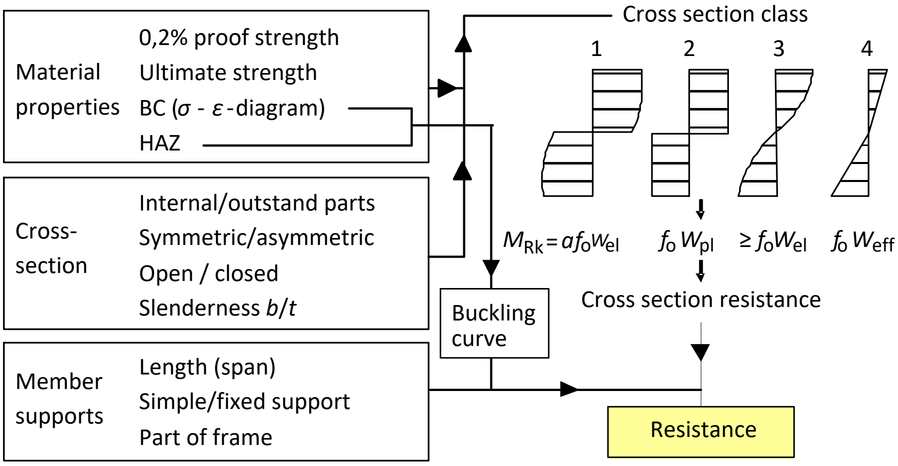


Figure 5.2: Scheme to calculate member resistance

5.6.3 Material Buckling Class (BC) and buckling curves

The buckling resistance of aluminium columns depends merely on initial bow (geometrical imperfection), residual stresses (structural imperfection) and stress-strain relationship. Furthermore, size and position of HAZ due to heat introduction during welding affect buckling resistance of longitudinally welded columns.

For steel members the residual stresses are the most important factor and, as the magnitude and distribution of the residual stresses depends on the cross-section type, there are different buckling curves for different cross-section types in the Eurocode for steel. As already mentioned, the residual stresses for extruded aluminium profiles are very small. However, the stress-strain relationship is strongly non-linear from the origin, which is the reason why, in the case of aluminium, the choice of buckling curves for extruded profiles depends on the alloy and temper expressed by the Buckling Classes A and B. In Figure 5.3 examples of stress-strain curves for the two Buckling Classes are shown. The corresponding buckling curves are designated 1 and 2 (see Figure 5.4).

For longitudinal welded aluminium members, however, there are residual stresses where the distribution is similar as in welded steel members. Although, compared to yield strength, the magnitude in aluminium is smaller than in steel. Nevertheless, the influence on the buckling resistance is similar as for steel, so the resistance should depend on the cross-section type. However, as aluminium members can have many different cross-section shapes and, for sake of simplicity, a reduction factor κ is given in Table 6.6 in EC9 for class A and class B materials.

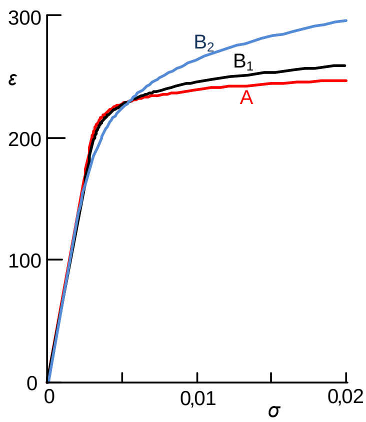


Figure 5.3: Example of a stress-strain curves for Buckling Class A and two curves for Buckling Class B

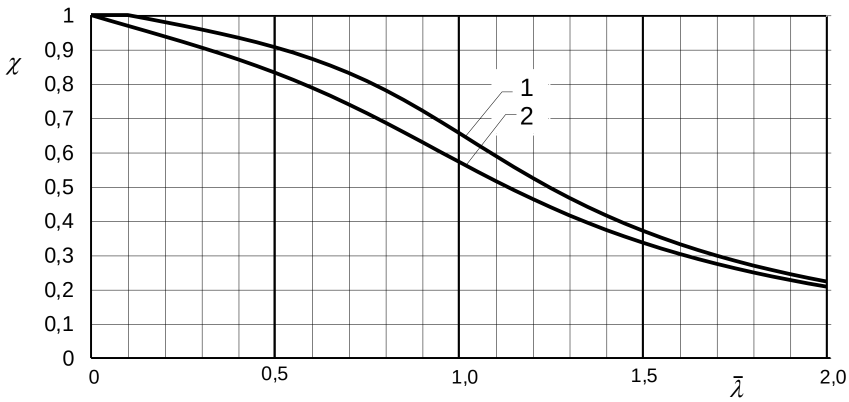
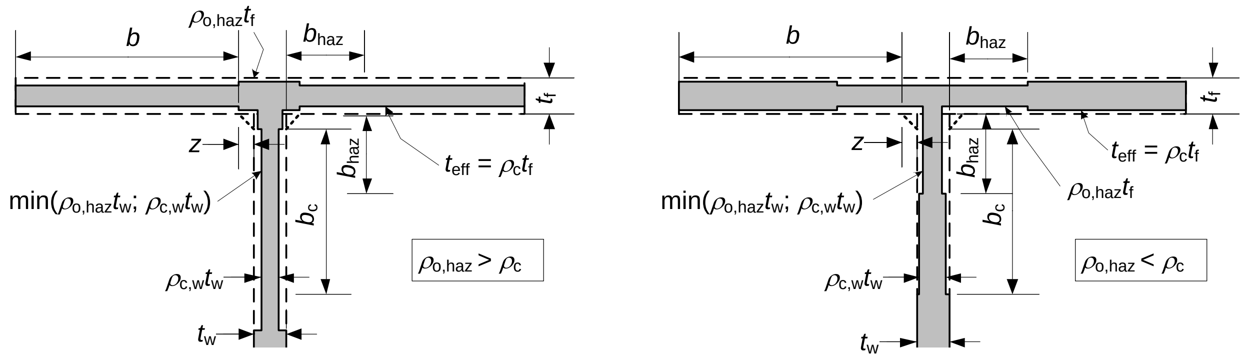


Figure 5.4: Reduction factor χ for flexural buckling

5.6.4 Effective thickness due to local buckling and HAZ softening

Contrary to steel, the effective thickness method instead effective width method is used to take reduction of resistance due to local buckling into account. Besides simpler calculations, the advantage is that combination with local reduction of the 0,2% proof strength $f_{o,haz}$ and of the ultimate strength $f_{u,haz}$ in HAZ is simpler (see Figure 5.5).



a. The reduction in HAZ is less than the reduction due to local buckling

b. The reduction in HAZ is larger than the reduction due to local buckling

Figure 5.5 – Effective cross-section of class 4 compression part of welded member

The severity of softening in HAZ is generally larger in BC A than in BC B material, where for some material the reduction is null or very small. For example, the 0,2% proof strength in HAZ is half the strength in the base material for EN-AW 6082-T6, whereas for temper T4 of the same alloy the reduction is just 10% according to Table 5.2 a) of Eurocode 9.

5.6.5 Bending and axial compression

The procedure and formulae for bending and axial compression are different from them for steel. In Eurocode 9, there are two special means to cover the influence of local buckling, plastic strain and second order effects:

- exponents η_i, β_i and δ_i on the terms in the interaction formulae to account for plastic strain and local buckling, where $i = y$ or z depending in buckling direction (formula below);
- factor $\omega_{x,y}$ and $\omega_{x,y,\mathrm{LT}}$ to take the second order moment distribution along the member into account.

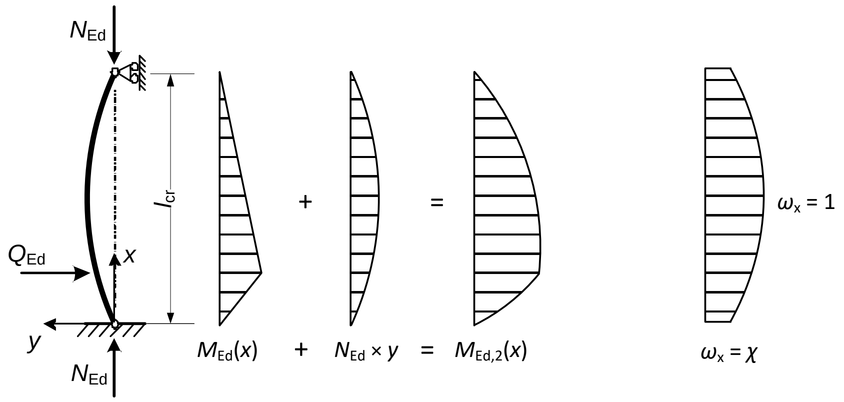


Figure 5.6: Moments and ω_x-diagram for a member in bending and compression

As an example, the formula for lateral-torsional buckling is:

$$\left(\frac{N_{\mathrm{Ed}}}{\omega_{x,z}\chi_{\mathrm{z}}N_{\mathrm{Rd}}}\right)^{\eta_c}+\left(\frac{M_{\mathrm{y,Ed}}}{\omega_{x,y,\mathrm{LT}}\chi_{\mathrm{y,LT}}M_{\mathrm{y,Rd}}}\right)^{\gamma_{\mathrm{c}}}+\left(\frac{M_{\mathrm{z,Ed}}}{M_{\mathrm{z,Rd}}}\right)^{\xi_{\mathrm{zc}}}\leq 1{,}00$$

With these exponents and factors, the same interaction formulae can be used for all cross-section classes 1 to 4.

The three terms in the formula are measures of how much the respective resistances are used due to axial force, *y-y*-axis bending (including influence of lateral-torsional buckling) and *z-z*-axis bending. They are therefore called *utilization grades*. The first term includes the influence of the bending moment of the axial force times the deflection.

The exponents, which are depending on the shape factors α_y and α_z, result in different shape of the interaction curves for different cross-sections (see Figure 5.7). For compression and strong axis bending, the shape factor is up to 1,15 with rather small plastic reserve in the web after that plastic strain started in the flanges. For weak axis bending the reserve is larger with shape factor up to 1,5 and slightly larger. This results in curves which are strongly convex upwards. For cross-section class 3 the curves start with straight lines for shape factor = 1 to strongly convex curves for class 2 cross-sections. As the exponents are functions of the shape factor, there is a smooth transition between class 2 cross-sections to class 4 cross-sections represented by the curves for cross-section class 3.

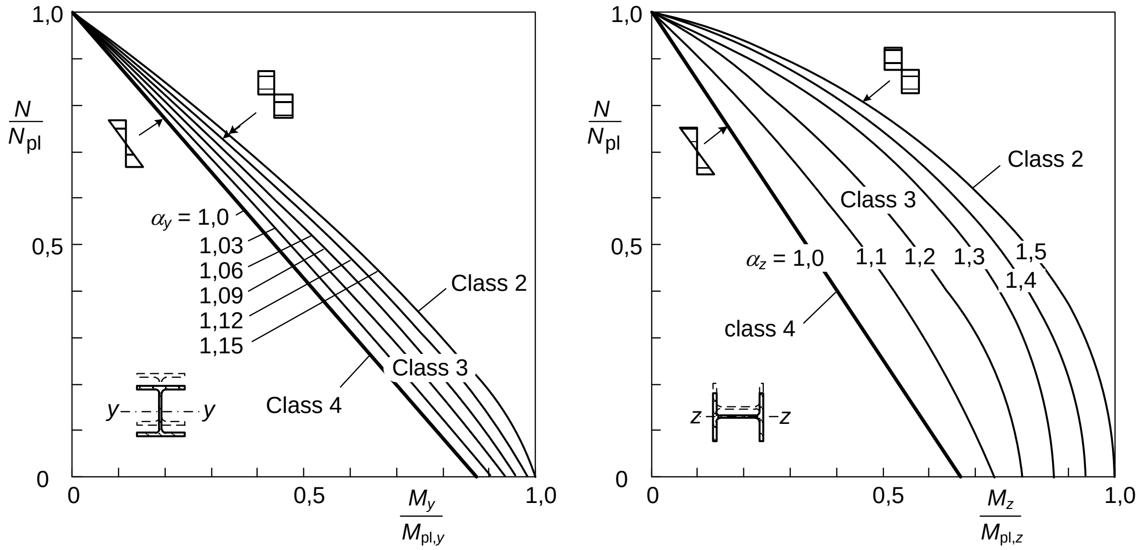


Figure 5.7: Interaction diagrams for strong axis and weak axis bending and compression of H-beams in different cross-section classes

The factor taking the distribution of the second order moment along the member into account is included in the interaction formula which, for strong axis buckling, is

$$\left(\frac{N_{\mathrm{Ed}}}{\omega_x\chi_{\mathrm{y}}N_{\mathrm{Rd}}}\right)^{\xi_{yc}}+\frac{M_{\mathrm{y,Ed}}}{M_{\mathrm{y,Rd}}}\leq 1{,}00$$

where the factor is

$$\omega_x = 1/\left(\chi+(1-\chi)\,sin\frac{\pi x}{l_{\mathrm{cr}}}\right)$$

$\xi_{yc} = \alpha_y^2 \cdot \chi_y \ \leq 0.8;$

α_y is the shape factor;

χ_y is the reduction factor for flexural buckling.

The ω_x factor is smallest in the middle of the span, where (l_{cr} being the buckling length)

$x = l_{\mathrm{cr}}/2$ so $\omega_x = 1/\left(\chi + (1-\chi)\, sin\frac{\pi}{2}\right) = 1$ and

$$\frac{N_{\mathrm{Ed}}}{\omega_x \chi_{\mathrm{y}} N_{\mathrm{Rd}}} = \frac{N_{\mathrm{Ed}}}{\chi_{\mathrm{y}} N_{\mathrm{Rd}}}$$

which is the utilization grade for buckling.

At the support

$x = 0$ so $\omega_x = 1/\chi_y$ and

$$\frac{N_{\mathrm{Ed}}}{\omega_x \chi_{\mathrm{y}} N_{\mathrm{Rd}}} = \frac{N_{\mathrm{Ed}}}{N_{\mathrm{Rd}}}$$

which is the utilization grade for section resistance as it is no second order bending moment in a simple support.

In between the utilization grade of the axial force follow $1/\omega_x$ which is the inverse of a sine curve. This is illustrated in Figure 5.8 a), where the grey part corresponds to the influence of the moment of the axial force times the deflection.

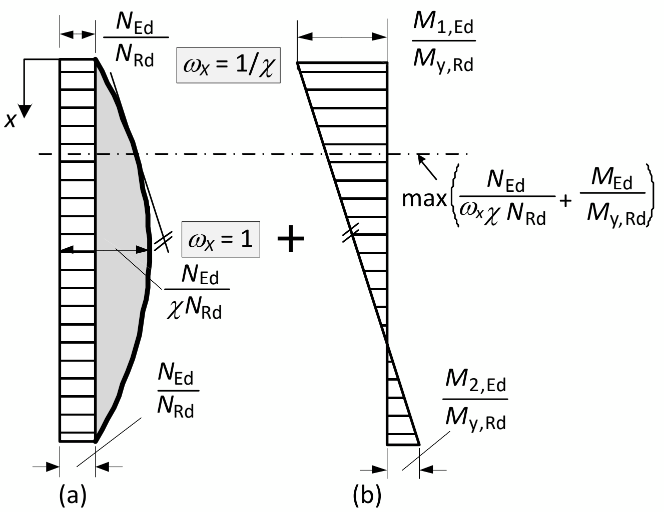


Figure 5.8 – Compression and bending according to the Eurocodes 9 for aluminium

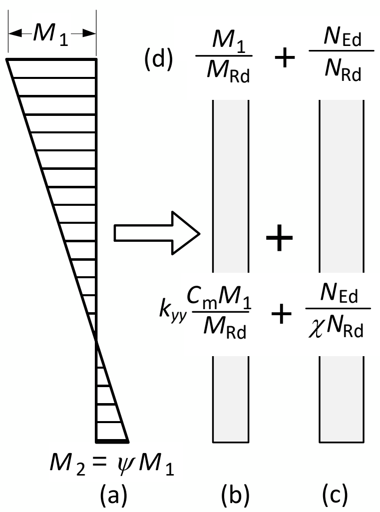


Figure 5.9: Compression and bending according to the Eurocodes 3 for steel

This utilization grade due to the axial force is added to the utilization grade of the bending moment with arbitrary distribution, as shown in Figure 5.8 (b). (The exponents on the utilization grades have been omitted for clarity). The sum of the utilization grades should be less than 1 in every section along the member. In the example in Figure 5.8 maximum of (a)+(b) occur where a line parallel to the moment diagram is a tangent to the sine curve.

In the Eurocode for steel, the principle is to find a constant moment which gives correct result together with constant utilization grade $N_{Ed}/(\chi_y N_{Rd})$, which means that a lot of factors are needed. This is illustrated in Figure 5.9 where the actual moment diagram (a) is transferred to an equivalent constant moment (b) which is then combined with the utilization grade due to the axial force (c). It is then necessary to check the resistance at the ends (d) separately.

As the exponent ξ_{yc} in this case is depending on the reduction factor χ_y, the interaction curves will be convex downwards for large relative slenderness. See example in Figure 5.10 (a).

With this estimation of the utilization grade along the member, it is possible to check reduction of localized HAZ due to transverse weld in any section along the member.

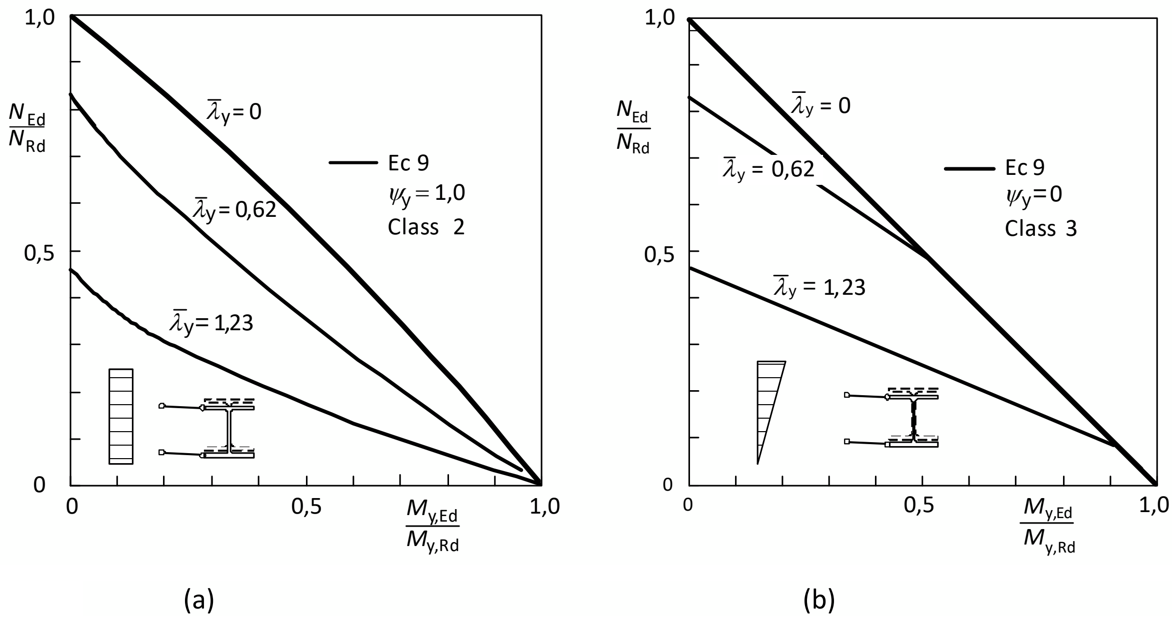


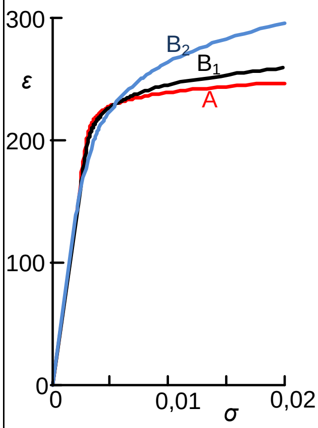
Figure 5.10 – Interaction curves for flexural buckling for (a) uniform moment and (b) variable moment

5.6.6 Overview of the main design

Relevant rules and design formulae according to Eurocode 9 are summarized in the following, divided in groups of topics. The numbers of reference of Sections, Figures and Tables correspond to the 2007 version of Eurocode 9, as amended in 2009.

A) Material strengths, partial factors, Buckling Classes, cross-section classes, local buckling and HAZ

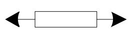
Subject	Reference to EN 1999-1-1	Notations and limits
1. Material strengths	3.2	f_o is the characteristic 0,2% proof strength for bending and overall yielding in tension and compression

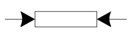
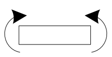
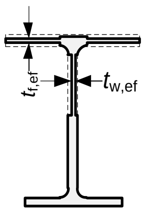
	Table 3.2 (a to e) Table 3.3 6.2.1	f_u is characteristic ultimate tensile strength for the local capacity of a net section in tension or compression f_w is characteristic strength of the weld metal Values for sheets, strips and plates are given in Table 3.2a and values for extruded profiles in Table 3.2b. For forgings values are given in Table 3.2c and for gravity castings in Table 3.3. If formulae in 6.2.2 to 6.2.10 do not apply, formula (6.15) can be used.
2. Partial factors	6.1.3	γ_{M1} is used for the resistance of cross-sections and resistance of members to instability based on f_o. Recommended value is 1,1. γ_{M2} is used for the resistance of cross-sections to fracture and resistance of joints in tension, shear and bearing based on f_u or f_w. Recommended value is 1,25.
3. Material Buckling Class BC 	3.2.2, 6.1.4.4, 6.1.5, 6.3.1 Table 5.2 a) and b)	The resistance if instability is involved (local, flexural, lateral torsional … buckling) is depending on the shape of the stress-strain curve. As this curve is different for different materials, the materials and tempers are divided into two Buckling Classes A and B. The Buckling Class for the alloys are given in Table 5.2 a) and b). Effective thickness and buckling curve are depending on the Buckling Class, but also if a member is longitudinally welded or not.
4. Slenderness	6.1.4.3	$\beta = \eta \frac{b}{t}$ b is the width of a cross-section part t is the thickness of a cross-section part η is the stress gradient factor $\eta = 0{,}70 + 0{,}30\psi \qquad (1 \geq \psi \geq -1)$ $\eta = 0{,}80/(1-\psi) \qquad (\psi < -1)$ ψ is the ratio of the stresses at the edges of the plate under consideration related to the maximum compressive stress.

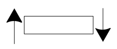
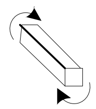
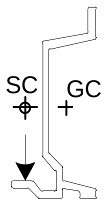
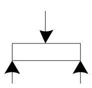
Slenderness limits	6.1.4.4, Table 6.2	Limits for the cross-section classes denoted β_1, β_2 and β_3 are given in Table 6.2.
5. Cross-section class	6.1.4.2	Class 1 cross-sections are those that can develop their plastic moment resistance forming a plastic hinge with the rotation capacity required for plastic analysis. The hardening effect may be taken into account according to Annex L. $\beta \leq \beta_1$
Limits for cross-section parts of members in bending	6.1.4.4	Class 2 cross-sections are those that can develop their plastic moment resistance but have limited rotation capacity because of local buckling. The resistance is based on perfectly plastic behaviour. $\beta_1 < \beta \leq \beta_3$
		Class 3 cross-sections are those in which the calculated stress in the extreme compression fibre can reach its proof strength, but local buckling is liable to prevent development of the full plastic moment resistance. The resistance is based on elastic or partly plastic behaviour. $\beta_2 < \beta \leq \beta_3$
		Class 4 cross-sections are those in which local buckling will occur before the attainment of proof strength in compression in any part of the cross-section. The resistance is based on an effective cross-section. $\beta > \beta_3$
6. Local buckling	6.1.5 6.1.5, Table 6.3	Local buckling in class 4 members should be taken into account by replacing the true section by an effective section, obtained by employing a local buckling factor ρ_c to reduce the thickness. Factor ρ_c is depending on the material Buckling Class and should be computed from formulae (6.11) or (6.12), separately for different parts of the section. The factor is different for internal cross-section parts and outstand parts and depends on whether the member is longitudinally welded or not.
7. HAZ	6.1.6	The characteristic value of the 0,2 % proof strength $f_{o,haz}$ and of the ultimate strength $f_{u,haz}$ in the heat affected zone should be taken from Table 5.2, which also gives the reduction factors

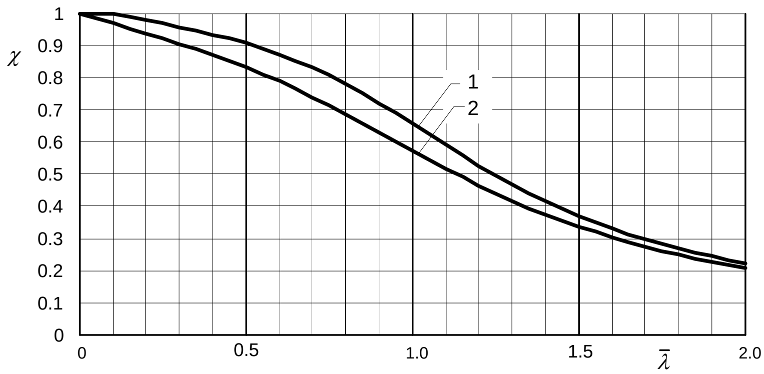
	6.1.6.3	$\rho_{\text{o,haz}} = \frac{f_{\text{o,haz}}}{f_o}$ and $\rho_{\text{u,haz}} = \frac{f_{\text{u,haz}}}{f_u}$ Local buckling effects and effects due to HAZ softening should be included by means of the effective thickness. The extent of b_{haz} of HAZ depends on the plate thickness and the welding method.
8. HAZ and local buckling	6.2.5.2, for example see Figure 5.5 above	For class 4 part with HAZ effects, the effective thickness is taken as the lesser of that corresponding to the reduced thickness t_{eff} and that corresponding to the reduced thickness $\rho_{\text{o,haz}}t$ in the softened part, and as t_{eff} in the rest of the compressed portion of the cross-section.
9. Transverse welds	6.3.1.1, 6.3.3.3	At a section with transverse weld $\rho_{\text{o,haz}}$ is replaced by $\rho_{\text{u,haz}}$. The reduction due to local welds is depending on where the reduction is located along the member, less reduction at the end than at the span of a member in compression. This is accomplished with the ω_{x} expression.
10. Holes	6.2.2.2, 6.3.3.4	The resistance is based on the net area through the section with the holes.

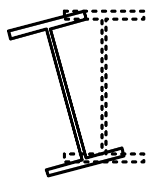
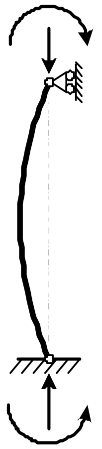
B) Resistances for different design situations

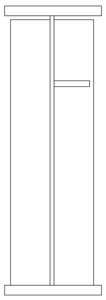
Design situation	Reference to EN 1999-1-1	Resistance
1. Tension 	6.2.3	The smallest of: general yielding along the member $N_{\text{o,Rd}} = A_g f_o / \gamma_{\text{M1}}$ local failure at a section with holes $N_{\text{net,Rd}} = 0{,}9 A_{\text{net}} f_u / \gamma_{\text{M2}}$ local failure at a section with transverse weld $N_{\text{u,Rd}} = A_{\text{u,eff}} f_u / \gamma_{\text{M2}}$ A_{g} is either the gross section or a reduced cross-section to take into account HAZ softening due to longitudinal welds. In the latter case A_{g} is found by taking a reduced area equal to $\rho_{\text{o,haz}}$ times the area of the HAZ; A_{net} is the net section area, with deduction for holes and a deduction, if required, to take into account the effect of HAZ softening in the net section through the hole. The latter deduction is based on the reduced thickness $\rho_{\text{u,haz}}t$; $A_{\text{u,eff}}$ is the effective cross-section area based on the reduced thickness

		$\rho_{\mathrm{u,haz}}t$;
2. Compression with no buckling 	6.2.4	The smallest of: in sections with unfilled holes $N_{\mathrm{net,Rd}} = A_{\mathrm{net}} f_u / \gamma_{\mathrm{M2}}$ in sections with transverse weld $N_{\mathrm{u,Rd}} = A_{\mathrm{u,eff}} f_u / \gamma_{\mathrm{M2}}$ in other sections $N_{\mathrm{o,Rd}} = A_{\mathrm{eff}} f_o / \gamma_{\mathrm{M1}}$ A_{net} is the net section area, with deductions for unfilled holes and HAZ softening, if necessary. For holes located in reduced thickness regions the deduction may be based on the reduced thickness, instead of the full thickness; A_{eff} is the effective section area based on reduced thickness taking into account local buckling and HAZ softening due of longitudinal welds but ignoring unfilled holes; $A_{\mathrm{u,eff}}$ is effective section area, obtained using a reduced thickness $\rho_c t$ for class 4 parts and reduced thickness $\rho_{\mathrm{u,haz}}$ for the HAZ material, whichever is smaller.
3. Bending moment  	6.2.5 6.2.5.2 6.2.5.1	The smallest of: in a net section $M_{\mathrm{net,Rd}} = W_{\mathrm{net}} f_u / \gamma_{\mathrm{M2}}$ in section with transverse weld $M_{\mathrm{u,Rd}} = W_{\mathrm{u,eff}} f_u / \gamma_{\mathrm{M2}}$ in any cross-section $M_{\mathrm{o,Rd}} = \alpha W_{\mathrm{el}} f_o / \gamma_{\mathrm{M1}}$ W_{el} is the elastic modulus of the gross section; W_{net} is the elastic modulus of the net section taking into account holes and HAZ softening, if welded. The latter deduction is based on the reduced thickness of $\rho_{\mathrm{u,haz}}t$; $W_{\mathrm{u.eff}}$ is the effective section modulus, obtained using a reduced thickness $\rho_{\mathrm{c}}t$ for class 4 parts and reduced thickness $\rho_{\mathrm{u,haz}}t$ for the HAZ material, whichever is smaller; α is the shape factor given in Table 6.4. If elastic design is used for class 1, 2 and 3 cross-sections, then α = 1 (for members with longitudinal welds $\alpha = W_{\mathrm{el,haz}}/W_{\mathrm{el}}$). (Some notations are here not the same as in the code for clarity, e.g $M_{\mathrm{o,Rd}}$ here and $M_{\mathrm{c,Rd}}$ in the code)
4. Shear and transverse load	6.2.6	The design shear resistance for non-slender sections

	6.7.4, 6.7.5 6.7.6	$V_{\mathrm{Rd}} = A_v \dfrac{f_o}{\sqrt{3}\gamma_{M1}}$ A_{v} is the shear area For slender webs and stiffened webs, the resistance for plate girders webs should be used. The design resistance for transverse loads are given in 6.7.5 and interaction in 6.7.6.
5. Torsion  	6.2.7	The design St. Venant torsion moment resistance without warping (e.g. closed sections) $T_{\mathrm{t,Rd}} = W_{\tau,\mathrm{pl}} \dfrac{f_o}{\sqrt{3}\gamma_{M1}}$ $W_{\tau,\mathrm{pl}}$ is the plastic torsion modulus. For torsion with warping the torsion moment is the sum of two internal effects $T_{\mathrm{Ed}} = T_{\mathrm{t,Ed}} + T_{\mathrm{w,Ed}}$. Where the torsional moment is combined with a shear force, the resistance is given by a reduced shear strength. Torsion can be avoided by shaping the cross-section so that the shear centre SC is located in the plane of the loading. Closed cross-sections can resist large torsion moments.
6. Bending and shear	6.2.8	If the shear force is less than half the shear resistance its effect on the moment resistance can be neglected, otherwise the shear force will reduce the moment resistance.
7. Bending and axial force	6.2.9 6.3.3	Interaction formula are given in 6.2.9. They are the ground for the formulae for buckling resistance of members in bending and compression in 6.3.3.
8. Bending, shear and axial force	6.2.10	If the shear force is less than half the shear resistance, its effect on the combined axial force and moment resistance can be neglected, otherwise the shear force will reduce the bending and axial force resistances.
9. Web bearing 	6.2.11 6.7.5	The design of un-stiffened or longitudinally stiffened webs subjected to localised forces caused by concentrated loads or reactions applied to a beam is based on formulae for plate girders.

10. Buckling resistance of member in compression  See also 6.6.5 above.	6.3.1 Table 6.5 6.3.1.2 Figure 6.11	Members subject to axial compression should be considered to fail in one of three ways: • flexural • torsional or flexural torsional • local squashing The design buckling resistance of a compression member without transverse welds is $N_{b,Rd} = \kappa\chi A_{eff}/\gamma_{M1}$ χ is the reduction factor for the relevant buckling mode, material Buckling Class; A_{eff} is the effective area taking into account local buckling and HAZ softening of longitudinal welds. For torsional and torsional-flexural buckling see Table 6.7. (For class 1, 2 and 3 cross-sections without longitudinal welds, $A_{eff} = A_g$); κ is the a factor to allow the weakening effect of longitudinal welding given in Table 6.5. If there are no welds, then $\kappa = 1$. The reduction factor χ is a function of the relative slenderness $\bar{\lambda} = \sqrt{\frac{A_{eff} f_o}{N_{cr}}} = \frac{L_{cr}}{i}\frac{1}{\pi}\sqrt{\frac{A_{eff}}{A}\frac{f_o}{E}}$ Two curves are given, designated 1 and 2.  For a member with localised welds, the resistance is reduced at the section with the weld. The resistance is depending on where the weld is located along the member defined by the function ω_x.

	6.3.3.5	
11. Lateral torsional buckling of member in bending 	6.3.2 Table 6.4 6.3.3.5	Lateral torsional buckling may be neglected in any of the following cases: a) bending takes place about the minor principal axis; b) the member is fully restrained against lateral movement throughout its length; c) the relative slenderness $\bar{\lambda}_{LT}$ between points of effective lateral restraint is less than 0,4. The design buckling resistance moment of a laterally un-restrained member should be taken as $M_{b,Rd} = \chi_{LT}\alpha_i W_{el,y} f_o/\gamma_{M1}$ $W_{el,y}$ is the elastic section modulus of the gross section, without reduction for HAZ softening, local buckling or holes; α_i is the shape factor subject to the limitation $\alpha_i \le W_{pl,y}/W_{el,y}$; χ_{LT} is the reduction factor for lateral torsional buckling.
12. Members in bending and axial compression 	6.3.3 6.3.3.1 6.3.3.2	Members subject to bending and axial compression may fail in one of the two modes • flexural buckling • lateral-torsional buckling Interaction formulas are given for members with axial compression in combination with bending about one or two axis and fail for flexural buckling. These formulas are given for: • open double symmetric cross-section • solid cross-section • hollow cross-section and tube • open mono-symmetrical cross-section Interaction formula for lateral- torsional buckling of open cross-section symmetrical about major axis, centrally symmetric or double symmetric cross-section is given. Formulas are also given for the following effects:

	 6.3.3.3 6.3.3.4 6.3.3.5	• members containing localized welds • members containing localized reduction of cross-section • unequal end moments and/or transverse loads
13. Plate girders 	6.7 6.7.2, 6.7.3 6.7.4, 6.8 6.7.6 6.7.5 6.7.7 6.1.5 6.3.2	A plate girder is a deep beam with a tension flange, a compression flange, and a web plate. The web is usually slender and may be reinforced by transverse or/and longitudinal stiffeners. Webs buckle in shear at relatively low applied loads, but considerably amount of post-buckled strength can be mobilized due to tension field action. Plate girders are sometimes designed with transverse web reinforcement in form of corrugations or closely spaced transverse stiffeners (extrusions). Plate girders can be subjected to combinations of moment, shear, and axial loading, and to local loading on the flanges. Because of their slender proportions they may be subjected to lateral torsional buckling, unless properly supported along the length. Failure (buckling) modes may be: • web buckling by compressive stresses • shear buckling • interaction between shear force and bending moment • buckling of web due to local loads on flanges • flange-induced web buckling • torsional buckling of flange (local buckling) • lateral torsional buckling
14. Stiffened panels	6.6 6.7	Un-stiffened plates as separate components under in-plane loading is given in 6.6 and 6.7. The same applies for stiffened plates under in-plane loading. Out-of-plane loading is not treated in EC9.

15. Cold-formed structures	EN1999-1-4	Provisions for cold-formed structural sheeting are given in EN 1999-1-4.
16. Shells	EN1999-1-5	Provisions for shell structures are given in EN 1999-1-5.

C) Design of joints

1. Basis of design	8.1.2 Annex M	The forces and moments applied to joints at the ultimate limit state should be determined by global analysis including: — second order effects; — the effects of imperfections; — the effects of connection flexibility.

D) Connections made with bolts, rivets and pins

2. Spacings	8.5 Table 8.2	The rules for bolted connections are given in 8.5, where minimum, regular, and maximum spacing, end and edge distances for bolts are given.
3. Slotted, oversized holes, etc	8.5.1, 8.5.7 Table 8.4, 8.5.14	Long and short slotted holes and oversized holes are treated. Countersunk bolts and rivets. Pin connections.
4. Categories for shear connections	8.5.3.1	Category A: Bearing type; Category B: Slip-resistant at serviceability limit state; Category C: Slip-resistant at ultimate limit state.
5. Categories for tension connections	8.5.3.2	Category D: Connections with non-preloaded bolts; Category E: Connections with preloaded high strength bolts.
6. Failure modes	Table 8.5	Failure modes for bolted connections may be: • block tearing, failure in shear in a row of bolts along the shear face of a bolt group and tension failure along the tension face of the bolt group; • shear failure in the bolt; • bearing failure of the bolt hole; • tension failure of the bolt;

		• punching shear around the bolt head or nut; • combined shear and tension failure.
7. Prying forces, equivalent T-stub in tension	8.5.10 Annex B	Connection details that carry tensile forces, and where the tensile forces do not go directly through the bolts, additional forces in the bolts have to be accounted for. These forces are called prying forces (Q) and they can be considerable large. See the Figure 5.11. Prying action is important for T-stub in tension. See Annex B.

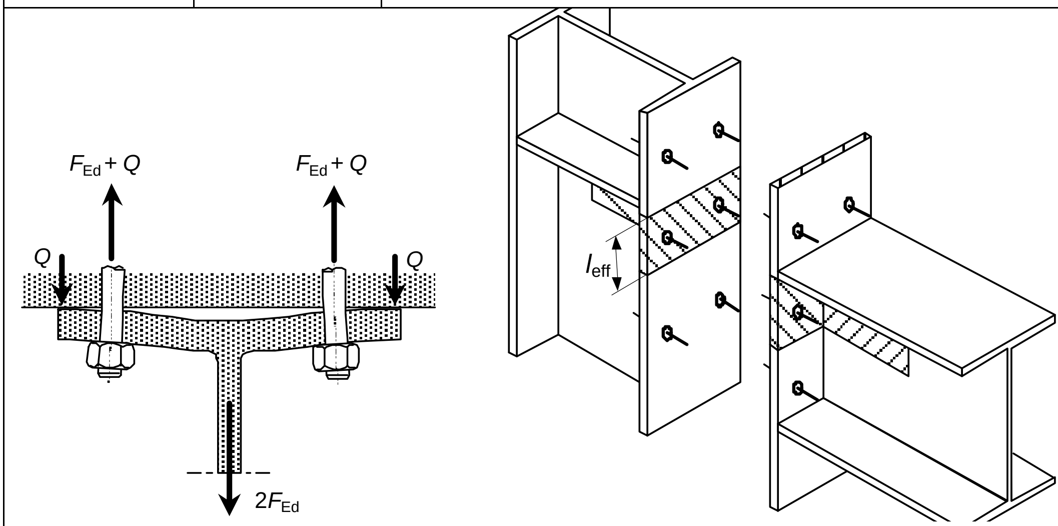


Figure 5.11: Prying action for T-stub in tension

E) Welded connections

1. General	3.3.4 3.3.4 EN 1011-4	The rules given in EC9, clause 8.6, apply to structures welded by MIG or TIG and with weld quality in accordance with EN 1090-3. Certified welders are highly recommended. Recommended welding consumables can be found in the references to the left.
2. Strength values	Table 5.2 Table 8.8	When welding hardened aluminium alloys, part of the hardening effect will be destroyed. In a welded connection it can be three different strengths: • the strength in the parents (not heat affected) material (f_o, f_u); • the strength in the heat affected zone ($f_{o,haz}, f_{u,haz}$); • the strength of the weld metal (f_w).

3. Check in weld and HAZ	Table 5.2 Table 8.8	Normally it will be necessary to check the stresses in the HAZ and in the welds. The strength in HAZ is dependent on the alloy, the temper, the type of product and the welding procedure. The strength in the weld (weld metal) is dependent on the filler metal (welding consumables) and the alloys being welded.
4. Single sided butt welds	8.6.3.2.2	Single sided butt welds with no backing is practically impossible to weld in aluminium. If single sided butt welds cannot be avoided, the effective seam thickness can be taken as: • the depth of the joint preparation for J and U types; • the depth of the joint preparation minus 3 mm or 25%, whichever is the less for V or bevel type. In addition to the single sided butt weld, a fillet weld may be used to compensate for the low penetration of the butt weld.
5. Practical precautions	8.6.1	When designing a welded connection some few practical precautions should be taken into account. • Provide good access to the welding groove. The “welding head” of the equipment used for welding aluminium is rather large, so there must be enough space around the weld. • Good access is also needed for checking the weld. All welds shall be 100 % visually examined in addition to some non-destructive testing (NDT). • Full penetration single sided butt welds are impossible to weld without any backing. • If possible, position the welds in areas where the stresses are low.
6. Butt welds	8.6.3	Heavy loaded members should be welded with full penetration butt welds. The effective thickness of a full penetration butt weld should be taken as the thickness of the thinnest connecting member. The effective length should be taken as the total length if run-on and run-off plates are used. If not, the total length should be reduced by twice the effective thickness. (Figure 5.12).
7. Design formulae for butt welds	8.6.3.2	Normal stress, tension or compression, perpendicular to weld axis: $\sigma_{\perp} \le f_w/\gamma_{\mathrm{Mw}}$ Shear stress: $\tau \le 0.6 f_w/\gamma_{\mathrm{Mw}}$ Combined normal and shear stress: $\sqrt{\sigma_{\perp}^2 + 3\tau^2} \le f_w/\gamma_{\mathrm{Mw}}$

<table>
<tr><td>8. Fillet welds</td><td>8.6.3.3</td><td>A fillet weld is defined with the throat thickness “a” given in mm. The Figure 5.12 shows how to measure the throat thickness. The effective length should be taken as the total length of the weld if:

• the length of the weld is at least 8 times the throat thickness;

• the length of the weld does not exceed 100 times the throat thickness with a non-uniform stress distribution;

• the stress distribution along the length of the weld is constant.</td></tr>
<tr><td>9. The forces acting on a fillet weld</td><td></td><td>The forces shall be resolved into stress components with respect to the throat section (see Figure 43)

These components are:

$\sigma_{\perp}$ normal stress perpendicular to the throat section;

$\sigma_{||}$ normal stress parallel to the weld axis;

$\tau_{\perp}$ shear stress acting on the throat section perpendicular to the weld;

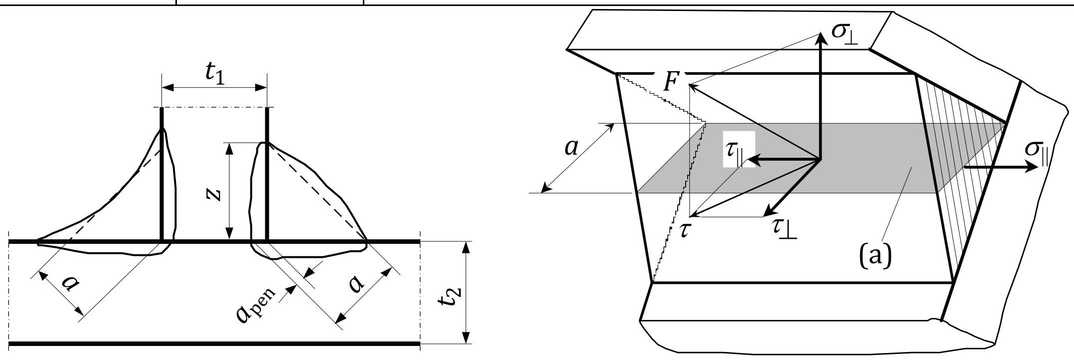
$\tau_{||}$ shear stress acting on the throat section parallel to the weld axis.</td></tr>
<tr><td colspan="3">


Figure 5.12</td></tr>
<tr><td>10. Heat affected zone</td><td>Figure 8.21</td><td>The stress in the heat affected zone has to be checked. The stress is calculated for the smallest failure plane for both butt welds and fillet welds. The sketches in Figure 5.13 indicate the failure plane for some welds:

W: weld metal, check of weld;

F: heat affected zone, check of fusion boundary;

T: heat affected zone, check of cross-section.</td></tr>
</table>

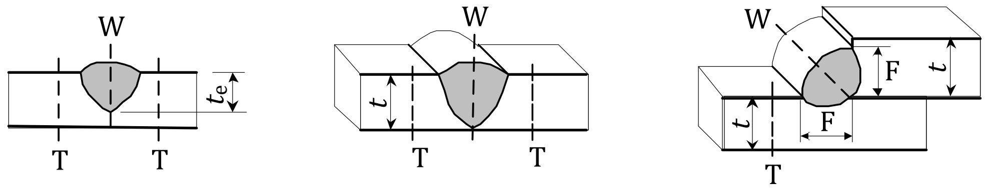


Figure 5.13

F) Friction Stir Welding, FSW

1. Friction Stir Welding	8.9	With the friction stir welding process, the weld is produced by a rotating tool which plastically softens the material at both sides of the weld line which preferably should be long and plane. Depending on the material thickness, the process requires stiff and strong welding fixtures and supports. Full penetration butt welds and lap joints can be produced, but not conventional fillet welds. Because the temperatures are below the melting point, welding problems which may occur with MIG or TIG welding are avoided.
2. Alternative structures	Figure 5.14	a) Single sided weld welded from one side, Figure 5.15 (1); b) profiles welded from top (2) and bottom (3); c) profile welded with half overlap from top (2) and bottom (3).

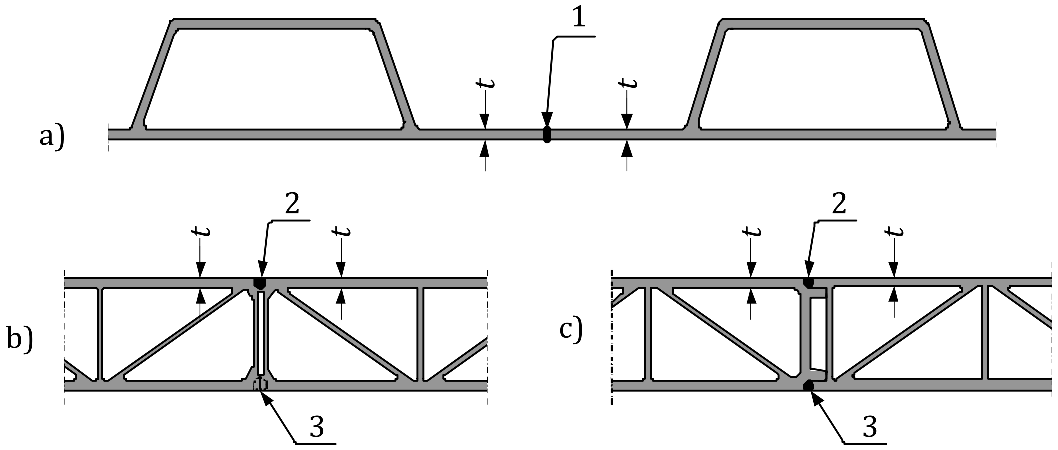
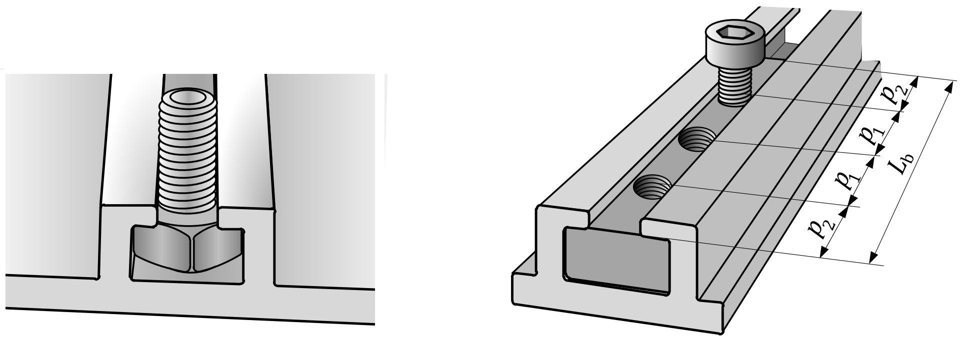


Figure 5.14: Friction Stir Welds, FSW

No design provisions are given for Friction Stir Welding in the 2007 version of EC9, as amended in 2009. However, these will be available in the new version of EC expected to be published in 2023.

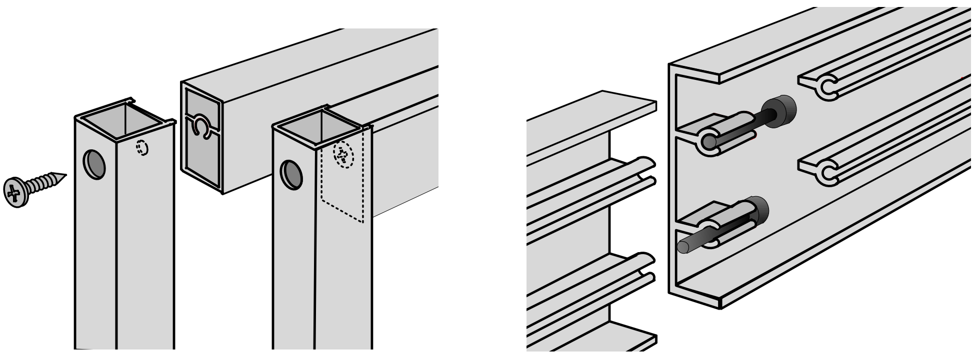
G) Bolt-channel joints and screw grooves

1. Bolt channel	8.9	The bolt-channel joining system consists of an extruded profile with a groove that is shaped to host the head or the nut of the bolts connecting the profile to the other components, see Figure 5.15 (a). When the connection is composed by more bolts arranged at a certain distance, a plate with threaded bolt holes can be inserted in the guide, see Figure 5.15 (b).



(a) (b)

Figure 5.15: Bolt channels



(a) (b)

Figure 5.16: Screw grooves

No design provisions are given for bolt channels and screw grooves the 2007 version of EC9, as amended in 2009. However, these will be available in the new version of EC expected to be published in 2023.

H) Adhesive bonded joints

1. Structural adhesive bonded joint	Annex M	Adhesive bonding needs an expert technique and should be used with great care. The design guidance in Annex P should only be used under the condition that:

		— the joint design is such that only shear forces have to be transmitted; — appropriate adhesives are applied; — the surface preparation procedure before bonding do meet the specifications as required by the application. The use of adhesive for main structural joints should not be contemplated unless considerable testing has established its validity, including environmental testing and fatigue testing if relevant.
2. Use	Annex M.4	Adhesive joining is very much used in specific structures (aeroplanes, vehicles) and can be suitably applied for certain building and civil engineering structures such as plate/stiffener combinations and other secondary stressed conditions.
3. Loaded area	Annex M.4	Loads should be carried over as large an area as possible. Increasing the width of joints usually increases the strength pro rata. Increasing the length is beneficial only for short overlaps. Longer overlaps result in more severe stress concentrations, in particular at the ends of the laps.
4. Strength	Annex M.5	The strength of an adhesive joint depends on the following factors: a) the specific strength of the adhesive itself; b) the alloy, and especially its proof strength if the yield stress of the metal is exceeded before the adhesive fails; c) the surface pre-treatment: chemical conversion and anodising, use of primers; d) the environment and the ageing; e) the configuration of the joint and the related stress distribution; Laboratory tests taking into account the whole assembly, i.e. the combinations of alloy/pre-treatment/adhesive, and the ageing or environment are important.

I) Fatigue

1. General		Structures with repeating loads may be susceptible to fatigue when the number of load cycles is high, even when the loads give low stresses in the structure. Fatigue failure starts with development of a crack at a point with stress concentrations. With continuous repeating loads the crack will grow, this will be shown as one striation in the failure surface

		for each load cycles. The distance between the striations is depending on the stress range and that is giving the growing speed.
2. Standards	EN 1999-1-3 EN 1090-3	Rules for fatigue design are given in EN 1999-1-3. The rules are based on quality levels given in EN 1999-1-3 and EN 1090-3.
3. Fatigue strength	EN 1999-1-3, 6.1, 6.2	The fatigue strength depends on: • type of detail (design); • stress range; • number of cycles; • stress ratio; • quality of manufacturing.
4. Stress range	EN 1999-1-3, 5.1.1	The stress range is defined as the algebraic difference between the stress peak and the stress valley in a stress cycle (Figure 5.17). At low stress ranges the crack grows slowly and with high stress range it grows fast.

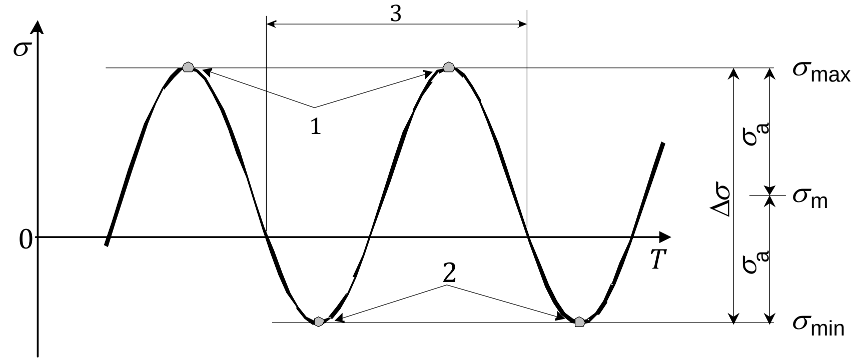


Figure 5.17: Stress range -1. Stress peak, 2. Stress valley, 3. Stress cycle, Δσ Stress range, σ_a Stress amplitude

5. Fatigue strength	EN 1999-1-3, 6.2	The properties of the parent material have very little influence on the fatigue strength in practical structures and components. For connections, the properties of the parent material have no influence at all. For a plate or extrusion with no manufacturing or only holes and notches the standard deviate between EN AW 7020 and all other structural alloys. The fatigue strength is given as SN curves for the different details. All detail categories given in EN 1999-1-3 have their own SN curve. A typical SN curve is shown in the Figure 5.18.

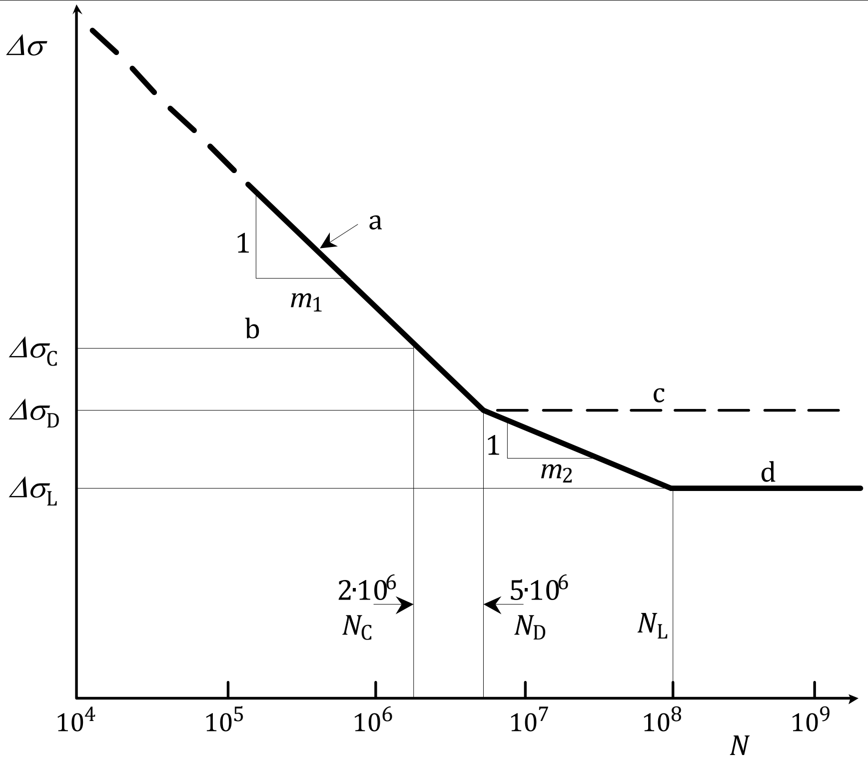


Figure 5.18: a. Fatigue strength curve; b. Reference fatigue strength; c. Constant amplitude fatigue limit; d. Cut-off limit

6. Detail Category	EN 1999-1-3, Annex J EN ISO 10042 EN 1090-3	Some typical details categories are shown in the Table J.1 The first column in the table gives the detail type number, the second row gives the detail category, the third gives a sketch of the detail and also showing the initiation site and the direction of the stress, the fourth gives the weld type, the fifth gives the stress parameter, the sixth gives the welding characteristics, the seventh gives the quality level for the internal imperfections and the eight gives the quality level for the surface and geometrical imperfections. The requirements for the quality levels are found in EN ISO 10042 and additional requirements are given in EN 1090-3. Detail Types 5.1 and 5.2 are examples where the same detail has different fatigue strength depending on the weld method. Detail type 13.1 and 13.2 show that an attachment (especially a long attachment) can have a detrimental influence on the fatigue strength. The SN curves that correspond to these detail categories are shown in Figure 5.19.

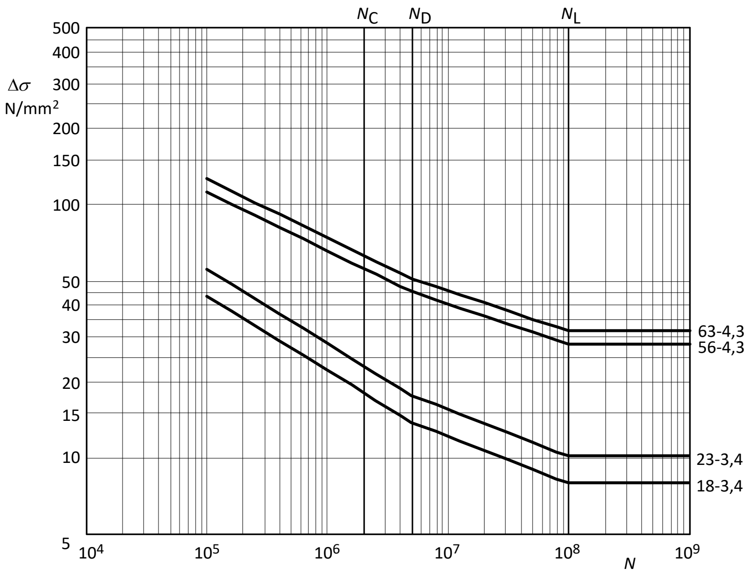
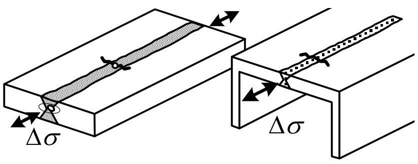


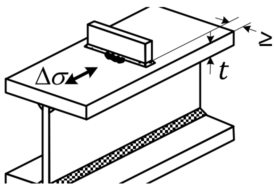
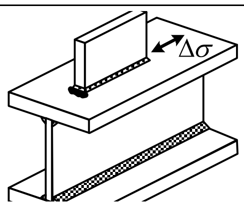
Figure 5.19 - Example of fatigue strength curves

L) Examples of detail categories

Part of Table J.5 in EN 1999-1-3

5.1	63-4,3	At weld discontinuity 	Full penetration butt weld. Weld caps ground flush	Nominal stress at initiation site		Continuous automatic welding	B	C
5.2	56-4,3						C	C

Part of Table J.13 in EN 1999-1-3

13.1	23-3,4	 Weld toe	Transverse attachment, thickness < 20 mm, welded on one or both sides	Net section	Stiffening effect of attachment / stress concentration at " hard point" of connection	C	C	For web-to-flange fillet welds, see Table J.5, type no. 5.4 or 5.5
13.2	18-3,4	 Weld toe	Longitudinal attachment length ≥ 100 mm, welded on all sides					

Chapter 6 – Examples of applications by using Eurocode 9

The low value of the elastic modulus *E* has a big influence on the deformations of aluminium structures, which means that the deformation at the serviceability limit state is often governing the design. The designer should not forget to check the deformation before the final check of the strength at the ultimate limit state.

References to *clause, Table or formula in Italic* refer to EN 1999-1-1 (2007 version, as amended in 2009). For reference to another Eurocode, *EN number* is also given in Italic.

Example 1: Vertical deflection and resistance of a simply supported beam

Design data

A simply supported glass roof beam of span 4.2 m (Figure 6.1) is subjected to the following un-factored loads:

Dead load	8.6 kN/m
Imposed roof load	10.5 kN/m
Snow load	6.8 kN/m

Serviceability limit state

Design an I-beam for the vertical deflection limit = span/360 (beam carrying plaster or other brittle finish).

From *clause 5.2.5* we find $E = 70000$ MPa (N/mm^2).

The characteristic combination of action according to *EN 1990* is used with all partial factors = 1.

$$E_d = 'G_k' + 'Q_{k,1}' + '\psi_{0,2} Q_{k,2}'$$

where the permanent action G_k is unfavourable and the imposed roof load $Q_{k,1}$ is the leading variable action.

From *EN 1991*, for snow loads at altitude > 1000 m, $\psi_0 = 0.7$. Therefore

$$q = 8.6 + 10.5 + 0.7 \cdot 6.8 = 23.9 \text{ kN/m}$$

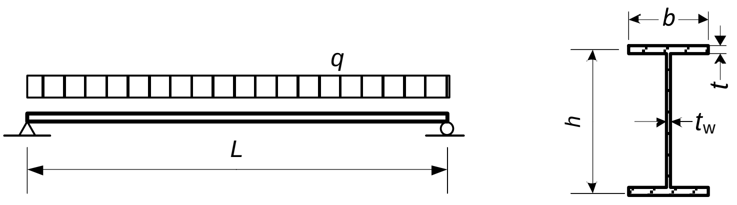


Figure 6.1: Simply supported beam and beam cross-section

Under a uniform distributed load, the maximum deflection *w* of a simply supported beam is given by

$$w = \frac{5}{384}\frac{qL^4}{EI}$$

from which the required second moment of area is solved

$$I_{\text{req}} = \frac{5}{384}\frac{qL^4}{Ew}$$

For a deflection limit of span/360 for brittle finish we get

$$I_{\text{req}} = \frac{5}{384}\frac{qL^4}{Ew} = \frac{5}{384}\frac{23.9 \cdot 4200^4}{70000 \cdot (4200/360)} = 1.18 \cdot 10^8\ \text{mm}^4$$

As there are no standard I-beams in aluminium, the flange slenderness is chosen to avoid reduction due to local buckling. For EN-AW 6063-T6, $f_o = 160\ \text{MPa}$, class A, without weld, the slenderness limit is

$\beta_3 = 6\varepsilon = 7.5$ for $\varepsilon = \sqrt{250/160} = 1.25$.

Choose flange slenderness $\frac{b}{t} = 2\beta_3$ =15.0.

An approximate formula for the second moment of area is

$I \approx 0.58 A_f h^2$, where $A_f = bt$ from which, for a chosen value h = 308 mm of the beam depth,

$$A_f = \frac{I_{\text{req}}}{0.58h^2} = \frac{1.18 \cdot 10^8}{0.58 \cdot 308^2} = 2150\ \text{mm}^2$$

As $b = 2\beta_3 t = 15.0t$, it results

$t = \sqrt{\frac{A_f}{2\beta_3}} = \sqrt{\frac{5150}{15}} = 12.0\ \text{mm}$ and $b = 2\beta_3 t = 15 \cdot 12.0 = 180\ \text{mm}$.

Chose web thickness $t_w = 6\ \text{mm}$ and check the resulting second moment of area:

$I = 2bt\left(\frac{h}{2}\right)^2 + \frac{t_w h^3}{12} = 2 \cdot 180 \cdot 12 \cdot 154^2 + \frac{6 \cdot 308^3}{12} = 1.17 \cdot 10^8\ \text{mm}^4 \approx I_{\text{req}}$, accepted.

As h is the distance between the centres of the flanges, then the total height will be 308 + 12 = 320 mm.

Ultimate limit state

Check the resistance with the simplified analysis method according to *clause 8.4*.

The design load combination according to *EN 1990* is used

$E_d = {'}\gamma_G G_k{'} + {'}\gamma_Q Q_{k,1}{'} + {'}\gamma_Q \psi_{0,2} Q_{\text{k},2}{'}$, where the partial factors are γ_G=1,2 and γ_Q=1,5.

It results

$$q_{Ed} = 1.2 \cdot 8.6 + 1.5 \cdot 10.5 + 1.5 \cdot 0.7 \cdot 6.8 = 33.2\ \text{kN/m}$$

$$M_{\text{Ed}} = 22.2 \cdot 4.2^2/8 = 49.0\ \text{kNm}$$

As $\frac{b}{2t} = \frac{180}{2 \cdot 12} = 7.5 = \beta_3$ cross-section class is 3 and ρ_{min} =1.0

$M_{Rd} = \frac{\rho_{min} W_{\text{el}} f_o}{\gamma_{\text{M1}}} = \frac{1.0 \cdot 1.17 \cdot 10^8}{\frac{308}{2}}\frac{160}{1.1} = 1.11 \cdot 10^8\ \text{Nmm} = 111\ \text{kNm} > M_{\text{Ed}} = 49.0\ \text{kNm}$, passed.

Example 2: Bending moment resistance of a class 4 cross-section

Aluminium profiles may have very different and complicated shapes. Examples of a series of profiles used in curtain walls and windows are shown in Figure 6.2.

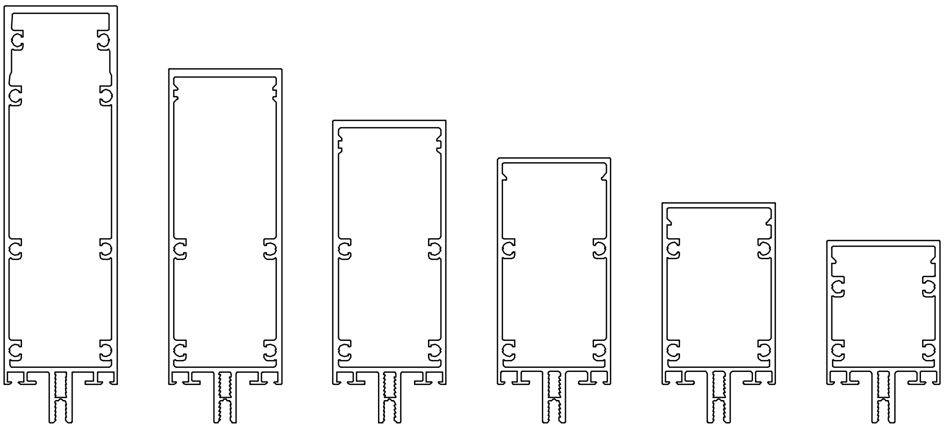
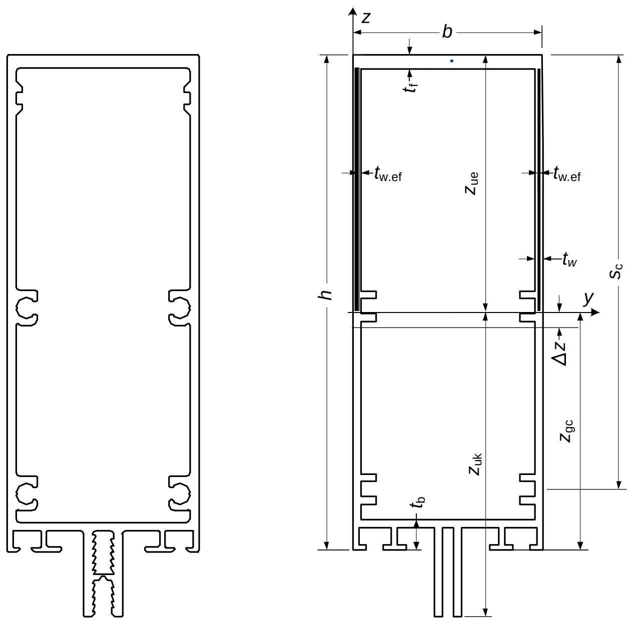


Figure 6.2: Examples of typical aluminium profiles for curtain walls and windows

Design data

The cross-section may have bolt channels and screw grooves, which may work as stiffeners of the slender parts of the cross-section. Number 3 profile of Figure 6.2 is chosen as an example of a class 4 cross-section for bending.



a) actual cross-section (b) effective cross-section (simplified)

Figure 6.3: Extruded aluminium profiles: (a) actual cross-section; (b) effective cross-section

The aim is to calculate the major axis bending moment resistance for the upper flange in compression. The material is EN-AW 6063-T6 which, according to *Table 3.2*, belong to Buckling Class A and has a proof strength $f_o = 160$ MPa . The partial factor of strength is $\gamma_{M1} = 1.1$.

The cross-section is complicated. However, most CAD programs give the "ordinary" cross-section constants: $A = 1073 \text{ mm}^2$, $I_y = 3.00 \cdot 10^6 \text{ mm}^4$, $W_{y,\text{el}} = 3.93 \cdot 10^4 \text{ mm}^3$ and $z_{\text{gc}} = 72.1$ mm.

To check local buckling, also the following geometrical data are necessary:

$b = 50$ mm, $t_f = 3.5$ mm, $h = 140$ mm, $t_w = 2$ mm, $z_{\text{ue}} = 77.5$ mm, and for the bottom flange $t_b = 7$ mm and $h_n = 17$ mm, see Figure 6.3.

The influence of the web stiffeners (screw ports) close to the centre of the webs is small and omitted when calculating the major axis moment resistance. Contrary, for axial force they may have noticeable influence.

Cross-section classification

In *Table 6.2*, $\varepsilon = \sqrt{250/160} = 1.25$.

For the **flange**, *clause 6.1.4.3(1)*: $\beta_f = (b - 2t_w)/t_f = (50 - 2 \cdot 2)/3.5 = 13.1$.

Slenderness limit, *Table 6.2* : $\beta_2 = 16\varepsilon = 16 \cdot 1.25 = 20$ and $\beta_1 = 11\varepsilon = 11 \cdot 1.25 = 13.8$;

flange is Class 1.

For the **web,** *clause 6.1.4.3(1)*, the stiffeners are neglected. As the tension flange is very much stiffened, the web is supposed to start at the middle of the bottom screw port. Then

$$\psi = -\frac{s_c - z_{\text{ue}}}{z_{\text{ue}} - t_f} = -\frac{125 - 77.5}{77.5 - 3.5} = -0.642$$

$$\beta_w = (0.7 + 0.3\psi)(s_c - t_f)/t_w = (0.7 + 0.3 \cdot (-0.642))(125 - 3.5)/2 = 30.8$$

$\beta_2 = 16\varepsilon = 16 \cdot 1.25 = 20$ and $\beta_3 = 22\varepsilon = 22 \cdot 1.25 = 27.5$;

web is Class 4.

Shape factor

The section classification is Class 4 and the shape factor is then based on the effective cross-section according to *Table 6.4* in *clause 6.2.5.1*. As the compression flange is Class 1, only the web thickness needs to be reduced.

For the material Buckling Class A according to *Table 3.2b,* the coefficients in formula (6.12) are $C_1 = 32$ and $C_2 = 220$, so

$$\rho_c = min\left[C_1\frac{\varepsilon}{\beta_w} - C_2\left(\frac{\varepsilon}{\beta_w}\right)^2, 1.0\right] = 32\frac{1.25}{30.8} - 220\left(\frac{1.25}{30.8}\right)^2 = 0.936. \qquad (6.12)$$

The effective thickness of the compression part of the webs is

$$t_{\text{w,ef}} = \rho_c t_w = 0.936 \cdot 2 = 1.872,$$

where the width of this part is

$$b_c = h - t_f - z_{gc} = 140 - 3.5 - 78.1 = 58.4 \text{ mm}.$$

Again, the CAD program is used. The section modulus for the upper edge (ue) and bottom edge (be) of the section are found to be almost identical

$W_{\text{ue}} = 3.828 \cdot 10^4 \text{ mm}^3$ and $W_{\text{be}} = 3.827 \cdot 10^4 \text{ mm}^3$.

Sometimes an iteration procedure is needed to assure that the width of the compression part of the web coincides with the calculated neutral axis for the effective cross-section, see *clause 6.2.5.2(2)* and *clause 6.7.2(5).* However, this is not necessary in this case as the overall cross-section is almost symmetric. The shape factor is then

$$\alpha_4 = \frac{min(W_{\text{ue}}, W_{\text{be}})}{W_{\text{el}}} = \frac{3.827}{3.933} = 0.973.$$

Bending moment resistance

Bending moment resistance is according to *(6.25)*

$$M_{\text{o,Rd}} = \alpha_4 W_{\text{el}} f_o / \gamma_{\text{M1}} = W_{\text{eff}} f_o / \gamma_{\text{M1}} = 3.827 \cdot 10^4 \cdot 160/1.1 = 5.57 \text{ kNm} \qquad (6.25)$$

Example 3: Bending moment resistance of a welded member with a transverse weld

Design data

Two extruded channel sections are welded together to a rectangular hollow section according to Figure 6.4. Calculate the major axis bending moment resistance for

a) section without transverse weld;
b) section with transverse butt welds across part of the web.

The material is EN-AW 6082-T6, which, according to *Table 3.2b*, belong to Buckling Class A and has a proof strength f_{o} = 260 MPa. The partial factor of strength is $\gamma_{\text{M1}} = 1.1$.

Width *b* = 100 mm, height *h* = 300 mm, flange thickness $t_{\text{f}} = 10$ mm and web thickness $t_{\text{w}} = 6$ mm.

Resistance of section without transverse weld

Cross-section classification, *clause 6.1.4*

Classification based on limits in *Table 6.4* for BC A, *with welds* gives cross-section class 3 for the flange. For the web for BC A, *without welds*, cross-section class is also 3.

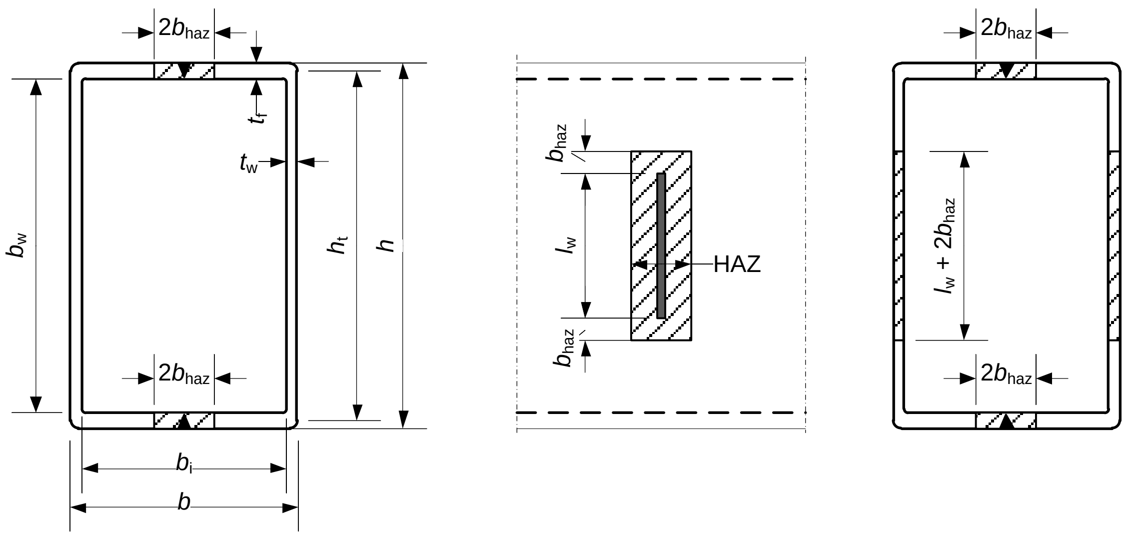


Figure 6.4: Welded aluminium profile

Heat affected zones, *clause 6.1.6*

The reduction factor for the strength in HAZ is found in *Table 3.2b*, and the extent is found in *clause 6.1.6.3*

$\rho_{\mathrm{o,haz}} = 0{,}48$ and $b_{\mathrm{haz}} = 30$ mm for $t_{\mathrm{f}} = 10$ mm.

The effective thickness within HAZ will be

$t_{\mathrm{haz}} = \rho_{\mathrm{o,haz}}\, t_f = 0.48 \cdot 10 = 4.8 \text{ mm}.$

The elastic section modulus allowing for HAZ is found by deleting the difference between the flange thickness and the effective thickness within the width $2b_{\mathrm{haz}}$ from the gross cross-section.

For the gross cross-section $\quad I_y = 8.926 \cdot 10^7 \text{ mm}^4$

For the reduced cross-section $\quad I_{\mathrm{y,haz}} = I_y - 2b_{\mathrm{haz}}(t_f - t_{\mathrm{haz}})2\left(\frac{h_t}{2}\right)^2 = 7.61 \cdot 10^7 \text{ mm}^4$

The section modulus $\quad W_{\mathrm{el,haz}} = I_{\mathrm{y,haz}} 2/h = 5.08 \cdot 10^5 \text{ mm}^3$

The plastic section modulus allowing for HAZ

$$W_{\mathrm{pl,haz}} = \frac{1}{4}\left(bh^2 - (b - 2t_w)(h - 2t_f)^2\right) - 2b_{\mathrm{haz}}(t_f - t_{\mathrm{haz}})h_t = 6.09 \cdot 10^5 \text{ mm}^3$$

Shape factor, *clause 6.2.5.1*

For cross-section class 3 the shape factor = 1.0 or may alternatively be calculated using *(6.27).* As the web area is a large part of the cross-section, formula *(6.27)* in *clause 6.2.5.1* is used.

$$\alpha_{3,\mathrm{w}} = \left[\frac{W_{\mathrm{el,haz}}}{W_{\mathrm{el}}} + \left(\frac{\beta_3 - \beta}{\beta_3 - \beta_2}\right)\left(\frac{W_{\mathrm{pl,haz}} - W_{\mathrm{el,haz}}}{W_{\mathrm{el}}}\right)\right] \qquad (6.27)$$

where the cross-section part with the smallest value of the ratio $(\beta_3 - \beta)/(\beta_3 - \beta_2)$ is the critical part. The web is decisive so

$$\alpha_{3,\mathrm{w}} = 0.937\,. \qquad (6.27)$$

Bending moment resistance

Bending moment resistance is according to *(6.25)*

$$M_{\mathrm{o,Rd}} = \alpha_{3,\mathrm{w}} W_{\mathrm{el}} f_o / \gamma_{\mathrm{M1}} = 132\ \mathrm{kNm} \qquad (6.25)$$

Resistance of section with transverse weld, clause 6.2.5.1

The resistance in section with the transverse weld is given by formula (6.24).

$$M_{\mathrm{u,Rd}} = W_{\mathrm{u,eff}} f_u / \gamma_{\mathrm{M2}}\,, \qquad (6.24)$$

where $W_{\mathrm{u,eff}}$ is effective section modulus, obtained using a reduced thickness $\rho_c t$ for class 4 parts and reduced thickness $\rho_{\mathrm{u,haz}} t$ for the HAZ material, whichever is smaller.

Cross-section classification and effective thickness are the same as for the section without transverse weld (Class 3), so there is no reduction due to local buckling.

The reduction factor for the ultimate strength in HAZ is according to Table 3.1 for EN-AW 6082-T6

$$\rho_{\mathrm{u,haz}} = 0.60\,.$$

So, the section modulus with allowance for HAZ, due to longitudinal weld and localized transverse weld, is

$$I_{\mathrm{u,eff}} = I_y - 2b_{\mathrm{haz}} t_f (1 - \rho_{\mathrm{u,haz}}) 2 \left(\frac{h_t}{2}\right)^2 - (1 - \rho_{\mathrm{u,haz}}) 2 t_w (l_w + 2b_{haz})^3 / 12 = 6.592 \cdot 10^7\ \mathrm{mm}^4$$

$$W_{\mathrm{u,eff}} = \frac{I_{\mathrm{u,eff}} \cdot 2}{h} = 5.73 \cdot 10^5\ \mathrm{mm}^3.$$

Bending moment resistance at section with transverse weld

Bending moment resistance is according to *(6.24)*:

$$M_{\mathrm{u,Rd}} = W_{\mathrm{u,eff}} f_u / \gamma_{\mathrm{M2}} = 142\ \mathrm{kNm}\,. \qquad (6.24)$$

This is actually larger than the resistance of the member with longitudinal welds only, which is $M_{\mathrm{o,Rd}} = 132\ \mathrm{kNm}$. Therefore, in this case the HAZ in the welds does not reduce the bending moment resistance of the member.

The strength in the weld, according to *Table 8.8* for filler metal 5356, is $f_w = 210\ \mathrm{MPa}$, which is larger than the strength in HAZ, thus not critical.

Example 4: Lateral-torsional buckling of member in bi-axes bending and compression

Design data

The beam-column according to Figure 6.5 is loaded by an eccentric axial load, main axis eccentricity at one end and minor axis eccentricity at both ends. At the top, the load is inserted via a rigid rectangular hollow section beam. There are simply support at the load points A and C. In this example lateral-torsional buckling is checked according to ***6.3.3.2**, formula (6.63)* and flexural buckling according to ***6.3.3.1**, formula (6.59).*

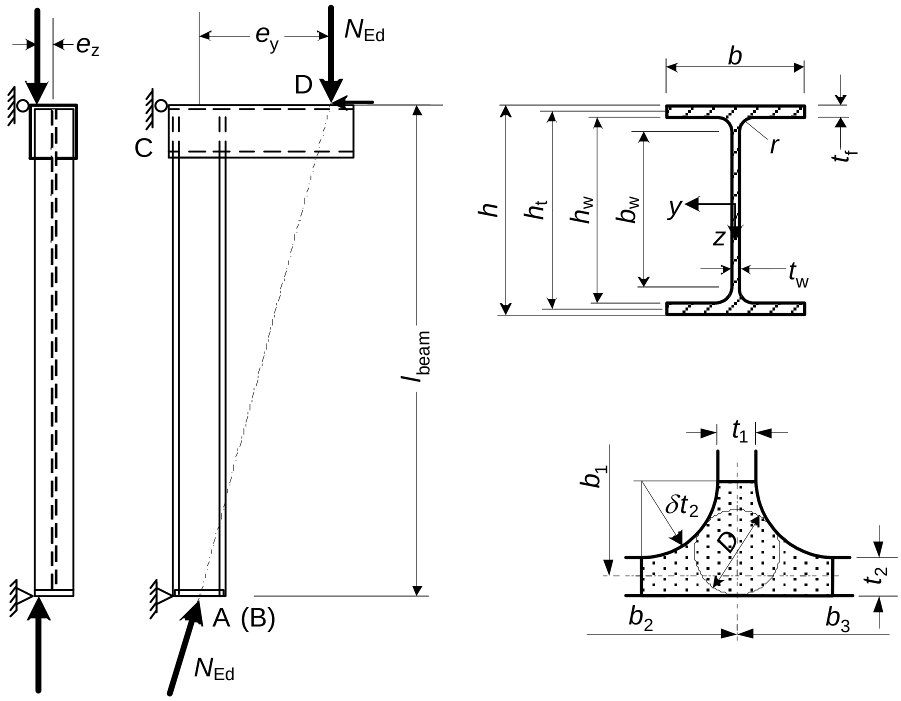


Figure 6.5: General arrangement, loading and cross-section

- Geometrical data:

Beam length	l_{beam} = 2500 mm	
Eccentricity at the top	e_y = 400 mm,	e_z = 30 mm
Eccentricity at the bottom	e_y = 0 mm,	e_z = 30 mm

- Cross-section data:

Section height	h = 200 mm,	flange width b = 100 mm	
Web thickness	t_w = 6 mm,	flange thickness	t_f = 9 mm
Fillet radius	r = 14 mm		

Web height $b_w = h - 2t_f - 2r = 154 \text{ mm}$

Material EN-AW 6082-T6 f_o = 260 MPa

Partial safety factor $\gamma_{\mathrm{M1}} = 1.1$

- External forces:

Axial compression force $N_{\mathrm{Ed}} = 60\mathrm{kN}$

Bending moment at end C $M_{\mathrm{y,Ed}} = N_{\mathrm{Ed}} e_y = 60 \cdot 0.4 = 24.0\mathrm{kNm}$

Bending moment at A and C $M_{\mathrm{z,Ed}} = N_{\mathrm{Ed}} e_z = 60 \cdot 0.03 = 1.8\mathrm{kNm}$

Cross-section classification under axial compression (*clause 6.1.4*)

The classification according to clause 6.1.4 gives class 2 for the flange and class 4 for the web, which means that in compression, the overall cross-section class is 4. The resistance is therefore based on the effective cross-section for the member in compression.

Cross-section classification for y–y axis bending (*clause 6.1.4*)

In *y–y* axis bending, the overall cross-section classification is Class 2. The resistance is therefore based on the plastic section modulus of the member.

Cross-section class under z–z axis bending (*clause 6.1.4*)

The web is in the neutral axis so, in *z–z* axis bending, the overall cross-section classification is Class 1. The resistance is based on the plastic section modulus of the member.

Design resistance for y–y axis bending (*clause 6.2.5*)

Although the resistance is based on the plastic section modulus, the elastic section modulus is needed to calculate the shape factors and the exponents in the interaction formulae. If the fillets are omitted, then the elastic section modulus is

$$W_{\mathrm{el},y} = I_y/(h/2) = 2.074 \cdot 10^5 \mathrm{mm}^3$$

Usually the plastic section modulus is not given by the CAD program. Including fillets and using the notations (see Figure 6.5). Being

$h_t = h - t_f = 191\mathrm{mm}$ and $h_w = h - 2t_f = 182\mathrm{mm}$

it results in

$$W_{\mathrm{pl},y} = bt_f h_t + \frac{1}{4} t_w h_w^2 + 2r^2(h_w - r) - \pi r^2 \left[h_w - 2r\left(1 - \frac{4}{3\pi}\right)\right]\frac{1}{2} = 2.364 \cdot 10^5 \mathrm{mm}^3.$$

The shape factor is, therefore:

$\alpha_y = W_{\text{pl},y}/W_{\text{el},y} = 1{,}140$

and the resistance for *y–y* axis bending

$M_{y,\text{Rd}} = \alpha_y W_{\text{el},y} f_o/\gamma_{\text{M1}} = 1.140 \cdot 2.074 \cdot 10^5 \cdot 260/1.1 = 55.9\text{kNm}$

Design resistance for z–z axis bending (*clause 6.2.5*)

If the fillets are omitted, then (including fillets $I_z = 1.510 \cdot 10^6 \text{mm}^4$)

$$I_z = \frac{1}{12}\left[2t_f b^3 + h_w t_w^3\right] = 1.503 \cdot 10^6 \text{mm}^4.$$

The elastic section modulus is $W_{\text{el},z} = I_z/(b/2) = 3.020 \cdot 10^4 \text{mm}^3$.

Although the influence of the fillets can be neglected, it is included here

$$W_{\text{pl},z} = \frac{1}{4}2t_f b^2 + \frac{1}{4}h_w t_w^2 + 2r^2(t_w + r) - \pi r^2\left[t_w + 2r\left(1 - \frac{4}{3\pi}\right)\right]\frac{1}{2} = 4.767 \cdot 10^4 \text{mm}^3$$

The shape factor and the resistance for *z–z* axis bending are:

$\alpha_z = W_{\text{pl},z}/W_{\text{el},z} = 1.578$

$M_{z,\text{Rd}} = f_o \alpha_z W_{\text{el},z}/\gamma_{\text{M1}} = 11.3\text{kNm}.$

Axial force resistance for y–y axis buckling (*clause 6.3.1*)

To calculate the effective cross-section area, the gross cross-section area is first calculated and then the reduction due to local buckling is made.

$A_{\text{gr}} = bh - (b - t_w) \cdot (h - 2t_f) + r^2(4 - \pi) = 3060 \text{ mm}^2$

$A_{\text{eff}} = A_{\text{gr}} - b_w(t_w - \rho_c t_w) = 2969 \text{ mm}^2.$

The buckling length is $l_{\text{cr,y}} = 2500 \text{ mm}$,

so, the buckling load and the slenderness are given by

$N_{\text{cr},y} = \frac{\pi^2 E I_y}{l_{\text{cr},y}^2} = 2293$ kN and $\bar{\lambda}_y = \sqrt{\frac{A_{\text{eff}} f_o}{N_{\text{cr},y}}} = 0.580$ (6.51)

The reduction factor for flexural buckling with $\alpha = 0.2$ and $\bar{\lambda}_0 = 0.1$, from *Table 6.6* for Buckling Class A, is $\chi_y = 0.880$.

Buckling resistance according to *(6.49)* for no welds and leaving ω_x is

$N_{y,\text{b,Rd}} = \chi_y f_o A_{\text{eff}}/\gamma_{\text{M1}} = 618 \text{ kN}.$ (6.49)

Cross-section resistance is needed in the interaction formulae

$N_{\text{Rd}} = f_o A_{\text{eff}}/\gamma_{\text{M1}} = 702 \text{ kN}.$

Axial force resistance for z–z axis buckling (*clause 6.3.1*)

The buckling length is $l_{\text{cr,y}} = 2520 \text{ mm}$. The buckling resistance according to *(6.49)*, based on $\chi_z = 0.195$ is given by

$N_{z,b,Rd} = \chi_z A_{eff} f_o / \gamma_{M1} = 137\ \mathrm{kN}.$ (6.49)

Lateral-torsional buckling of beam in bending (*clause 6.3.2.1*)

The elastic lateral-torsional buckling load is found in *Annex I*. The torsion constant is found in *Annex J* and the warping constant in *Annex J, Figure J.2 case 5*a, which gives:

$I_w = (h - t_f)^2 I_z / 4 = 191^2 \cdot 1.51 \cdot 10^6 / 4 = 1.377 \cdot 10^{10}\ \mathrm{mm}^6$

The last term in (*J.1*) is not needed so the torsion constant, including the fillets, is

$I_t = \sum bt^3/3 - 0.105 \sum t^4 + \sum \alpha D^4$ (*J1.a*)

where D is found in *Annex G, Figure J.1 case 2:*

$D = ((\delta + 1)^2 + (\delta + 0.25 t_1/t_2) t_1/t_2)(t_2/(2\delta + 1))$

With $t_1 = t_w = 6\ \mathrm{mm}$ and $t_2 = t_f = 9\ \mathrm{mm}$ then $\delta = r / t_2 = 14/9 = 1.556$ (see Figure 6.5)

and $\alpha = \frac{(0.10\delta + 0.15)t_1}{t_2} = 0.204$. Therefore:

$D = 16.8\ \mathrm{mm}.$

Now, from equation *(J1.a)*, the value of I_t , for two flanges with fillets, is $I_t = 9.402 \cdot 10^4\ \mathrm{mm}^4$.

It is important to notice that:

if the fillets are omitted then $I_t = 6.24 \cdot 10^4\ mm^4$; by adding just 5% material, the torsion constant is increased 51% !

The elastic critical moment for lateral-torsional buckling is given by the general formula

$M_{cr} = \mu_{cr} \frac{\pi \sqrt{E I_z G I_t}}{L}$ (*I.2*)

where the relative non-dimensional critical moment μ_{cr} is found in *Annex I*.

Without presenting it in detail, their values are $\mu_{cr} = 2.272$ and $M_{cr} = 46.8\ \mathrm{kNm}.$
The relative slenderness $\bar{\lambda}_{LT}$ and the bending moment resistance is found in *6.3.2,* which gives

$M_{y,b,Rd} = 37.7\ \mathrm{kNm}.$

Interaction formulae

Both flexural buckling according to *clause 6.3.3.1* and lateral-torsional buckling according to *clause 6.3.3.2* need to be checked, see *6.3.3.2(2)*.

For major axis (*y*–axis) bending

$$\left(\frac{\omega_x N_{Ed}}{\chi_y N_{Rd}}\right)^{\eta_y} + \frac{M_{y,Ed}}{M_{y,Rd}} \le 1.00$$ (6.59)

For lateral-torsional buckling

$$\left(\frac{\omega_x N_{\mathrm{Ed}}}{\chi_z N_{\mathrm{Rd}}}\right)^{\eta_z} + \left(\frac{\omega_{\mathrm{xLT}} M_{\mathrm{y,Ed}}}{\chi_{\mathrm{LT}} M_{\mathrm{y,Rd}}}\right)^{\beta_z} + \left(\frac{M_{\mathrm{z,Ed}}}{M_{\mathrm{z,Rd}}}\right)^{\delta_z} \leq 1.00 \tag{6.63}$$

To shorten the formulae, the following notations are introduced

$K_0 = \frac{N_{\mathrm{Ed}}}{N_{\mathrm{Rd}}} = \frac{60}{702} = 0.0855$ and $B_0 = \frac{M_{y,\mathrm{Ed}}}{M_{y,\mathrm{Rd}}} = \frac{24}{55.9} = 0.430.$

The exponents η_y, η_z and δ_z in the interaction formulae are given in 6.3.*3.1(1)* and *6.3.3.2(1)*.

Conservatively, all exponents may be taken as 0.8. To show the complete procedure, the formulae for the exponents are shown.

$\eta_y = \chi_y \min\{\alpha_y^2; 1{,}56)\}$ but $\eta_y \geq 1.0 \quad \rightarrow \eta_y = 1.14$

$\eta_z = \chi_z \min\{\alpha_z^2; 1{,}56)\}$ but $\eta_z \geq 1.0 \quad \rightarrow \eta_z = 0.80$

$\beta_z = \chi_z \min\{\alpha_z^2; 1{,}56)\}$ but $\beta_z \geq 1.0 \quad \rightarrow \beta_z = 1.56$

$\delta_z = \chi_z \min\{\alpha_z^2; 1{,}56\}$ but $\delta_z \geq 0{,}8 \quad \rightarrow \delta_z = 0.80$

Lateral-torsional buckling check (*clause 6.3.3.2*)

The formula for defining the design section is according to *6.3.3.5(2)*

$$cos\left(\frac{x_s\pi}{l_c}\right) = \frac{(M_{\mathrm{Ed,1}}-M_{\mathrm{Ed,2}})}{M_{\mathrm{Rd}}} \cdot \frac{N_{\mathrm{Rd}}}{N_{\mathrm{Ed}}} \cdot \frac{1}{\pi(1/\chi-1)}\text{, but } x_{\mathrm{s}} \geq 0 \tag{6.71}$$

where $l_c = l_{\mathrm{cr},z}$, $M_{\mathrm{Ed,1}} = M_{y,\mathrm{Ed}}$, $M_{\mathrm{Ed,2}} = \psi_y M_{y,\mathrm{Ed}} = 0$ and the ratio between the moments at the ends is $\psi_y = 0$.

Formula *(6.71)* can now be evaluated

$$cos\left(\frac{x_s\pi}{l_{\mathrm{cr},z}}\right) = \frac{B_0}{K_0}\frac{(1-0)}{\pi(1/\chi_z-1)} = 0.387 \text{ but } x_{\mathrm{s}} \geq 0$$

$\frac{x_s\pi}{l_{\mathrm{cr},z}} = \mathrm{acos}(0.387) = 1.173\ \mathrm{rad}$ and $x_s = 1.173 \cdot 2500/\pi = 934\ \mathrm{mm}.$

The interaction formulae ω_x and $\omega_{x\mathrm{LT}}$ according to *6.3.3.5(1)* are

$$\omega_{\mathrm{x}} = 1/\left(\chi + (1-\chi)\sin\frac{\pi x_{\mathrm{s}}}{l_{\mathrm{c}}}\right) \tag{B.1}$$

$$\omega_{\mathrm{xLT}} = 1/\left(\chi_{\mathrm{LT}} + (1-\chi_{\mathrm{LT}})\sin\frac{\pi x_{\mathrm{s}}}{l_{\mathrm{c}}}\right) \tag{6.70}$$

The three terms in the interaction formula (6.63) can be evaluated separately.

$$K_z = \left(\frac{K_0}{\omega_z \chi_z}\right)^{\eta_c} = \left(\frac{K_0}{\chi_z}\left(\chi_z + (1-\chi_z)\sin\frac{\pi x_s}{l_{cr,z}}\right)\right)^{\eta_z} = 0.491$$

$$B_y = \left(\frac{B_x}{\omega_{xLT}\chi_{LT}}\right)^{\beta_z} = \left(\frac{B_0}{\chi_{LT}}\left(1-(1-\psi_y)\frac{x_s}{l_{cr,y}}\right)\cdot\left(\chi_{LT} + (1-\chi_{LT})\sin\frac{\pi x_s}{l_c}\right)\right)^{\beta_z} = 0.229$$

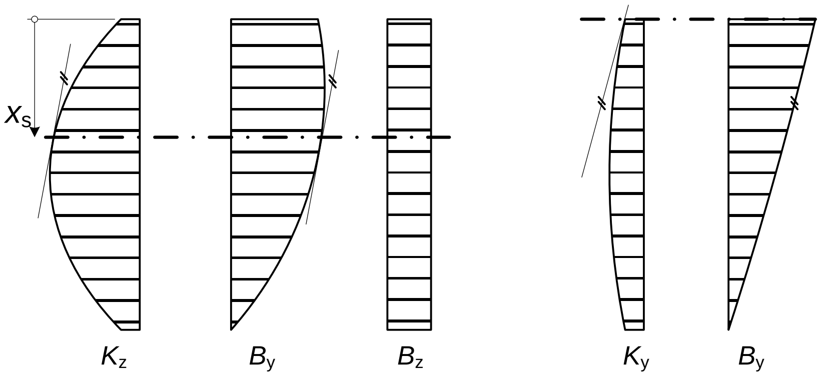
$$B_z = \left(\frac{M_{z,Ed}}{M_{z,Rd}}\right)^{\delta_z} = \left(\frac{1.80}{11.3}\right)^{0.8} = 0.231$$

Being

$K_z + B_y + B_z = 0.491 + 0.229 + 0.231 = 0.951 < 1,$

lateral-torsional buckling check is accepted.

For lateral-torsional buckling, the design section is close to the centre of the beam due to large second order bending moment (Figure 6.6 a). Figure 6.6 shows the behavioural differences between lateral-torsional buckling (a) and flexural buckling (b).



(a) Lateral-torsional buckling (b) Flexural buckling

Figure 6.6: ***K*- and *B*-diagram and design sections (dash-dotted)**

Flexural buckling check (*6.3.3.1*)

The formula *(6.71)* for defining the design section according to *6.3.3.5(2)* for *y–y*–axis buckling is evaluated as for z-z-axis buckling, except that $l_{cr,z}$ is replaced by $l_{cr,y}$ and χ_z is replaced by χ_y. As the reduction factor χ_y is close to 1.0 (0.880) the top section will be the design section. See Figure 6.6 (b).

Formula (6.59) should then be checked with $\eta_z = \eta_y$ and $\omega_x = \chi_y$.

Being

$K_y + B_y = 0.060 + 0.430 = 0.490 < 1,$

flexural buckling check is accepted.

Example 5: Welded connection between diagonal and chord member

Design data

Referring to Figure 6.7, calculate the tensile resistance of a welded connection of an L-diagonal into a chord member. The material in the diagonal and chord is EN-AW 6005A, with ultimate strength $f_u = 270$ MPa and $\rho_{\mathrm{u,haz}} = 0.61$. Filler metal is 5356, having $f_w = 180$ MPa according to *Table 8.8*. The angle between the diagonal and the chord is $\beta_0 = 42°$. The distance from edge to centre of gravity of the angle section 57 mm×6 mm is $e_1 = 17$ mm and thickness $t_1 = 6$ mm.

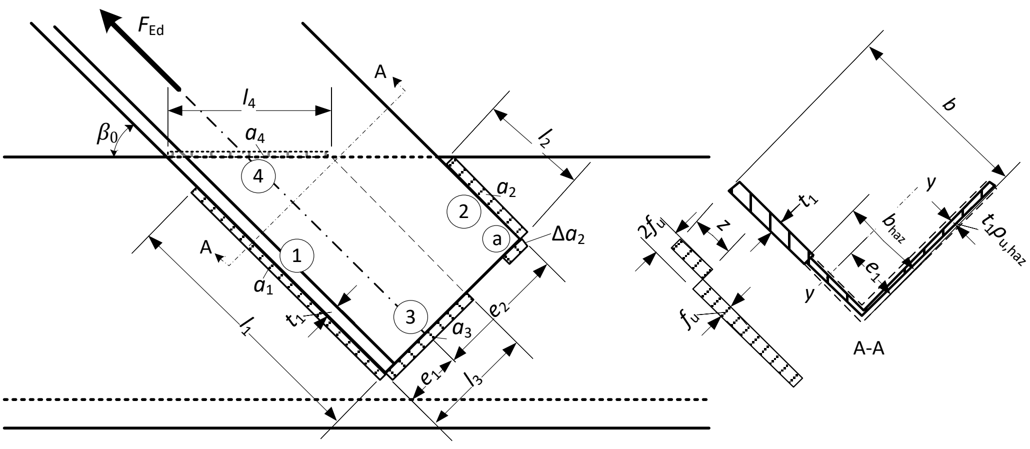


Figure 6.7: Connection of diagonal into chord of build-up member

The resistance of the four welds is derived with formula *(8.31)* and the corresponding values are given in Table 6.1, where also the moment due to the eccentricity is calculated. By reducing the weld 1 to 75 mm, the moment value is practically 0. The resistance of the oblique weld 4 is derived from formula (*8.31*) and the stresses according to Figure 6.8. The result is a factor

$f(\beta) = (2\sin^2\beta + 3\cos^2\beta)^{-1}$

for $f_{\mathrm{w}}/\gamma_{\mathrm{Mw}}$ to get the strength of the weld, where $\beta = \beta_0$ in Figure 6.8.

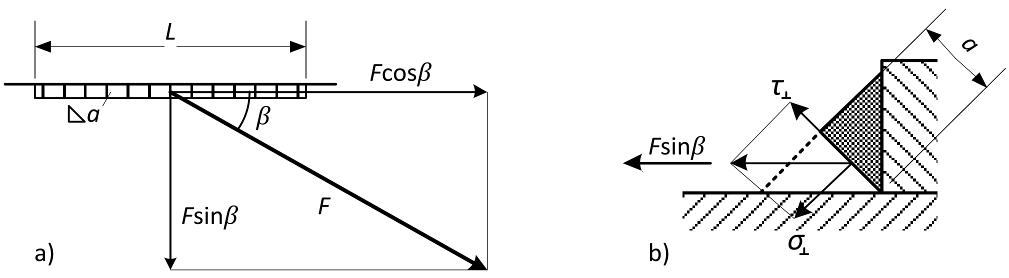


Figure 6.8: Stresses in oblique fillet weld

In practice the welds 3 and 4 are extended over the whole width of the angle section. Alternatively weld 2 is completed with a weld Δa_2 (a) and the length of weld 1 is increased.

Weld	*L* /mm	*A* /mm	β_0	*E* /mm	f_w/γ_{Mw} MPa	*f*(β)	F_{Rd} /kN	M_e /kNm
1	75	3	0	17	144	0.577	18,7	0,318
2	32	3	0	-40	144	0.577	8,0	-0,319
3	34	3	90	0	144	0.707	10,4	0
4	51	3	42	0	144	0.626	13,8	0
						Sum	50.8	-0.001

Table 6.1: Weld data

From Table 6.1 the sum of the resistance of the welds is $F_{\mathrm{Rdw}} = 50.8\ \mathrm{kN}$.
The resistance in the heat affected zone HAZ is based on the net section area of the angle section, considering HAZ extended all over the flange in the joint plane and for b_{haz} in the perpendicular flange, being the extent $b_{\mathrm{haz}} = 25\ \mathrm{mm}$ according to *clause 6.1.6.3* and the reduction factor $\rho_{\mathrm{u,haz}} = 0.61$. The net section, according to *6.2.3,* is then
$A_{\mathrm{net}} = t(b - b_{\mathrm{haz}}) + t(b + b_{\mathrm{haz}} - t)\rho_{\mathrm{u,haz}} = 6 \cdot (57 - 25) + 6 \cdot (57 + 25 - 6) \cdot 0.61 = 470\ \mathrm{mm}^2$.
It is supposed that the tension force is acting in the plane of the joint. Then a bending moment is acting on the angle section, which is carried by plastic distribution of stresses in the cross-section according to Figure 6.7. The compression part *z* is derived in such a way that the moment in the plane of the joint (mid of angle leg) is zero (note 2*tz* on the right end side of the equation). Then

$$t(b - t/2)^2/2 - t(b_{\mathrm{haz}} - t/2)^2/2 \cdot \left(1 - \rho_{\mathrm{u,haz}}\right) = 2tz(b - z/2)$$

from which

$$z = b - \sqrt{b^2 - (b - t/2)^2/2 + (b_{\mathrm{haz}} - t/2)^2\left(1 - \rho_{\mathrm{u,haz}}\right)/2} = 13.6\ \mathrm{mm}.$$

The design resistance is therefore:

$$F_{\mathrm{Rd}} = A_{\mathrm{net}}\frac{f_u}{\gamma_{\mathrm{M2}}} - tz\frac{2f_u}{\gamma_{\mathrm{M2}}} = 470 \cdot \frac{270}{1.25} - 6 \cdot 13.6 \cdot \frac{2 \cdot 270}{1.25} = 66.3\ \mathrm{kN}$$

which is larger than the resistance of the welds $F_{\mathrm{Rdw}} = 50.8\ \mathrm{kN}$ (Table 6.1).

Example 6: Resistance of equivalent T-stub

Design data

Calculate the resistance of a T-sub corresponding to a pair of bolts within c according to Figure 6.9.

Material properties and dimensions are the following:

EN-AW 6005A, Table 3.2a: $f_{\mathrm{o}} = 200\ \mathrm{MPa}$ and $f_{\mathrm{u}} = 250\ \mathrm{MPa}$

HAZ properties: $f_{\mathrm{o,haz}} = 115\ \mathrm{MPa}$ and $f_{\mathrm{u,haz}} = 165\ \mathrm{MPa}$

Thickness of flange plate: $t_{\mathrm{f}} = 15\ \mathrm{mm}$

Lever arm: $g = 20\ \mathrm{mm}$

Edge distance: $e_{\min} = 20\ \mathrm{mm}$

Bolt distance: $c = 30\ \text{mm}$

Steel bolt 8.8: $d = 10\ \text{mm}$.

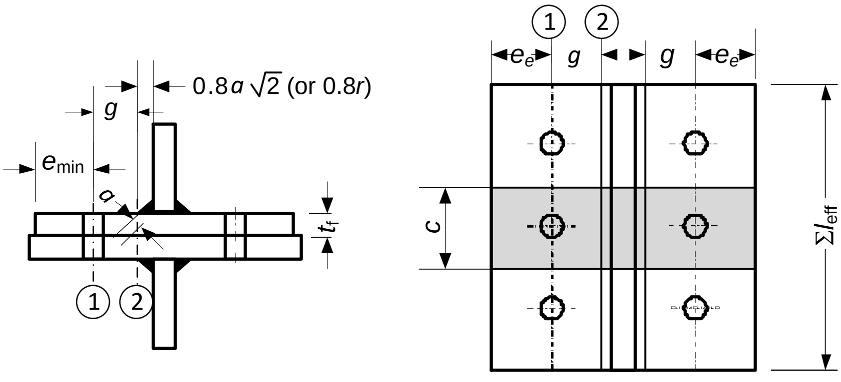


Figure 6.9: Equivalent T-stub

According to *8.10(2)*:

Ultimate strain: $\varepsilon_u = 8\,\%$ from *Table 3.2a*

Elastic strain: $\varepsilon_o = \dfrac{f_o}{E} = \dfrac{200}{70000} = 0.00286$

Strain relation: $\psi = \dfrac{\varepsilon_u - 1.5 \cdot \varepsilon_o}{1.5 \cdot (\varepsilon_u - \varepsilon_o)} = \dfrac{0.08 - 1.5 \cdot 0.00286}{1.5 \cdot (0.08 - 0.00286)} = 0.6543$ *(B.9)*

Stress relation: $\dfrac{1}{k} = \dfrac{f_o}{f_u}\left(1 + \psi \dfrac{f_u - f_o}{f_o}\right) = \dfrac{200}{250}\left(1 + 0.6543 \cdot \dfrac{250 - 200}{200}\right) = 0.931$ *(B.8)*

Edge distance: $e_e = \min(e_{min}, 1.25g) = 20\ \text{mm}$

8.8 steel bolt: d = 10 mm, $d_0 = d + 1\ \text{mm} = 11\ \text{mm}$, $f_y = 640\ \text{MPa}$ and $f_{ub} = 800\ \text{MPa}$

Yield strength: $B_o = 0.9 \dfrac{A_s f_y}{\gamma_{M2}} = 0.9 \cdot \dfrac{58 \cdot 640}{1.25} = 26.7\ \text{kN}$ *(B.10)*

Ultimate strength: $B_u = 0.9 \dfrac{A_s f_{bu}}{\gamma_{M2}} = 0.9 \cdot \dfrac{58 \cdot 800}{1.25} = 33.4\ \text{kN}$ $(= F_{t.Rd})$ *(8.17)*

Effective length 2: $l_{eff,2} = 30\ \text{mm}$

Effective length 1: $l_{eff,1} = l_{eff,2} - d_o = 30 - 11 = 19\ \text{mm}$

Moment resistances in section 1 and 2:

$$M_{u,1} = \frac{1}{4} \cdot t_f^2 \cdot \sum (l_{eff,1} f_u) \cdot \frac{1}{k} \cdot \frac{1}{\gamma_{M2}} = \frac{1}{4} \cdot 15^2 \cdot 19 \cdot 250 \cdot 0.931 \cdot \frac{1}{1.25} = 0.199 \text{ kNm} \qquad (B.5)$$

$$M_{u,2} = \frac{1}{4} \cdot t_f^2 \cdot \sum (l_{eff,2} f_{u,haz}) \cdot \frac{1}{k} \cdot \frac{1}{\gamma_{M2}} = \frac{1}{4} \cdot 15^2 \cdot 30 \cdot 165 \cdot 0.931 \cdot \frac{1}{1.25} = 0.207 \text{ kNm} \qquad (B.6)$$

$$M_{o,2} = \frac{1}{4} \cdot t_f^2 \cdot \sum (l_{eff,2} f_{o,haz}) \cdot \frac{1}{\gamma_{M1}} = \frac{1}{4} \cdot 15^2 \cdot 30 \cdot 115 \cdot \frac{1}{1.1} = 0.176 \text{ kNm} \qquad (B.7)$$

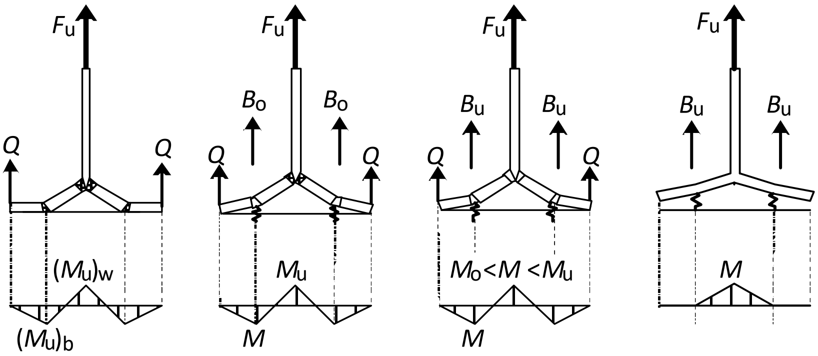


Figure 6.10: Failure modes

If there are no welds in section 2, replace $f_{u,haz}$ with f_u and $f_{o,haz}$ with f_o.

Mode 1: Flange failure by developing two hardening plastic hinges at the web-to-flange connection (w) (= $M_{u,2}$) and two at the bolt location (b) (= $M_{u,1}$) (for g and e_e, see Figure 6.9)

$$F_{u,Rd} = \frac{2(M_{u,2})_w + 2(M_{u,1})_b}{g} = \frac{2 \cdot 207 + 2 \cdot 199}{20} = 40.6 \text{ kN} \qquad (B.1)$$

Mode 2a: Flange failure by developing two hardening plastic hinges with bolt forces at the elastic limit

$$F_{u.Rd} = \frac{2M_{u,2} + n\sum B_o}{g + e_e} = \frac{2 \cdot 236 + 20 \cdot 2 \cdot 26.7}{20 + 20} = 38.5 \text{ kN} \qquad (B.2)$$

Mode 2b: Bolt failure with yielding of the flange at the elastic limit

$$F_{u,Rd} = \frac{2M_{o,2} + n\sum B_u}{g + e_e} = \frac{2 \cdot 176 + 20 \cdot 2 \cdot 33.4}{20 + 20} = 42.2 \text{ kN} \qquad (B.3)$$

Mode 3: Bolt failure

$$F_{u,Rd} = \sum B_u = 2 \cdot 33.4 = 66.8 \text{ kN} \qquad (B.4)$$

Design resistance is the smallest value of the four failure modes

$F_{u,Rd} = 38.5 \text{ kN}$ for Mode 2a.

Chapter 7 – Execution of aluminium structures

A milestone for the harmonisation of the marketing of construction products in the EU was the CPD-Construction Products Directive (EEC) No 89/106, issued 1989 and then replaced in 2011 by the CPR-Construction Products Regulation (EU) No 305/2011. In the framework of the current legislative framework (the CPR), a set of harmonised standards are currently in use to assess and declare the performance of metallic products and ancillaries. Among these we have:

- **EN 1090-1:2009+A1:2011** Execution of steel structures and aluminium structures **- Part 1: Requirements for conformity assessment of structural components;** supported by:
    - **EN 1090-3:2019** Execution of steel structures and aluminium structures - **Part 3: Technical requirements for aluminium structures; 2008, revised and amended 2019;**
    - **EN 1090-5:2017** Execution of steel structures and aluminium structures - **Part 5: Technical requirements for cold-formed structural aluminium elements and cold-formed structures for roof, ceiling, floor and wall applications;**
- **EN 15088:2005** Aluminium and aluminium alloys - Structural products for construction works **- Technical conditions for inspection and delivery.**

The implementation of EN 1090-1 had always been one of the biggest challenges for the manufacturers of aluminium (and steel) structures since, in many countries, manufacturers were not accustomed to follow strict production rules under the control of newly established control authority (the so-called notified bodies).

Concerning the manufacturing of aluminium structures or aluminium structural components, the manufacturer has generally to follow and fulfil the provisions laid down in EN 1090-1, while EN 1090-3 specifies requirements for the execution of aluminium structural components and structures made from rolled sheet, strip and plate, extrusions, cold drawn rod, bar and tube, forgings, castings.

Annex A of EN 1999-1-1 defines four execution classes (EXC), i.e. a product characteristic linked to the manufacturing of a component which defines the engineering effort required to realise specific project parameters. EXC2 is the most common specification; complexity increases as the number rises.

EN 1999-1-1 helps for the determination of the Execution Class:

Consequence class		CC1		CC2		CC3	
Service category		SC1	SC2	SC1	SC2	SC1	SC2
Production category	PC1	EXC1	EXC1	EXC2	EXC3	EXC3 [a)]	EXC3 [a)]
	PC2	EXC1	EXC2	EXC2	EXC3	EXC3 [a)]	EXC4
[a)] EXC4 should be applied to special structures or structures with extreme consequences of a structural failure in the indicated categories as required by national provisions.							
Where: with Service Category differentiation is done between static design (SC1) and fatigue design (SC2) with Production Category differentiation is done between not welded (PC1) and welded components (PC2) with Consequence Class is meant the provisions defined in EN 1990 Table B.1							

Table 7.1: Determination of execution class (Table A.3 in EN 1999-1-1)

Consequences class	Description	Example of buildings and civil engineering works
CC3	High consequence for loss of human life, or economic, social or environmental consequence very great.	Grandstands, public buildings where consequences of failures are high (e.g. a concert hall)
CC2	Medium consequence for loss of human life, or economic, social or environmental consequence considerable.	Residential and office buildings, public buildings where consequences of failure are medium (e.g. an office building)
CC1	Low consequence for loss of human life, or economic, social or environmental consequence small or negligible.	Agricultural buildings where people do not normally enter (e.g. storage buildings), greenhouses

Table 7.2: Definition of consequences classes (Table B1 in EN 1990)

In order to avoid the choice of unnecessarily high execution classes by designers, some countries have decided to define individually the applicable EXC depending of the kind of the structure, e.g. by listed of structural examples in the National Application Documents (NAD).

The consequences of the different EXCs defined by the designing engineer or the customer for the manufacturing consist essentially in different requirements concerning the amount of testing and documentation. But also, consequences concerning the personnel exist and their economical aspect may not be underestimated, see the Table 7.3 about the required technical knowledge of welding coordination personnel.

<table>
<tr><th rowspan="4">Execution class</th><th rowspan="4">Parent material</th><th colspan="4">Type of welding consumables</th></tr>
<tr><th colspan="2">Type 3, Type 4</th><th colspan="2">Type 5</th></tr>
<tr><th colspan="2">Nominal thickness of material in mm</th><th colspan="2">Nominal thickness of material in mm</th></tr>
<tr><th>t ≤ 12[a]</th><th>t > 12</th><th>t ≤ 12[a]</th><th>t > 12</th></tr>
<tr><td rowspan="2">EXC2</td><td>3xxx, 5xxx</td><td rowspan="2">B</td><td rowspan="2">S</td><td>B</td><td rowspan="2">S</td></tr>
<tr><td>Other</td><td>S</td></tr>
<tr><td rowspan="2">EXC3</td><td>3xxx, 5xxx</td><td rowspan="2">S</td><td>S</td><td>S</td><td rowspan="2">C</td></tr>
<tr><td>Other</td><td>C</td><td>C</td></tr>
<tr><td>EXC4</td><td>all</td><td colspan="4">C</td></tr>
<tr><td colspan="6">B Basic technical knowledge according to EN ISO 14731;
S Specific technical knowledge according to EN ISO 14731;
C Comprehensive technical knowledge according to EN ISO 14731.</td></tr>
<tr><td colspan="6">NOTE This table gives no recommendation about possible combinations of constituent materials (parent materials and filler metal) to be welded. For allowed and recommended combinations, see EN 1999-1-1.</td></tr>
<tr><td colspan="6">[a] Endplates up to 25 mm.</td></tr>
</table>

Table 7.3: Required technical knowledge of welding coordination personnel (Table 9 in EN 1090-3)

Chapter 8 - Maintenance

Seeking for an almost lifelong maintenance-free material, aluminium is the material of choice. Its durability combined with its resistant coating and typical finishes, such as anodization, makes aluminium structures extremely easy to maintain. Maintenance work for aluminium products is almost unnecessary, due to the excellent corrosion behaviour of this element. While protective measures might concern those elements that are used to assemble structural members (e.g. bolts), most aluminium structures can remain unprotected if the environmental conditions allow that. Nevertheless, we see more and more structures (e.g. pedestrian bridges) which are coated in many attractive colours, although this is not strictly necessary to protect from corrosion but adds aesthetic value to the structure itself. Coating of aluminium with organic materials is a means to improve the attractiveness of aluminium-based structural solutions. The durability of organic coatings on aluminium is excellent. Uniform colours as well as personalised designs or patterns can be applied to it.

Another way of treating aluminium surfaces is done through a very specific process called *"anodization*". In case of continuous anodization, this process allows to create a stable oxide layer with a controlled thickness up to 25 µm, on top of a natural oxide surface layer of a few nm. The stable oxide layer guarantees additional resistance to UV, scratching and corrosion of the product. Indeed, this additional layer is very resistant against weathering agents and often appreciated for its decorative versatility (see Figure 8.1). Thicker anodized protective layers can be achieved through batch anodization.

Figure 8.1: Example of pallet of colours from anodization process (Coil, Aloxide products)

As structural materials are often exposed to outside weather conditions, soiling can ruin the original decorative appearance of the surfaces. At the same time soiling increases the risk of corrosion. Therefore, cleaning at least once a year ,using neutral cleaning agents, is recommended to preserve the original look. Acids or alkaline cleaning agents should be avoided while neutral solutions (pH 5 to 8) in combinations with mechanical cleaning are to be privileged. Organic coatings may be removed by organic solvents, which make no harm to aluminium.

In general, it is recommended to visually inspect structures that are not frequently used or checked, to verify that no major alterations have taken place, such as damaging parts of the structure by accident, vandalism or deliberately by unauthorized persons. The state of coatings and possible deposition of dust, soil or leaves are to be checked as they might influence the appearance of the structure or create corrosion risks.

Structures that are exposed to cyclic loads are prone to fatigue. Mandatory regular checks are necessary, as fatigue fractures can occur under much lower stress compared to static load conditions. EN 1999-1-3 distinguishes two cases:

Safe life method/principle

This method is applied in case the structure has been designed, or a priori can be assumed, to perform safely for a specific period of time with an acceptable small probability of failure by fatigue cracking.

Damage tolerant design/method

This method can be applied in case the structure is designed damage tolerant, which means the structure withstands local defects safely until maintenance work is carried out.

For the safe life method, a systematic inspection for fatigue cracks is negligible, which is why this method is typically applied where inspections are generally considered as not possible. In case of an applied damage tolerant design, a systematic inspection system of the structure is an inherent part of this method. Critical spots must be checked with respect to fatigue cracks and measures must be taken to repair or to replace the respective component. Details about the frequency, the beginning of inspections in combination with different safety factors and for different consequence classes are given in EN 1999-1-3.

In order to support the decision what method is to be preferred, a Note in EN 1999-1-3 (i.e. 2.2.2(1)) informs that "*the damage tolerant design method may be suitable for application where a safe life assessment shows that fatigue has a significant effect on design economy and where a higher risk of fatigue cracking during the design life may be justified than is permitted using safe life design principles*." Such method is intended to result in the same reliability level as obtained by using the method of safe life design.

For engineering structures in connection with roads (e.g. bridges, traffic sign structures, etc.) a higher frequency of inspections is recommended as accidents can happen at any time. However, this does not only concern aluminium. In this case, regulations about the frequency of inspections given by national or local authorities (in some cases even by insurance companies) are to be followed.

Chapter 9 – Sustainability of aluminium structures

9.1.The aluminium cycle

Aluminium is a material that shows its value at every stage of its life cycle, from production till the end of use. Aluminium can rightfully claim to be a material with "permanent" characteristics or properties, meaning the inherent properties do not change during use and following repeated recycling into new products. Obviously, used aluminium has to be collected and sorted properly to make it available for its next use phase.

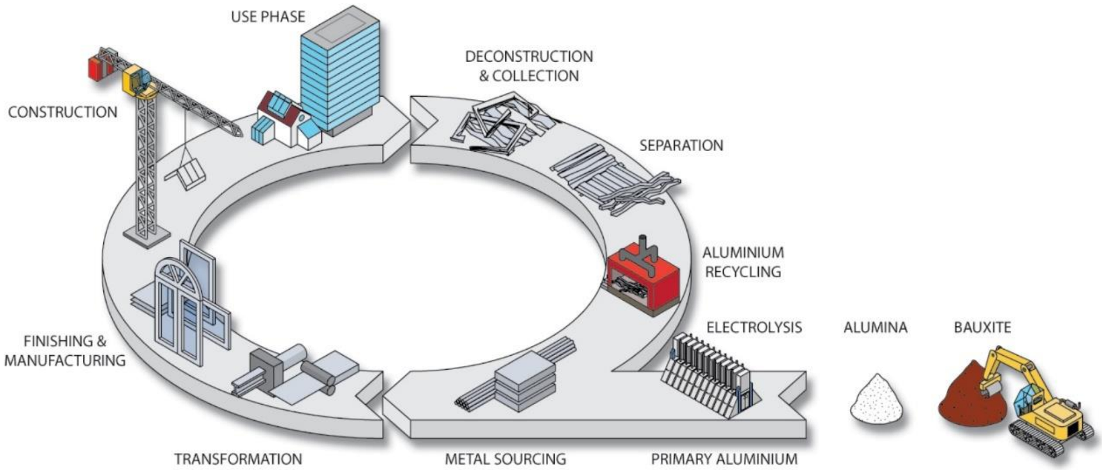


Figure 9.1: the aluminium cycle

A way to acquaint environmental information of aluminium products is by the means of so-called Environmental Product Declarations (EPD). An independently verified and registered document that communicates transparent and comparable information about the environmental life-cycle impact of products.

9.2. Metal sourcing

More than half of the aluminium currently produced in the European Union originates from recycled raw materials, and this trend is increasing. As the energy required to recycle aluminium is about 5% of that needed for primary production, the ecological benefits of recycling are obvious. Due to the long lifespan of buildings and transport vehicles, the available quantity of end-of-life aluminium scrap today is limited to what was put on the market many years ago. This volume being much less than the current demand, the missing quantity needs to be supplied by the primary aluminium industry. Bauxite, the ore from which primary aluminium is produced, originates mainly from Australia, Brazil, West Africa and the West Indies, as well as from other tropical and sub-tropical regions. Rehabilitation of existing mining areas balances out the commissioning of new mining areas. 98% of mines have rehabilitation plans, and the area returned to native forests is expected to be higher than the original vegetation before mining[2].

[2] *4th Sustainable Bauxite Mining Report. International Aluminium Institute - 2008*

Primary aluminium is obtained by the electrolysis of alumina (aluminium oxide) that is extracted from bauxite. Total greenhouse gas emissions from European aluminium were reduced by 45% between 1990 and 2005.

9.3. Transformation

Aluminium flat products are obtained through the rolling process, whereby large aluminium slabs are fed into rolling mills that turn aluminium into sheets of various thicknesses. The process normally begins with a hot rolling method, moving the block back and forth through a reducing roller. Final rolling is a cold roll process, and the sheet can be reduced to a thickness of 0.15 mm. The sheet can be further thinned into foil of a thickness of 0.007 mm. Further information of the availability of sheets and plates for structural purposes can be found in chapter 3.9.1.

Aluminium is one of the few metals that can be casted in all metal casting processes. The most common methods include die casting, permanent mould casting and sand casting. Castings can be made to virtually any size. Thus, the material has little restrictions in design and is of great advantage for architects.

Aluminium profiles are obtained through the extrusion process. A hot cylindrical billet of aluminium is pushed through a shaped die (for more information, refer to chapter 3.9.2). The ease with which aluminium alloys can be extruded into complex shapes allows the designer to "put metal exactly where it is needed", and also to introduce multi-functional features. In construction, aluminium extrusions are not only used for structural purposes, but are also commonly used in commercial and domestic buildings for windows, doors and curtain wall frame systems and many other applications.

9.4. Use phase

Aluminium is highly appreciated for its very long in-service life, low maintenance and other assets like light weight, corrosion resistance and functionality. These assets are explained more in detail in chapters 4 and 8.

9.5. Deconstruction

A study by Delft University of Technology revealed aluminium's considerable end-of-life recovery rate in the building sector. Aluminium collection rates taken from a large sample of commercial and residential buildings in six European countries were found to be above 92% (on average 96%), demonstrating the value and preservation of the material at the end of the aluminium product life cycle.

9.6 Recycling

The high intrinsic value of aluminium is a major economic incentive for its recycling. Indeed, aluminium scrap can be repeatedly recycled without any loss of value or properties. Furthermore, the energy required is a mere fraction of that needed for primary production, often as little as 5%, yielding obvious ecological benefits.

In many instances, aluminium is combined with other materials such as steel or plastics. Mostly they are mechanically separated from aluminium before being molten: shredding followed by eddy current and sink-float separation. Aluminium can then be melted either by remelters or refiners.

- Remelters mainly process wrought alloy scrap. They produce extrusion billets or rolling slabs.
- Refiners melt all kinds of scrap, including mixed alloys and soiled scrap. They mainly produce casting alloys for foundries.

As technology evolves, a growing number of remelters are now able to process coated and polymer containing scraps with no or limited preparation processes. One solution is the use of a two-chamber furnace: finishes to the aluminium (e.g. coating) are burnt away in the first chamber and gas emissions are collected in efficient fume capture equipment while aluminium heating mainly takes place in the second chamber. Alternatively, plants using pyrolysis technology use the organic particles attached to the aluminium scrap as fuel for the pyrolysis process that takes place in rotating drums while the heat burns away the organic material off the metal.

Liquid aluminium can then be transported directly to foundries, casted into ingots, extrusion billets or rolling slabs ready to get reused. Consequently, the life cycle of an aluminium product is not the traditional “cradle-to-grave” sequence, but rather “cradle-to-cradle”.

Chapter 10 - Applied examples

All the examples of real structures shown here represent relevant cases where the aluminium has been selected because it has been demonstrated that this solution was more competitive than the one in steel. In all cases, the basic pre-requisites already defined in Chapter 4 have been fundamental for deciding the aluminium choice.

When the incidence of the structural weight is fundamental, the utilization of the aluminium can represent a valid alternative to steel. In addition, the complete absence of maintenance, due to corrosion resistance, increases the advantages for those structures situated in humid environments.

10.1. Lattice space structures

Several applications of lattice space structures can be found in South America (Brazil, Colombia, Ecuador). The historical background in this field is represented by a very spectacular space structure which has been erected for the Interamerican Exhibition Center of San Paulo in Brazil in 1969 (Fig. 10.1). This structure covers an area of about 67600 m^2 with a mesh 60x60 m. The depth of the lattice layer is 2,36 m. It was entirely site-bolted on the ground and after lifted at the final level of 14 m by means of 25 cranes located in the corners of the mesh, in the position of the actual supports. The weight of the lattice structure was 16 kg/m2; the number of bars was 56820 and their total length one after another was 300 km. The erection time was extraordinary quick (27 hours!), by using a number of 550000 bolts in 13724 nodes. The materials were: aluminium alloys of 6063 and 6351 series T6 for cylindrical bars; Al 99,5 for trapezoidal sheeting and galvanized steel bolts for connections. Very similar is the case of the International Congress Centre in Rio de Janeiro, where the same mesh 60x60 m has been used, covering in total 33000 m^2 (Fig.10.2).

Among many different applications, mention can be given to lattice system covered by aluminium sheeting for roofing the Sport Hall of Quito, Ecuador (Fig. 10.3) and to the lattice vault for roofing the swimming pool of the Country Clubs of Hatogrande and Guymaral in Bogotá, Colombia (Fig. 10.4).

10.2 Reticulated domes

The reticulated domes represent the most challenging application of aluminium alloys in the structural field, allowing the realisation of important constructions (sporting houses, exhibition centres, congress halls, auditoriums, etc). These applications are very interesting for the rapidity of erection, the connection systems and the remarkable dimensions.

The first applications of this kind of structures were: the “Dome of Discovery” erected in London for the South Bank Exhibition during the Festival of Britain (1951), composed by three directional reticulated arches, with a diameter of 110 m and 24 kg/m^2 weight (Fig.10.5) and the geodetic dome erected for covering the “Palais des Sports” in Paris, by using the Kaiser Aluminium system with 61 m diameter and 20 m height (1959) (Fig.10.6). Both were like prototypes in their field, being the largest and the first, respectively.

10.3 Geodetic domes

More recently, interesting structural systems for geodetic domes made of aluminium have been set-up in U.S.A. The Tem-cor, Conservatex and the Geometrica systems are used both for roofing industrial plants with ecological purposes (Fig. 10.7) and for large roofing public buildings (Fig.10.8). A famous application in USA was the "Spruce Goose" dome in Long Beach (California), which contained an airplane claiming the largest wingspan of any aircraft in history and being still the largest flying boat ever made in 1947. Since 2009, it was the largest dome in the World with a diameter of 126 meters (Fig. 10.9).

A significant application of geodetic dome has been done in the restauration of the Museum of "Mercati Traianei" in Rome (Fig. 10.10), by using a special aluminium bar-to-node system, called "geo-system" (designer F.M. Mazzolani).

Many geodetic domes are used for industrial applications, like for roofing coal storage plants (Fig. 10.11). The transformation into coal of the thermo-electrical power plant of ENEL in Torrevaldaliga North (Civitavecchia) required a complete change in handling and storing systems, as the fuel is passed from liquid (oil) to solid (coal). The prevention of the dispersion of dust into the environment resulted in a total confinement system for handling the coal from the harbour to the boiler through conveyor belt network. For this purpose, two geodetic domes with the diameter of 144 meters have been designed and built by using the MERO system (Fig. 10.12, designer F.M. Mazzolani). Today, they are the largest aluminium domes in the World and in 2012 they received the European Aluminium Award with the following statement of the jury:
"The jury was admiring the overall quality of the structure, which shows that aluminium is a preferable choice for such large constructions. This dome is both an innovative, environmentally friendly and aesthetic solution for coal storage".

10.4 Special structures

There are special structures having the function to support fixed elements, the prevalent dimension being horizontal (i.e. portal frames for traffic sign, Fig. 10.13) or vertical (i.e. antennas, lighting towers and electrical transmission, Fig. 10.14). For these structures, the elimination of maintenance represents a fundamental prerequisite. At the same time, the extrusion process can improve the geometrical properties of cross-sections in such a way to obtain the minimum weight and the highest structural efficiency. In addition, the lightweight of aluminium allows prefabricated systems, very easy for transportation and erection, giving rise to competitive solutions in comparison with other materials.

Many towers for electrical transmission lines have been erected in Europe (Fig.10. 14). Two important aluminium towers have been erected in Naples. The first is the tower for parabolic antennas of the Electrical Department of Naples erected in 1986 (Fig. 10.15, designer F.M. Mazzolani). This design received the international award "Hundred Years of Aluminium". The reason of the aluminium choice was basically due to lightness (the tower has been erected on the top of an existing reinforced concrete staircase) and corrosion resistance (no problems of maintenance). Its height is 35 m from the top of the staircase (in total 50 m about). It is composed by a cylinder 1800 mm internal diameter and 20 mm thickness. The fabrication was shop-welded, by dividing the total height in three parts, which were field-bolted during the erection.

The second example is the "Information Tower" near the football stadium in Naples, which has been equipped by antennas and screens in order to follow from out-side the stadium games (Fig. 10.16, designer F.M Mazzolani). It was built for the Score International Games of 1990.

A field, where the aluminium properties play a determinant role, is the one of the hydraulic applications (pipelines, reservoir). The rotating crane bridges for large settling circular pools in water sewage treatment plants is a typical case. In particular, the corrosion resistance allows to eliminate any protection also in presence of corrosive environment, while the lightness corresponds to energy saving during the operating phases of the plant. Eight rotating crane bridges have been erected in the Po Sangone sewage plant of Torino (Italy) in 1985 (Fig. 10.17, designer F.M. Mazzolani).

10.5 Off-shore applications

It seems important to underline that nowadays the offshore applications can be considered the main future trend for aluminium alloys. In fact, they offer to this industry enormous benefits under form of cost savings, ease of fabrication and proven performance in hostile environments (Fig. 10.18). Stair towers, mezzanine flooring, access platforms, walkways, gangways, bridges, towers and cable ladder systems can all be constructed in pre-fabricated units for simple assembly offshore or at the fabrication yard (Fig.10.19). Mobility and ease of installation are maintained even for large structural elements, such as link bridges and telescopic bridges. Helidecks have been made by using aluminium alloy since the early seventies, so they have now a fully tried experience. Moreover, they are designed to be modular and bolted connections, allowing quick erection, easy shipping and handling. In addition, they offer weight reduction of up to 70% over steel, meeting the highest safety standards and proving up to 12% cost saving (Fig. 10.20). Complete crew quarters and utilities modules, from large purpose-built modules to flexible prefabricated units, have been recently developed. The modules may be used singly or assembled in group to form multi-story complexes, linked by central transverse corridors and stair towers (Fig. 10.21).

10.6 Bridges

All kind of structural schemes typical for steel bridges have been experienced in aluminium alloys. Also, the technology based on the use of composite structures made of aluminium beams and concrete decks has been applied in some bridges built since the sixties in USA and later in France.

The Arvida Bridge in Quebec, Canada (1950), the challenging prototype of motorway bridge made of aluminium alloy, was built according to the Maillart's scheme with a total span of 150 m, an arch of 87 m of span and total weight of 200000 kg (Fig.10.22). Other motorway bridges have been built in France and in The Netherlands (Fig.10.23).

The footbridge is a structural typology where the aluminium alloys are successfully employed; examples of aluminium foot bridges can be easily found all over the world (Fig. 10.24).

Owing to the low live load, additional advantages due to lightness are evident in case of moving bridges, which first example was the moving bridge at the entrance of the Aberdeen harbour in Scotland (Fig. 10.25). More simple and easy to manage are the moving aluminium bridges for pedestrians (Fig.10.26).

Prototypes for new floating bridges, composed by floating units, have been recently validated and built so to allow crossing of water straits (Fig.10.27).

A new important field of application is the one of military bridges in which lightness and corrosion resistance play a fundamental role. At present, it is possible to reach 40 meters of span with prefabricated elements easy to transport and to erect. The main applications have been developed in Great Britain, Germany, Sweden and Canada (Fig.10.28).

A lightweight system for replacing damaged concrete bridge decks has been developed and used in Sweden, based on an orthotropic plate of aluminium hollow extrusions (Fig.10.29). This solution can be in many cases very competitive as an alternative to the conventional solutions. When it substitutes a concrete deck, a reduction in weight is about 10 times, going from 600 to 700 kg/square meter to 50 o 70 kg/square meter. This weight reduction has made it possible to increase the service load and use the existing foundations without any consolidation operation.
During the seventies, a rehabilitation program for ancient suspension bridges of 19th Century have been developed in France. Aluminium alloy deck and girders have been successfully applied in the refurbishment of three bridges: the

Montmerle (Fig.10.30) and the Trevoux (Fig.10.31) bridges on the Soane river with two bays of 80 m; the Groslée bridge on the Rhône river with a single bay of 174 m (Fig.10.32).

More recently, in the context of a wider rehabilitation programme of the zone, the structural retrofit project has been done for the oldest Italian suspension bridge, the “Real Ferdinando” on the Garigliano river, which was built in 1832 and was destroyed in 1944 during the World War II by the German army in retreat. The restoration criteria were chosen in order to satisfy several requirements: a) historical preservation, b) stiffening of the deck, c) respect of the modern design code provisions, d) adoption of innovative technologies and materials. Comparing other materials, the use of aluminium alloy has been selected for the main and transversal girders of the new deck, allowing the conservation of the original geometrical configuration and appearance (Fig.10.33, designer F.M. Mazzolani).

Aluminium has proven to be the material of choice also for the BITSCHNAU toolbridge system, which was developed to take advantage of the main characteristic of the material, such as low maintenance needs, resistance to corrosion, very long operational life, persistent and high-quality sustainability and best price-performance ratio. The system is made with sea water-resistant aluminium alloy, whose surface can be decorated through anodizing or varnish coating. This type of special bearing constructions (Fig 10.34 and 10.35) make it possible to equalize level differences of abutment up to 20mm and are arranged for the service vehicle up to 7,5 metric tons (with the possibility to increase transport load up to 12,5 metric tons or more).

10.7 Architectural buildings

Going from structural to architectonical applications, a significant aluminium building is the former Aluminium Centrum in Utrecht (The Netherlands), designed by Micha de Haas, which main body is supported by a “forest” of tubular columns, part in the water and part in the bank side (Fig. 10.36).

10.8 Other applications

Aluminium proved to be a suitable material also for other special applications such rear underrun protection components for mobile cranes and heavy trucks (Fig. 10.37, 10.38 and 10.39), whose main purpose is to limit as much as possible the intrusion of cars in case of road accidents. Also in this case, the selection of an aluminium beam was the best cost-technical-weight compromise when compared to other existing solutions. The beam weight could be significantly reduced in comparison to a steel beam solution, while benefitting at the same time from optimal corrosion protection and functional integration.




Figure 10.1 The lattice space structure of the Interamerican Exhibition Center of San Paulo (Brazil): erection phases.

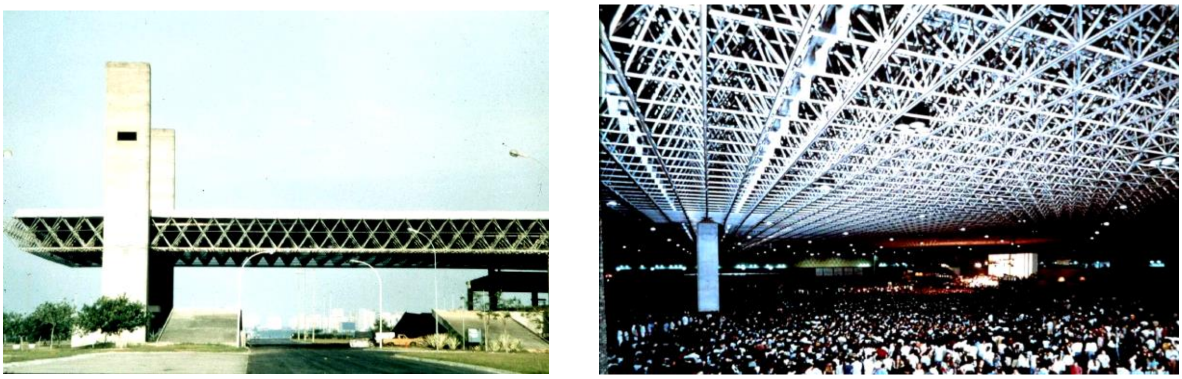


Figure 10.2 The Conference Centre of Rio de Janeiro (Brazil).

Figure 10.3. Sport Hall of Quito (Ecuador).

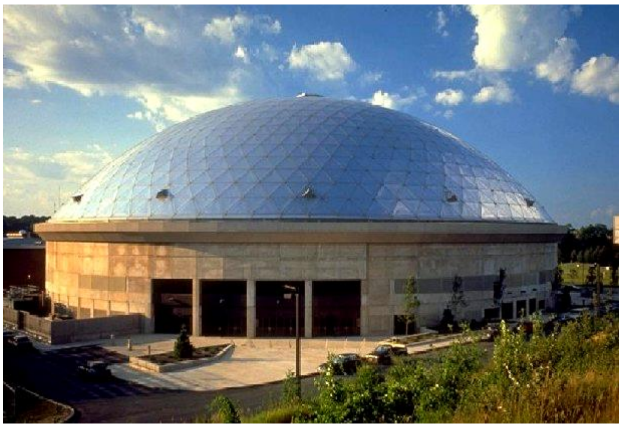
Figure 10.4: Hatogrande (left) and Guymaral (right) Country Clubs in Bogotá (Colombia)

Figure 10.5 Dome of Discovery (London)

Figure 10.6: Palais des sports (Paris)

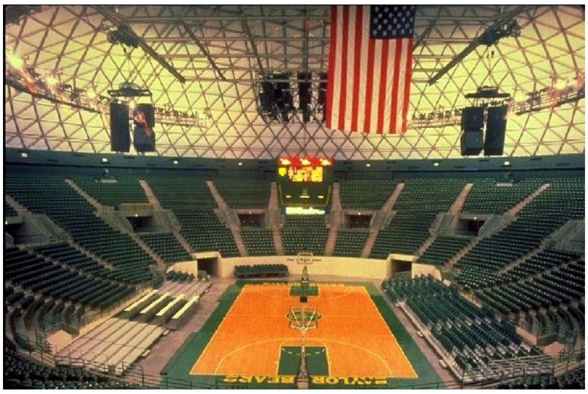
Figure 10.7: Geodetic domes for industrial applications

a) South Pole scientific station



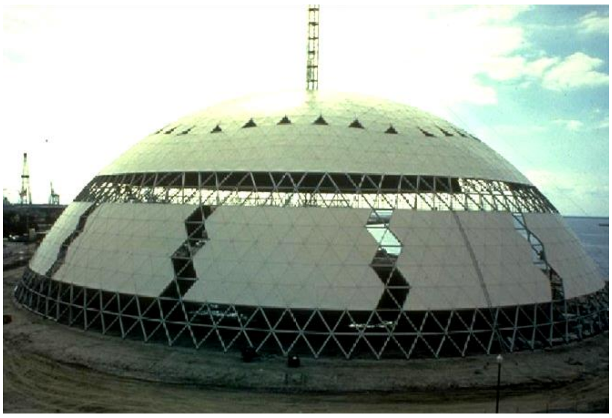
b) University of Connecticut (USA)

c) Bell County Arena (Temple, Texas, USA)



d) Baylor University Ferrell Events Centre (Waco, Texas, USA)

Figure 10.8: Tem-cor geodetic domes.



Figures 10.9: The “Spruce Goose” dome in Long Beach (USA): erection phases

Figure 10.10: The geodetic dome in the "Mercati Traianei" Museum in Rome (Italy)

Figure 10.11: Aluminium domes for coal storage plants

Figure 10.12: The ENEL geodetic domes in Torrevaldaliga (Rome, Italy)

Figure 10.13: Traffic sign portal

Figure 10.14: Electrical transmission tower

Figure 10.15: The ENEL antennas tower (Naples, Italy)

Figure 10.16: The "Information" tower (Naples, Italy)

Figure 10.17: The sewage plant pool of Po-Sangone (Torino, Italy)

Figure 10.18: Off-shore platform

Figure 10.19: Prefabricated unit for off-shore platform

Figure 10.20: Helideck

Figure 10.21: Complete crew quarter on off-shore platform

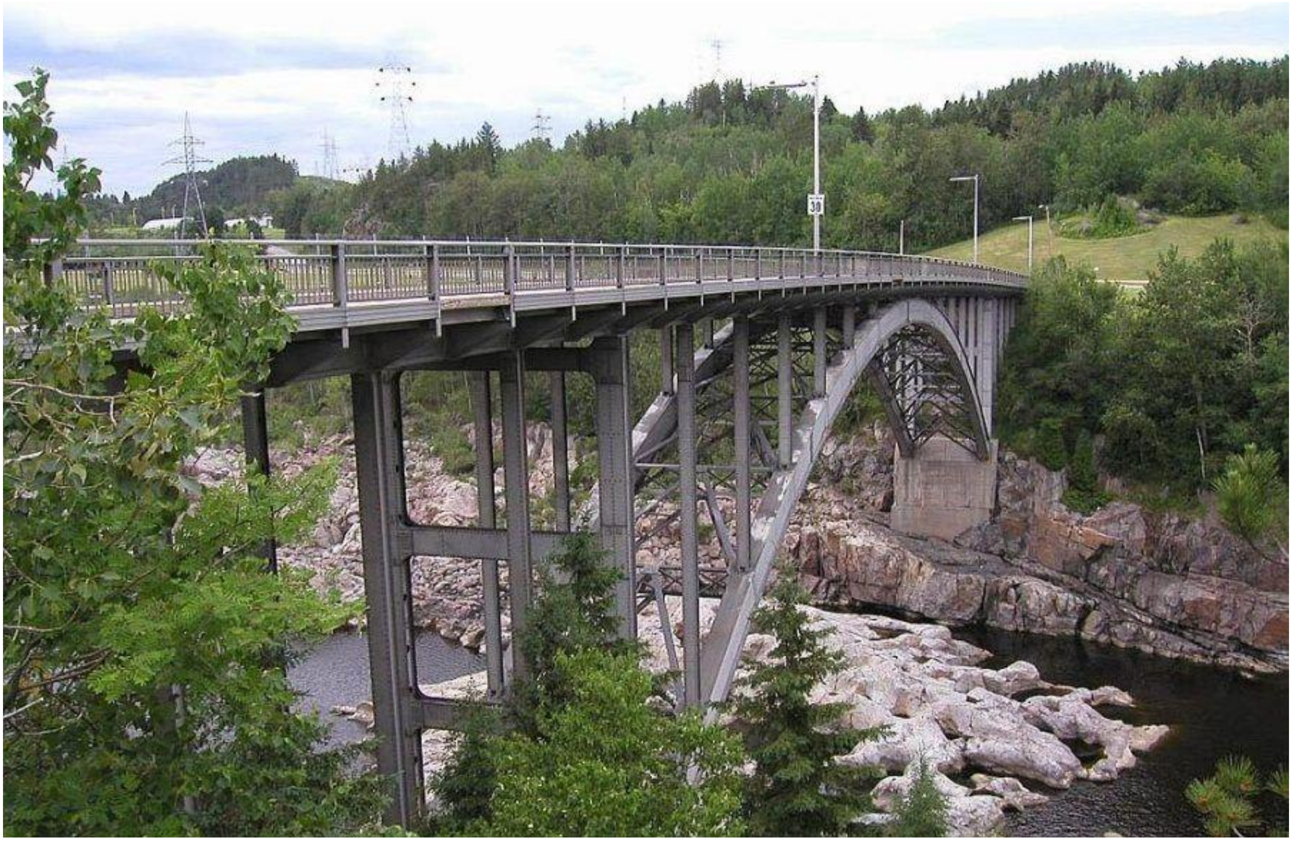


Figure 10.22: The “Arvida” bridge in Quebec (Canada).

Figure 10.23: Motorway bridge crossing a canal in Amsterdam (The Netherlands)

Figures 10.24 Pedestrian bridges

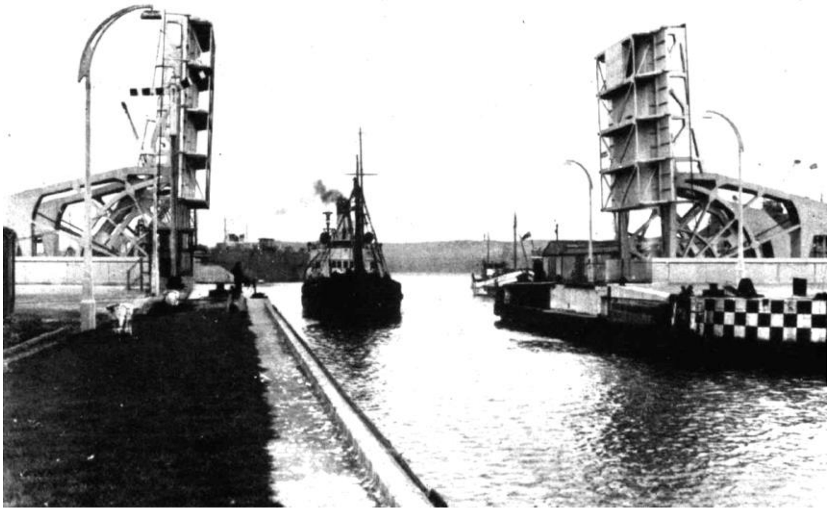


Figure 10.25 The opening bridge of the Aberdeen harbour (Scotland)

Figure 10.26 A simple moving footbridge (DE)

Figure 10.27 Floating bridge

Figures 10.28 different types of military bridges

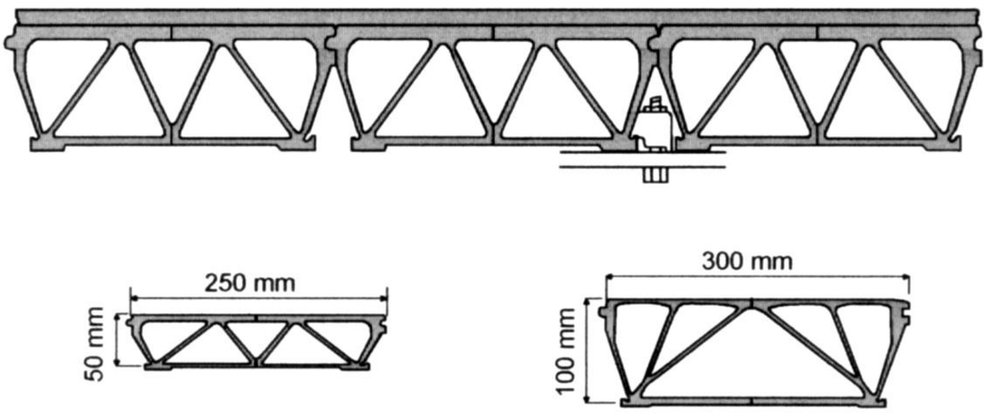
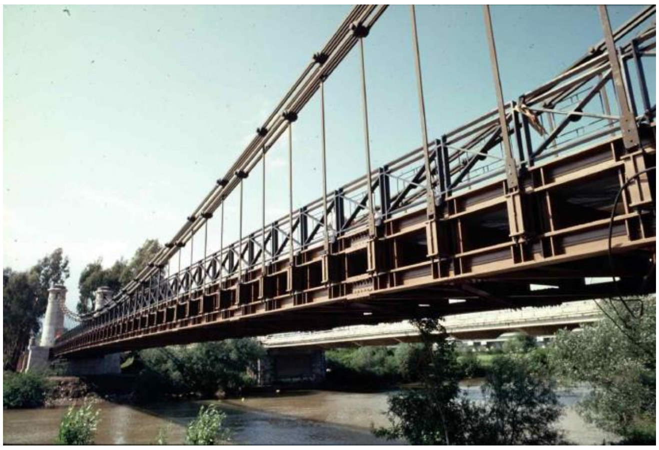


Figure 10.29 Extrusion plates for bridge decks

Figure 10.30 The Montmerle bridge on the Saône river (France)

Figure 10.31 The Trevoux Bridge on the Saône river (France)

Figure 10.32: The Groslée bridge on the Rhône river (France)



Figures 10.33: The “Real Ferdinando” bridge on the Garigliano river (Italy).

Figures 10.34 and 10.35: Aluminium Bridges (toolbridge© by Bitschnau)

Figure 10.36: The former "Aluminium Centrum" in Utrecht (The Netherland).

Figures 10.37, 10.38 and 10.39: Rear underrun protection component, car intrusion test, aluminium beams (pictures by Hydro)

Terms and definitions

ageing
treatment of a metal aiming at a change in its properties by precipitation of intermetallic phases from supersaturated solid solution

annealing
thermal treatment to soften metal by reduction or removal of strain hardening resulting from cold working and/or by coalescing precipitates from solid solution

buckling
sudden change in shape of a structural component under load (e.g. the bowing of a column subjected to compression forces)

casting
product at or near finished shape, formed by solidification of the metal in a mould or a die

cold working
forming of solid metal without preheating (also said work-hardening or strain hardening)

extrusion ingot
ingot, intended and suitable for extruding, typically with a solid, circular cross-section but sometimes with a central hollow or a flattened cross-section

extrusion billet
extrusion ingot cut to length

Heat Affected Zone (HAZ)
a non-melted area of metal that has undergone changes in material properties as a result of being exposed to high temperatures

heat treatable alloy
alloy which can be strengthened by a suitable thermal treatment

hot working
forming of solid metal after pre-heating

formability
relative ease with which a metal can be formed by rolling, extruding, drawing, deep drawing, forging, etc.

lattice imperfection
imperfection in the regular geometrical arrangement of the atoms in a solid

lacquering
process that involves a surface application that increases the strength and tear resistance of the aluminium as well as the gloss and adherence of other coatings
EN 12258-1 says: coating with a formulation based on a dissolved material which forms a transparent layer primarily after drying by evaporation of the solvent

non-heat-treatable alloy
alloy which is not strengthened by thermal treatment (only strengthened by hot or cold working).

quenching

cooling a metal from an elevated temperature by contact with a solid, a liquid or a gas, at a rate rapid enough to retain most or all of the soluble constituents in solid solution

solution heat-treatment

heating an alloy at a suitable temperature for a sufficient time to allow one or more soluble constituents to enter into solid solution, where they are retained in a supersaturated state after quenching (rapid cooling)

strain-hardening

modification of a metal structure, by cold working, resulting in an increase in strength and hardness, generally with loss of ductility

temper

condition of the metal produced by mechanical and/or thermal processing, typically characterized by a certain structure and specified properties

Standards & Normative references

EN 1090-1:2009+A1:2011: Execution of steel structures and aluminium structures. Requirements for conformity assessment of structural components

EN 1090-3:2019: Execution of steel structures and aluminium structures. Technical requirements for aluminium structures

EN 1090-5:2017: Execution of steel structures and aluminium structures. Technical requirements for cold-formed structural aluminium elements and cold-formed structures for roof, ceiling, floor and wall applications

EN 12020-1: Aluminium and aluminium alloys - Extruded precision profiles in alloys EN AW-6060 and EN AW-6063 – Part 1: Technical conditions for inspection and delivery

EN 12020-2: Aluminium and aluminium alloys - Extruded precision profiles in alloys EN AW-6060 and EN AW-6063 - Part 2: Tolerances on dimensions and form

EN 12258-1, Aluminium and aluminium alloys - Terms and definitions - Part 1: General terms

EN 15088, Aluminium and aluminium alloys - Structural products for construction works

EN 1780-1, Aluminium and aluminium alloys - Designation of alloyed aluminium ingots for re-melting, master alloys and castings - Part 1: Numerical designation system

EN 1780-2, Aluminium and aluminium alloys - Designation of alloyed aluminium ingots for remelting, master alloys and castings - Part 2: Chemical symbol-based designation system

EN 1780-3, Aluminium and aluminium alloys - Designation of alloyed aluminium ingots for re-melting, master alloys and castings - Part 3: Writing rules for chemical composition

EN 1990, Eurocode 0: Basis of structural design

EN 1999, Eurocode 9: Design of aluminium structures

EN 1999-1-1: General structural rules

EN 1999-1-2: Structural fire design

EN 1999-1-3: Structures susceptible to fatigue

EN 1999-1-4: Cold-formed structural sheeting

EN 1999-1-5: Shell structures

EN 515: Aluminium and aluminium alloys - Wrought products - Temper designations

EN 573-1: Aluminium and aluminium alloys - Chemical composition and form of wrought products - Part 1: Numerical designation system

EN 573-2: Aluminium and aluminium alloys - Chemical composition and form of wrought products - Part 2: Chemical symbol-based designation system

EN 573-3: Aluminium and aluminium alloys - Chemical composition and form of wrought products - Part 3: Chemical composition and form of products

EN 755: Aluminium and aluminium alloys. Extruded rod/bar, tube and profiles

EN 755-1: Aluminium and aluminium alloys- Extruded rod/bar, tube and profiles – Part 1: Technical conditions for inspection and delivery

EN 755-2: Aluminium and aluminium alloys- Extruded rod/bar, tube and profiles – Part 2: Mechanical properties

EN 755-3/9: Aluminium and aluminium alloys- Extruded rod/bar, tube and profiles – Parts 3 to 9: Tolerances on dimensions and form

Eurocodes, Joint Research Centre – online available at https://eurocodes.jrc.ec.europa.eu/

Regulation (EU) No 305/2011 on Construction Products, online available at https://eur-lex.europa.eu/legal-content/EN/ALL/?uri=celex:32011R0305

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