# HIGH STRENGTH STEEL DESIGN AND EXECUTION GUIDE

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Front cover: Friend's Arena, Stockholm. The top chords of the roof truss were made from S460 steel; S690 and S900 steel were used in the bottom chords and outer diagonals respectively.

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T: +44 (0)1344 636525
F: +44 (0)1344 636570
E: reception@steel-sci.com

**www.steel-sci.com**

To report any errors, contact:
publications@steel-sci.com

# HIGH STRENGTH STEEL DESIGN AND EXECUTION GUIDE

**Nancy Baddoo** MA CEng FICE
**Dr Anqi Chen** BEng PhD

# FOREWORD

High strength steels are being used increasingly in a range of applications in the construction industry, particularly for heavy columns, transfer beams, trusses and bridge girders. This guide provides advice for designers, fabricators, product manufacturers and clients on the selection, design and execution of high strength steel structures.

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# SUMMARY


The purpose of this guide is to present comprehensive guidance on when and how the benefits of steels with strengths from 420 to 700 MPa can be exploited in practical design situations in the construction industry. Information on product availability, execution and welding is also given.

The use of higher strength steel can enable substantial savings in structural weight and material costs; although high strength steels are more expensive than conventional strength steels, the price increase is usually less than the rate at which the strength increases. The reduction in weight leads to a cost saving in the foundations, welding, fabrication, transportation and erection. The weight savings depend on the type of member and mode of loading, but in many practical situations might range from 10 to 40 %. Significant reductions in carbon dioxide emissions are also possible. The use of higher strength steels can also result in more clearance, greater design freedom and less congestion. However, the benefits can be outweighed by the greater tendency to unstable failure modes, or for serviceability criteria such as deflection or vibration to govern design. Greater control is also needed during certain execution processes, especially welding.

Some of the forms of construction where high strength steel could lead to significant economic benefits are found to be:

- Heavily loaded columns in high-rise buildings, where the use of higher strength steel enables a reduction in thickness of the steel elements. This applies to columns which are sufficiently stocky so that their design is not limited by overall buckling. It also applies to columns with low to medium slenderness because the buckling curve used for higher strength steel is more advantageous for some types of cross-section than for grades up to S420 because of the smaller effect of residual stresses.
- Long span roof trusses can be designed efficiently using higher strength steel when not limited by deflections. This applies to both the tension and compression members of low to intermediate slenderness.
- Bridges with heavier loading, such as in railway applications or in highly stressed regions of bridges. This also includes the negative bending region of continuous composite bridges subject to high shear and bending, and pylons supporting cable stayed bridges or curved bridges subject to combined bending and torsion.

- Deep transfer beams supporting columns from the levels above. Increasing the web strength may be advantageous in high shear cases, but not in high bending-low shear cases.
- Concrete filled sections made from high strength steel and possibly using higher strength concrete, where allowance for compatible strains in the steel and concrete should be made.
- Steel-concrete composite fabricated beams using higher strength steel in the bottom flange to increase the bending resistance of the composite section. This is advantageous in beams with a span to depth ratio less than 20 that are not governed by serviceability considerations of deflection and vibration.
- Composite fabricated beams with large web openings (with an aspect ratio > 2), where the local bending resistance of the Tee sections above and below the opening could be increased by use of higher strength steel in the web. This could also extend to the use of higher strength steel in the horizontal stiffeners where large web openings (with an aspect ratio > 2.5) are designed.

# INTRODUCTION

## 1.1 Use of high strength steels in construction

Steel structures maintain a dominant presence in the construction industry as a result of continuous advances in material properties, production methods and innovative design and construction techniques. Modern steel production techniques, such as thermomechanical rolling, and quenching and tempering, enable the economic production of high strength steels (HSS) with the weldability, fracture toughness and ductility required for structural applications. Nowadays, weldable steel products such as plates with yield strengths of up to 1300 MPa are available.

The global HSS market is growing quickly, due to the increasing applicability of HSS across various industries and technological advances in their production.

HSS are widely used in applications where weight reduction, with corresponding decreases in emissions and energy use, leads to considerable cost savings, such as cars, trucks, lifting equipment and yellow goods. They are also used for pipelines, tanks and pressure vessels, as well as for components on offshore structures and in nuclear, thermal and hydroelectric plants.

Twenty years ago, in the UK, S275 was the standard steel grade for open sections (I-shaped sections, channels etc) and S355J2H was the standard grade for hollow sections. (Note that S275 designates a steel with minimum yield strength of 275 MPa.) In other parts of the world, lower strength grades such as S235 tended to be the more common grades.

However, the availability of steel sections in S275, particularly the larger sections, has become extremely limited in the UK, and other European countries, as the market has moved towards the higher standard strength grade S355. Nowadays S355 tends to be the standard steel grade for open sections and there is likely to be a minimum quantity or cost and programme penalties imposed for specifying S275. Additionally, S420 is becoming more widely available for hollow sections. The use of steels S460 and stronger is increasing, particularly in large structures for heavily loaded sections, and also in selective parts of structures subject to heavy loads.

Demand from construction specifiers has led to European product standards for hot-rolled products (EN 10025[1] and EN 10149[2]) now covering steels with strengths up

*Table 1.1 Definition of high strength by construction sector (Source: Research Fund for Coal and Steel, 2011)*

| | | Specified Minimum Yield Strength [MPa] | | | | | | | | | | | | Factors Affecting Use of HSS | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| **Sector** | **Product form** | **235** | **275** | **300** | **355** | **400** | **420** | **450 460** | **500** | **550** | **690** | **890** | **1000 +** | **Advantages** | **Factors limiting use** |
| Buildings | Profiles | Lo | Lo | Med | Med | Hi | Hi | Hi | | | | | | Long spans, aesthetics | Deflection, strain dissipation |
| Foundations, quay walls | Sheet piles, piles | Lo | Lo | Med | Med | Hi | Hi | Hi | | | | | | Long spans, aesthetics | Deflection, strain dissipation |
| Bridges: road (small & medium spans) | Fabricated girders/profiles | Lo | Lo | Med | Med | Hi | Hi | Hi | | | | | | Longer spans, installation | Fatigue (welds), toughness |
| Bridges: road | Fabricated girders | | | | Lo | Lo | Med | Med | Hi | Hi | Hi | | | Longer spans, installation | Fatigue (welds), toughness |
| Bridges: rail | Fabricated girders/profiles | Lo | Med | Med | Hi | | | | | | | | | Limited (fatigue dominates) | Fatigue (welds) |
| Fixed offshore rigs | Welded plate | | | | Med | Med | Med | Med | Hi | Hi | | | | Transport, installation | Fatigue (welds), corrosion fatigue |
| Mobile offshore rigs | Welded plate | | | | Lo | Lo | Med | Med | Med | Hi | Hi | Hi | | Reduced weight, ease of installation | Stress corrosion |
| Mobile cranes | Welded tubulars/ profiles | | | | | | Lo | Lo | Lo | Med | Med | Med | Hi | Reduced weight, longer spans | Toughness |

Legend

Lo = Considered as low strength for the sector    Med = Considered as normal strength for the sector    Hi = Considered as high strength for the sector to 960 MPa and the forthcoming revisions of the standards for hollow sections (EN 10210[3] and EN 10219[4]) are expected to do likewise.

Running in parallel with developments to product standards, the scope of the main part of the European steel design standard, Eurocode 3 (EN 1993-1-1[5]) is being extended to cover steels with yield strengths up to 700 MPa. A new part of Eurocode 3 giving supplementary rules for steels with strengths up to 960 MPa is currently under development. The scope of the European execution standard, EN 1090-2[6] covers hot rolled steel and cold formed steel products with strengths up to 700 MPa. Work is underway preparing an annex which gives additional requirements for steels with strengths up to 960 MPa.

HSS are defined in this guide as steels with a yield strength between 420 and 700 MPa. Steels with yield strength greater than 700 MPa are often called advanced high strength steels (AHSS). It should be noted that the definition of ‘high strength’ differs depending on the particular sector of the construction industry. Table 1.1 shows the definitions agreed by the European Research Fund for Coal and Steel.

## 1.2 Why use high strength steels?

The main drivers for using HSS are explained below.

### Reduction in self-weight

This is especially important in structures where self-weight is critical or where the designer needs to minimise plate thicknesses. The use of a higher strength steel can enable substantial savings in structural weight and material costs. Although HSS are more expensive than conventional strength steels, the price increase is less than the rate at which the strength increases:

- S460 is 30 % stronger but only about 10-15 % more expensive than S355.
- S690 is nearly twice as strong but only about 30 % more expensive than S355.

The weight savings made possible by higher strength depend on the type of member, mode of loading and design strategy.

### Smaller foundations

A lighter structure may require smaller and/or shallower foundations, which may lead to savings on materials as well as construction times. This advantage decreases when lateral loads prevail.

### Easier transportation

A lighter structure may cut the cost of transportation, but this is not always the case as transportation is often limited by size rather than weight.

### Easier fabrication and construction

The use of higher strength will enable thinner sections which are quicker to weld together and paint. Lighter structural members are also easier to lift into place, and the lifting operations are less likely to be restricted by the capacity of the tower crane.

### Lower $CO_2$ emissions

The use of high strength steel leads to lower $CO_2$ emissions and energy use (both directly linked to the reduced amount of materials used and also indirectly due to lower transportation costs). The $CO_2$ emissions from the production of higher strength steels are only slightly higher than those for conventional structural steels (see Section 4). Hence significant reductions in $CO_2$ emissions are possible.

### More space

Smaller beams may lead to more space for services or a lower building. In commercial buildings, reducing the sizes of vertical structural elements can lead to more office space to lease. Larger clearance, greater design freedom and less congestion can be very important in certain types of structures, including offshore oil and gas platforms, both during construction and in-service.

### Cost effective alternative to local strengthening

In some cases the use of a higher strength may be a more cost effective alternative to local strengthening at joints in a structure made from conventional strength steel. Local stiffening in the form of doubler plates and transverse stiffeners can be very expensive due to the fabrication involved. These types of solutions can be avoided by using a higher steel grade for the main member (and perhaps thicker flanges). A higher steel grade and the use of column sections with thicker flanges can eliminate the need for transverse stiffeners.

### Potential to reduce fire protection

The fire resistance of a structural member could be improved by using a higher strength steel if the room temperature design strength is still taken as S355. As the strength retention characteristics of steels up to S700 are the same as S355 steels, at elevated temperatures, the use of a HSS could lead to a reduction in the requirements for fire protection, for example, shorter drying time required for a thinner coat of intumescent paint and also lower cost. Section 7.3 discusses alloys with improved fire resistance.

## 1.3 Brief review of the development of HSS

HSS have been used for structural applications such as bridges, buildings, legs of offshore jack-up rigs, cranes etc. for more than four decades. HSS products are now widely available in a range of product forms including plate, sheet, strip, hollow sections, open rolled sections and bars/rods.

Weldable high strength structural steels are delivered in three conditions: normalized/ normalized rolled (N), thermomechanical rolled (M) and quench and tempered (Q). More information about these supply conditions is given in Section 2.1.

For steel grades with moderate strength and toughness requirements, standard hot rolling (i.e. as-rolled), or normalizing of the steel material is sufficient to obtain the necessary mechanical properties. Structural steels up to S355 and S420 at lighter thicknesses can be produced by these various process routes using a variety of steel alloy designs.

For higher strength structural steels, or steels requiring enhanced low temperature toughness performance, a much finer-grained microstructure is desirable. This is achieved by thermomechanical (TM) rolling or a combination of TM rolling coupled with water cooling (also referred to as thermo-mechanically controlled processed (TMCP) steels). In particular, these HSS also make use of microalloying elements such as niobium (Nb) to develop finer-grains and vanadium (V) for additional strengthening. Furthermore, for the same grade of strength, TM rolled steels tend to have a lower carbon and alloy contents, and so overall better weldability than normalized steels. It should also be noted that finer grains will simultaneously contribute towards a higher yield strength as well as improved toughness.

Quenched and tempered steels (mainly plates) are generally used in non-structural applications such as mining and where heavy plate thicknesses are required and not produced under M or N delivery conditions. However, due to the higher carbon content in Q steels (and also other alloying elements), tighter controls are needed for welding. Although Q steels are available in strengths down to S460, they tend only to be used in structural applications where strengths beyond the scope of M steels are required, i.e. typically S690 and up to S960.

Weathering steels, with improved atmospheric corrosion resistance, are now available in strengths up to S700. These steels are readily weldable using appropriate welding practices and are being used not only in bridges but also as exposed structural elements in buildings.

As mentioned above, in order to meet strength and toughness requirements, all HSS will generally contain one or a combination of microalloying elements such as niobium (Nb), vanadium (V) or titanium (Ti) at very small additions of < 0.10 % (i.e. < 1000 grams/ tonne). Each individual microalloy performs a specific metallurgical role. For example, the addition of niobium is made to produce a finer-grained steel which not only increases the yield strength but also delivers better low temperature toughness. In turn, this lowers the carbon content of the steel and thus improves weldability.

Molybdenum is often added, in addition to standard microalloying, to make high strength steels which meet additional performance requirements. This is the case for plate products supplied in the Q condition or generally when yield strength above 500 MPa, particularly demanding combinations of strength and toughness, or increased wear resistance are to be achieved. The molybdenum addition is usually in the range of 0.1 to 0.7 %; the actual amount increases with product strength and thickness. Increased molybdenum content is especially beneficial for avoiding excessive heat affected zone softening in weld areas.

Appendix A gives a detailed description of the production processes for HSS products.

Figure 1.1 shows the historical development of the processes for producing rolled steel products to present day, extending an earlier chart by Willms[7].

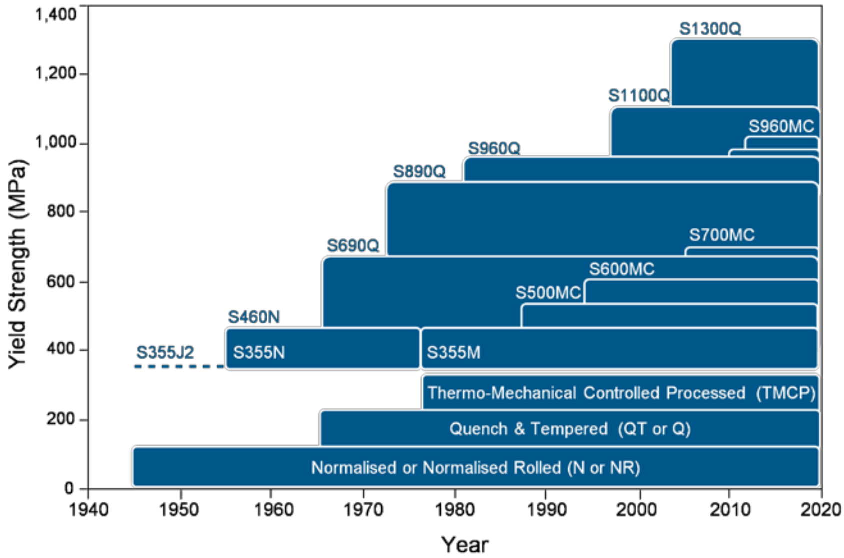


*Figure 1.1 Historical development of production process for rolled steel products*

## 1.4 Overview of applications

### 1.4.1 Buildings

The use of HSS for relatively stocky columns can lead to useful weight savings. For example, a weight reduction of 15 - 25 % can be realised by replacing an S355 steel with an S460 steel for buildings beyond 4 to 7 storeys, depending on load levels. The weight saving increases with the applied loading.

Composite high strength steel-concrete structural members can also be very economic solutions. Composite action is developed by filling a hollow section with concrete, or encasing a section in concrete, and it reduces the tendency for the slender HSS member to local and lateral torsional buckling.

In car parks, HSS are used for beams as part of the composite floor system, and also for columns.

HSS reinforcing bar with yield strength of 600 MPa is also available for use in cores, floors and foundations.

HSS are used in buildings for lateral stability systems, transfer beams and bracing. Figure 1.2 shows the transfer structure at the Edinburgh St James development which involved the use of S460 sections.

*Figure 1.2 Transfer structure at Edinburgh St James development*

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HSS are also used in the light steel framing sector. C section profiles in wall panels tend to have steel thicknesses 1.0 or 1.2 mm for housing and 1.6 to 2 mm for residential buildings. S350, S390 and S450 steel grades to EN 10346[8] are most commonly used, but S550 steel is used for housing in Australia, which leads to a reduction in steel thicknesses down to 0.7 mm. Steel decking profiles, widely used in composite floors in steel framed buildings, are stiffened and embossed to develop good composite action with the concrete placed on it. The current range of deck profiles uses S280 to S450 steel grades. Applications for higher strength strip steels are those in which load bearing capacity is important, and where control of deflections is not so crucial, for example, for load-bearing walls rather than floors. The use of thinner steel may lead to secondary advantages in terms of the construction operation, such as lower weights, ease of fixing etc.

### 1.4.2 Bridges

The following specific benefits may result from the use of HSS in bridges:

- lower permanent loads and hence lower foundation bearing forces (very important for the replacement of bridge decks on existing piers and abutments),
- fewer girders (greater spacing between them),
- shallower girders to solve vertical clearance problems,
- increased spans to reduce the number of piers on land or obstructions in water,
- easier to weld splices (due to reduced material thickness).

Additionally, the weight of a moveable bridge (for example to allow passage for boats), governs the design of the mechanical parts. Hence for these types of bridges, HSS allows cost savings which far exceed the cost savings from a reduction in material.

HSS are most likely to be economic solutions for structural members in bridges where stresses are high, local or lateral-torsional buckling is not critical and fatigue and fracture do not govern design.

Appropriate applications for HSS include:

- Supports of continuous composite bridges (i.e. highly stressed negative bending regions),
- Chords and bracing members of heavily loaded trusses, such as road and rail through bridges,
- Pylons and cables in cable stayed bridges,
- Composite concrete-filled columns supporting bridges.

In bridges, HSS are used for the tension zones of long spans and for truss bridges, where self-weight is the dominant load condition. In the US and Sweden, hybrid bridge girders have been constructed. S460M was used in the Millau-Viaduct in France where 43000 t of steel plate were used in thickness up to 80 mm for the entire central box and some connecting elements.

Many bridges have stringent toughness requirements at low temperatures, which may require the use of steels with enhanced toughness.

Theoretical cost comparisons showed that the benefit of HSS plate girders in bridges increases as the span and load increases: the most economic solutions were combinations of steel grades where the steel used for the web was weaker than the grade used for the flange[9]. HSS can also be used in the web close to the supports where the shear stresses are high. An alternative hybrid solution is to use conventional strength steel for all webs and positive moment top flanges and HSS for negative moment top flanges and all bottom flanges (i.e. the flanges in tension).

The Zandhazen railway bridge is the longest railway arch bridge in Europe. It spans 255 m and is 55 m in height (Figure 1.3). S460M was used for the box sections that form the arches and main girders because of the need to minimise the self-weight of the structure for ease of transportation and installation. As neither fatigue strength nor stiffness governed the design of the main structural elements, the use of S460 led to a total weight reduction of approximately 30 %. In certain places, the choice of S460 enabled a reduction in plate thickness from 160 mm down to 90 mm[10].

*Figure 1.3 Zandhazen railway bridge*

*Figure 1.4 Northern Spire bridge*

© Sunderland City Council

The Northern Spire bridge in Sunderland, which opened in 2018, is a two span cable stayed bridge with an A frame pylon of 105 m height designed by Buro Happold (Figure 1.4). The main deck hybrid girders have S460 flanges and S355 webs.

High strength weathering steel with a yield strength of 485 MPa was used in the 387 m long cable stayed Haliç Metro swing bridge at the Golden Horn, Istanbul, Turkey, which was opened in 2014.

### 1.4.3 Stadia and arenas

HSS are most suitable for tension members such as the bottom chords in a truss, and for compression members with short buckling lengths, such as some top chords in a truss. The use of S460 steel generally allows a weight reduction exceeding 15 % when compared to a solution using S355 steel. This reduction in weight is a function of the truss span and the relative magnitude of the permanent (dead) loads compared to the variable (imposed) loads; generally, the structure deadweight is a considerable proportion of the design load, and so a reduction in deadweight is of great value. Additionally, less stringent deflection limits apply for long span trusses used in stadia and arenas because the overall height of the building is large, and stiffness can be increased by increasing truss depth. A software for designing and optimizing tubular trusses is available at https://www.ssab.com/products/steel-categories/framecalc-user. This tool allows designers to optimize the truss with key parameters such as geometry, loads and steel grades, making it easy to compare different steel grades in trusses and evaluate the benefits of HSS.

In the STIGA Sports Main Arena in Sweden, the roof trusses span 52 m (Figure 1.5). An optimisation process showed that the most cost-effective solution was to use S355 for the braces and S700 steel for the bottom chord, which is predominantly in tension. Since the top chord is mainly in compression, it was not possible to fully exploit the benefits of a higher strength and the use of S420 gave a compromise. The total weight of the trusses was 100 t, notably lighter than the solution using conventional strength steel[11].

*Figure 1.5 Roof trusses in Stiga Sports Arena, Sweden*

© Svante Lundbäck SE

### 1.4.4 Tension bars

HSS bars are widely used in threaded tension bar systems, typically in strengths of S460 and S520, for applications including post tensioning, ground engineering, tension structures and glass facades. Figure 1.6 shows the roof of the Tizi Ouzou Stadium in Algeria, designed by Atak Engineering, which uses M85 S520 tie rods to support the roof.

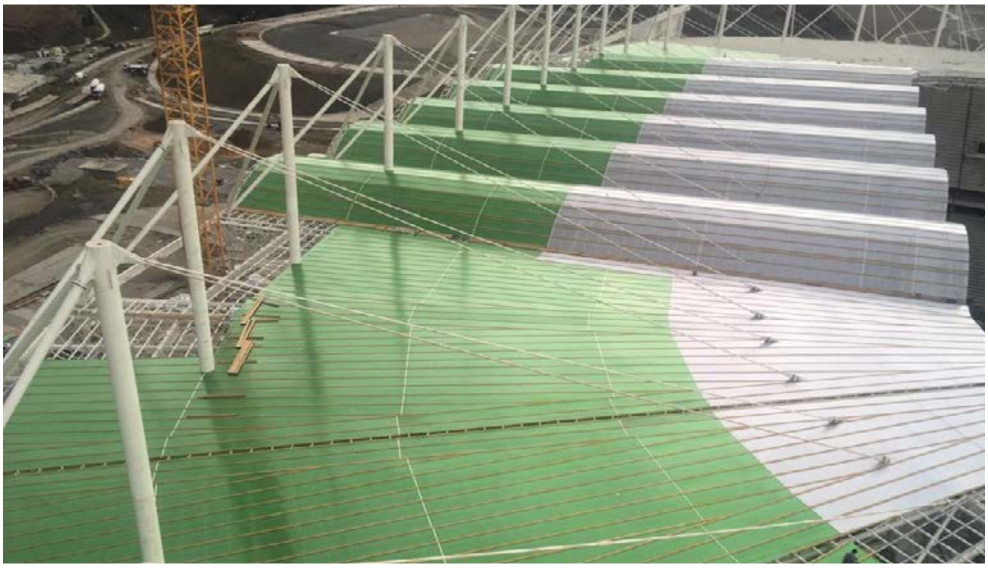


*Figure 1.6 Macalloy Tension Bars supporting the Tizi Ouzou Stadium, Algeria*

© Macalloy and Atak Engineering

### 1.4.5 Foundations

Steel sheet piles are hot-rolled sections, manufactured in accordance with EN 10248[12]. The generally recognised ‘base grade’ for sheet piles is S355GP. However, stronger grades, such as S390GP and S430GP, are now available which has prompted the introduction of S460GP into the next edition of EN 10248. A HSS pile section enables a smaller pile to be used, as long as serviceability requirements are still met, which makes installation easier. A stronger pile also assists with durability. If, over the life of a marine sheet pile wall, corrosion is expected to reduce pile thickness, use of a higher grade means that the residual strength of the wall is enhanced. This can either facilitate more economic design or longer design life for the structure.

Higher strength steel bearing piles (to support vertical loads, where ground conditions preclude the use of shallow foundations) are advantageous as section thicknesses can be reduced when considering pile in-place design and during pile driving. Bearing piles are available as Universal bearing H-sections, tubular sections and box sections.

Steel grade S355J2H according to EN 10219-1 is the most common material for tubular sections, but steel grades up to S460MH are used especially in tubular piles with diameters up to 300 mm. Use of even higher steel grades for tubular piles as bearing piles and as primary elements in combined walls is often beneficial and some manufacturers have European Technical Assessment (ETA) approval for up to S550J2H steel grade.

Steel line pipe is also used for bearing piles, produced to API 5L[13] up to HSS grades X80 (yield strength of approximately 550 MPa).

Applications where HSS is particularly beneficial includes deep foundations, retaining walls in deep ports and foundations in congested sites, where the reduction in the required footprint of the foundation is very useful.

Steel anchors are used for retaining structures such as retaining walls, quay walls and bridge abutments. The anchors provide tie back to the surrounding soil, thereby restricting horizontal deflection of walls and reducing bending moments in the wall. A form of anchor commonly used is tie rods. Tie rods are manufactured from round steel bar with forged or threaded ends that allow a variety of connections to be made to sheet piles, tubes, H-piles, combi walls and diaphragm walls. A range of products with strengths varying from 500 to 700 MPa are used in accordance with standards such as EN 10025-3 &-6, EN 10149-2[2], EN ISO 898-1[14] and EN ISO 683-2[15]. Whilst the higher steel grades will always produce the lightest weight anchor, they may not be suitable for stiffness requirements (horizontal deflections at Serviceability Limit States) or on-site welding.

### 1.4.6 Temporary structures

HSS are also used for temporary structures where the combination of high strength and low weight is valuable, such as supporting frameworks during tunnel construction, in mining or to provide stability for structures during the erection process.

The Verrand Viaduct in Italy is an orthotropic deck bridge and part of the Mont Blanc-Aosta highway. The lattice launch girder used for the construction of the bridge was 85 m long and made of S690 steel tubular sections[16]. The use of HSS meant that no additional strengthening of the members in the bridge deck was required for the construction phase[17].

### 1.4.7 High strength steel bolts

Steel bolts of class 8.8 and 10.9 are usually used as the connectors in bolted joints between high strength steel members. They are high strength quenched and tempered steels.

## 1.5 Scope of this guide

This guide gives information on design and execution of structures made from steels of strength 420 MPa up to 700 MPa. For comparison purposes, steels with a yield strength of 355 MPa are used as a reference point.

HSS are fairly commonly used in demanding applications on offshore structures, for example jack-up rigs, However, these types of structure are generally designed and fabricated to standards specific to offshore applications and so are outside the scope of this guide. Structures associated with the wind generation industry are also outside the scope.

The guide is aligned to European product, design and execution standards, but as far as possible an international perspective is given.

# SPECIFICATION

## 2.1 Introduction

The designation system adopted for steels in European product standards is as follows:

- S indicates a structural steel
- Minimum specified yield strength for thickness ≤ 16 mm, expressed in MPa (and therefore N/mm$^2$)
- Delivery condition: N, M or Q
- Steel quality
  - No symbol indicates minimum impact energy at temperature not lower than -20 °C;
  - L or L1 indicates specified minimum impact properties at temperature not lower than -50 °C for N and M steels;
  - L or L1 indicates minimum specified values of impact energy at temperatures not lower than -40 °C and -60 °C for Q steels
- H indicates hollow sections;
- C indicates the steel is suitable for cold forming in accordance with minimum defined bend radii.

For example:

1. S690QL1 is a structural steel (S) in accordance with EN 10025-6 with a specified minimum yield strength at ambient temperature of 690 MPa (690), in the quenched and tempered (Q) and with a specified minimum value of impact energy at -60 °C.
2. S420NC is a structural steel (S) in accordance with EN 10025-3 with a specified minimum yield strength at room temperature of 420 MPa (420) in the normalized or normalized rolled condition (N) suitable for cold forming (C).
3. S420MLH is a hollow section made of structural steel (S) in accordance with EN 10219-2 with a specified minimum yield strength for thickness not greater than 16 mm of 420 MPa (420), thermomechanical rolled steel (M), with a minimum impact energy value of 27 J at -50 °C (L), hollow section (H).

## 2.2 Hot rolled products of structural steels

EN 10025[1] is the harmonised European structural steel specification for most steelwork in building and civil engineering applications. A very wide range of plates and open sections such as I-sections, channels and angles are made from these steels. This specification covers five different families of structural steels:

| | |
|---|---|
| EN 10025-2 | Non-alloy structural steels |
| EN 10025-3 | Normalized/normalized rolled weldable fine grain structural steels |
| EN 10025-4 | Thermomechanical rolled weldable fine grain structural steels |
| EN 10025-5 | Structural steels with improved atmospheric corrosion resistance |
| EN 10025-6 | Flat products of high yield strength structural steels in the quenched and tempered condition |

HSS from S420 to S960 are specified to EN 10025-3, EN 10025-4 and EN 10025-6, depending on the delivery conditions of the material. N and M steels are available in strength grade S420. N, M and Q steels are available in strength grade S460. Higher strength grades from S500 to S960 are only produced through the quenched and tempered process.

Part 5 of EN 10025 specifies the requirements for weathering steels (indicated by the letter ‘W’) in strengths up to S460 with a range of qualities from J0 to J5 (see Section 3.6.4).

In addition to products meeting the requirements of EN 10025, steel producers can also make customer-specific steel products with improved properties, such as improved weldability or other improved fabrication properties.

## 2.3 Hollow sections

### 2.3.1 Cold formed hollow sections

EN 10219[4] is the harmonised standard for cold formed welded structural hollow sections of circular, square or rectangular forms and applies to hollow sections formed cold without subsequent heat treatment. These hollow components are used in a wide range of construction and mechanical related applications. EN 10219 consists of two parts, Part 1 gives technical delivery conditions and Part 2 gives tolerances, dimensions and section properties. The standard covers S460NH/NLH, S420MH/MLH and S460MH/MLH cold formed hollow sections, as well as other conventional strength steel sections. However, structural hollow sections in strengths above S460, meeting the requirements of EN 10219 where applicable, have been available for many years. This has prompted the development of a new Part of EN 10219 (EN 10219-3) which is due to be published and will cover steels above S460 and up to S960, as well as weathering steels.

### 2.3.2 Hot finished hollow sections

EN 10210[3] is the harmonised standard for hot finished hollow sections of circular, square, rectangular and elliptical forms. It applies to hollow sections formed hot, with or without subsequent heat treatment, or formed cold with subsequent heat treatment above 580 °C to obtain equivalent metallurgical conditions to those obtained in the hot formed products. These hollow components are used for all structural and mechanical applications, including multi-storey columns, space frames, lattice beams and frames for machinery and trailers. EN 10210 consists of two parts - Part 1 gives technical delivery conditions and Part 2 gives tolerances, dimensions and section properties. It covers S420NH/NLH and S460NH/NLH steels, among other conventional strength steel sections. However, structural hollow sections in strengths above S460, meeting the requirements of EN 10210 where applicable, have been available for many years and a new Part 3 of EN 10210 (EN 10210-3), due to be published soon, will cover steels above S460 and up to S960, as well as weathering steels.

## 2.4 Hot rolled flat products for cold forming

EN 10149[2] is the European standard specifying requirements for flat products made of weldable, hot-rolled, high yield strength steels for cold forming. It consists of three parts:

| | |
|---|---|
| EN 10149-1 | General technical delivery conditions |
| EN 10149-2 | Technical delivery conditions for thermomechanical rolled steels |
| EN 10149-3 | Technical delivery conditions for normalized or normalized rolled steels |

Part 2 is applicable to products in thicknesses from 1.5 mm to 20 mm in strength grades up to S460MC and in thicknesses from 1.5 mm to 16 mm for S500MC to S700MC. It also covers steels up to S960MC for thicknesses up 10 mm. Part 3 is applicable to products in thicknesses from 1.5 to 20 mm up to 420NC.

## 2.5 Bars and rods

Hot rolled round and square steel bars should comply with EN 10060[18] and EN 10059[19] with regards to the requirements of dimensions and tolerances on shape. The steel grades should comply with EN 10025 or EN ISO 683[20].

## 2.6 Products covered by ETAs

Some HSS products are not yet included in a harmonised product standard but instead are covered by a European Technical Assessment (ETA). An ETA gives information on the performance assessment of the product and gives manufacturers a voluntary way to CE-mark a construction product. An ETA can be found on the EOTA website by searching for the ETA number.

### 2.6.1 Tension bars

The high strength round bar Macalloy tension rod system is covered by ETA-17/0849[21]. The diameter of the bars ranges from 10 mm to 100 mm.

### 2.6.2 Hot rolled long steel products

An example of one of the products available from producers in Europe is the HISTAR® hot finished high strength range of I sections, produced by ArcelorMittal. They are available in S355M/ML and S460M/ML, covered by ETA-10/0156[22]. The ETA gives the same mechanical, chemical and toughness properties as EN 10025-4, but the design strength for HISTAR® S460 does not reduce until the thickness is greater than 100 mm.

### 2.6.3 Hot rolled long steel products

ETA-12/0526[23] covers tubular piles made by SSAB and includes steel grades up to S550J2H. The range of pile sizes is from micropiles with diameter of 76.1 mm to large diameter piles with diameters up to 1220 mm. The ETA includes both longitudinally welded piles and spirally welded piles manufactured according to EN 10219.

## 2.7 Availability of product forms

HSS products for structural and civil engineering applications have been available for many years, from steel producers across the world. This section presents an overview of the availability of HSS products in the current market. Procurement of HSS products may differ compared to conventional strength steel products, and for large orders, the most efficient procurement route will be directly from the steel mill. Minimum order sizes and longer lead times are to be expected. The availability of the products is also dependent on the steel grade and sub grade. It should be noted that the steel grades given in the product standards are not all available. Some steel products are not covered by a product standard. Some products may exceed the mechanical properties required by the relevant standard.

The choice of whether to specify an M or N steel is less about requiring a specific mechanical property but more to do with the thickness required and how the steel will be subsequently processed by the client. If for instance the client is looking to perform a relatively complex multi run welding process and needs to undertake either preheating or multiple stress relieving cycles on the product, as may be the case for thicker steels, then an N steel is most appropriate. However, for thinner materials, for example < 30 mm, an M steel could suffice, omitting the need for any pre- or post-weld heat-treatment. If the client is looking at processing the steel on site and wants the highest strength possible but does not want to carry out any difficult stress relieving or preheating due to difficult site conditions, an M steel should be selected depending on product thickness availability.

If a short lead time is required, M steels are likely to be the best option and the most economic choice since they are widely available and do not require any additional heat treatment process. N and Q steels, on the other hand, generally require a heat treatment process, except for normalized rolled. N steels should be chosen if hot bending is required.

Table 2.1 to Table 2.8 show the availability of different HSS products in the range of S420 to S700 (and up to S960 for plate). In general, S420 and S460 grades are more widely available than higher strength steels. It is recommended to check the availability with prospective steel producers and stockholders during the concept design stage.

*Table 2.1 Available sizes of HSS structural plate*

| Steel Grade | Standard | Steel Quality | Plate thickness [mm] |
|---|---|---|---|
| S420 | EN 10025-3: EN 10025-4 | M, ML | 6.0 – 150 |
| | | N, NL | 6.0 – 250 |
| S460 | EN 10025-3: EN 10025-4; EN10025-6 | M, ML | 6.0 – 200 |
| | | N, NL, Q | 6.0 – 150 |
| S500 | EN 10025-4; EN10025-6 | M, ML | 10.0 – 75 |
| | | Q, QL, QL1 | 6.0 – 200 |
| S550 | EN10025-6 | Q, QL, QL1 | 6.0 – 150 (200*) |
| S620 | EN10025-6 | Q, QL, QL1 | 8.0 – 150 |
| S690 | EN10025-6 | Q, QL, QL1 | 6.0 – 150 (255*) |
| S890 | EN10025-6 | Q, QL, QL1 | 6.0 – 100* |
| S960 | EN10025-6 | Q, QL, QL1 | 6.0 – 120* |
| S1100 | To be classified | Q | 8.0 – 40* |

* Not compliant with EN 10025-6

*Table 2.2 Available sizes of HSS circular hollow sections (CHS)*

| Steel Grade | Steel Quality | Product standard | Outer Diameter [mm] | Wall thickness [mm] |
|---|---|---|---|---|
| S420 | NLH, MH, MLH | EN 10210, EN 10219 | 21.3 – 660.0 | 2.0 – 25.4 |
| S460 | MH, MLH | EN 10219 | 42.0 – 610.0 | 2.0 – 25.4 |
| S500 | MH | EN 10219* | 42.0 – 323.9 | 2.0 – 12.5 |
| S550 | MH | EN 10219* | 42.0 – 323.9 | 2.0 – 12.5 |
| S690 | QH, QLH, QL1H | EN 10210* | 21.3 – 660.0 | 2.3 – 25.0 |
| S700 | MH, MLH, QLH | EN 10219* | 42.4 – 323.9 | 2.0 – 10.0 |

* Outside the current scope of the product standard, but meeting its requirements where applicable.

*Table 2.3 Available sizes of HSS square hollow sections (SHS)*

| Steel Grade | Steel Quality | Product standard | Size [mm] | Wall thickness [mm] |
|---|---|---|---|---|
| S420 | NH | EN 10210 | 40x40 – 400x400 | 3.0 – 25.4 |
| S460 | MH, NLH | EN 10219, EN 10210 | 30x30 – 400x400 | 2.0 – 25.0 |
| S500 | MH | EN 10219* | 40x40 – 300x300 | 2.0 – 12.5 |
| S550 | MH | EN 10219* | 40x40 – 300x300 | 2.0 – 12.5 |
| S690 | QLH, QL1H | EN 10210* | 40x40 – 400x400 | 2.9 – 25.0 |
| S700 | MH, MLH | EN 10219* | 30x30 – 300x300 | 2.0 – 10.0 |

* Outside the current scope of the product standard, but meeting its requirements where applicable.

*Table 2.4 Available sizes of HSS rectangular hollow sections (RHS)*

| Steel Grade | Steel Quality | Product standard | Size [mm] | Wall thickness [mm] |
|---|---|---|---|---|
| S420 | NH | EN 10210 | 50x30 – 500x300 | 3.0 – 25.4 |
| S460 | MH | EN 10219 | 50x30 – 400x200 | 2.0 – 12.5 |
| S500 | MH | EN 10219* | 50x30 – 400x200 | 2.0 – 12.5 |
| S550 | MH | EN 10219* | 50x30 – 400x200 | 2.0 – 12.5 |
| S690 | QLH, QL1H | EN 10210* | 50x30 – 500x300 | 2.9 – 25.0 |
| S700 | MH, MLH | EN 10219* | 50x30 – 400x200 | 2.0 – 10.0 |

* Outside the current scope of the product standard, but meeting its requirements where applicable.

*Table 2.5 Available sizes of HSS sheet and strip to EN 10149-2*

| Steel Grade | Thickness [mm] |
|---|---|
| S420MC | 1.5 – 15 |
| S460MC | 1.5 – 20 |
| S500MC | 1.6 – 15 |
| S550MC | 1.6 – 15 |
| S600MC | 2.0 – 10 |
| S650MC | 2.0 – 10 |
| S700MC | 2.0 – 10 |

*Table 2.6 Available sizes of HSS cold formed open sections*

| Section type | Min. Yield strength [MPa] | Product Standard | Wall thickness [mm] |
|---|---|---|---|
| U section* | 650 | EN 10149-2 (flat material)<br>EN 10162[24] (tolerances) | 2.5 – 10.0 |
| | 700 | | 2.5 – 10.0 |

* Other cold formed open sections made of HSS are available upon request from steel producers.

| Steel grade | Steel Quality | Product standard | Sections | Max. flange thickness [mm] |
|---|---|---|---|---|
| S460 | NH | EN 10025-4<br>ETA-10/0156<br>EN 10365[25] | IPE 550, 600 – 750<br>UB 610 x 229 – 1100 x 400<br>HE 260 – 280, 300 – 1000<br>HL 920 – 1100<br>HD 260 – 400<br>UC 254 x 254 – 356 x 406<br>HP 305 – 400<br>UBP 305 x 305 – 356 x 368 | 140 |
| | MH | | | 125 |

*Table 2.7 Available sizes of HSS hot rolled open sections*

| Product | Product standard | Min yield strength [MPa] | Diameter/ Dimension [mm] |
|---|---|---|---|
| Macalloy 460 tension bar system | ETA-17/0849 | 460 | M10 – M100 |
| Macalloy 520 tension bar system | ETA-17/0849 | 520 | M10 – M100 |
| Ancon tension system | - | 360 – 520 | M8 – M56 |
| Dextra engineered bar systems | - | 460 – 700 | M16 – M162 |

*Table 2.8 Available sizes of HSS tension bar systems*

Stockists can offer hollow sections which may be surplus from oil and gas projects and are specified as API grades (in accordance with API 5L[13]) which are not included in EN standards. However the tubes may still meet the requirements of EN 10210[3] / EN 10219[4]. Table 2.9 lists these steels and their yield strengths for reference.

| Steel Grade | Yield strength [MPa] |
|---|---|
| API X52 | 360 |
| API X56 | 390 |
| API X60 | 415 |
| API X65 | 450 |
| API X70 | 485 |
| API X80 | 555 |

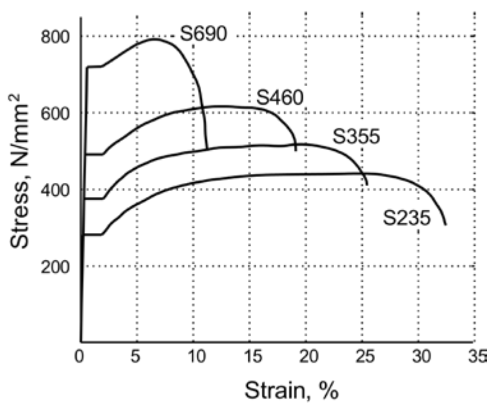
*Table 2.9 Nominal values of yield strength for API line pipe*

# PROPERTIES

## 3.1 Mechanical properties

### 3.1.1 Stress-strain characteristics

The different chemical composition and heat treatment processes lead to different stress-strain characteristics for HSS compared to those for conventional strength steel. Figure 3.1 demonstrates the typical changing stress-strain behaviour with increasing steel strength for HSS hot rolled products. As the strength increases, ductility reduces, although this reduction is not significant enough to affect the design of the majority of structures. The ratio of ultimate to yield strength also reduces as the strength increases, which leads to a few design rules requiring modification.




Note: The curves represent material in the following supply conditions:
S235 & S355: as rolled
S460: normalized
S690: quenched & tempered

*Figure 3.1 Full range stress-strain curves for HSS hot rolled products*

The stress-strain characteristics of cold formed and thermomechanical processed HSS products tend to be more rounded than those for hot rolled products, and do not show a sharp yield point but rather a gradual yielding. For materials without a clear yielding point, for design purposes, the yield strength is conventionally taken as the 0.2 % proof strength.

In design calculations, the characteristic yield strength $f_y$ and characteristic ultimate strength $f_u$ are taken as the minimum specified values for the yield strength ($R_{eH}$) and tensile strength ($R_m$) given in the relevant steel product standards. The following sub-sections give the mechanical properties of HSS flat and hollow products in accordance with European standards.

The modulus of elasticity of steel is independent of the strength and is taken as 210000 MPa for structural design. It is important to note that the minimum specified strength tends to reduce with an increase in material thickness. This is because more alloying elements are needed to achieve the required mechanical properties in thicker sections. Some of these alloying elements have to be limited at the expense of a slight reduction in strength because their addition increases the carbon equivalent value (Section 10.2.1), which in turn reduces the weldability.

In order to design structures made of HSS that can resist high speed impact or blast loads, it can be assumed that HSS have the same strain rate hardening characteristics as conventional strength steel[26].

### Structural plates and open sections

Table 3.1 to Table 3.3 present the nominal values of the yield strength $f_y$ and the ultimate strength $f_u$ for HSS specified to EN 10025[1]. The minimum requirements for ductility of the material are also presented. They cover structural steel plates delivered in N, M and Q conditions.

*Table 3.1 Nominal mechanical properties for normalized or normalized rolled HSS up to 100 mm thick (EN 10025-3)*

| Steel grade | Minimum yield strength $R_{eH}$ [a] [MPa] Nominal thickness [mm] | | | | | Tensile strength $R_m$ [a] [MPa] Nom. thickness [mm] | Minimum percentage elongation after fracture [a] $L_0 = 5.65\sqrt{S_0}$ Nominal thickness [mm] | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|
| | ≤ 16 | > 16 ≤ 40 | > 40 ≤ 63 | > 63 ≤ 80 | > 80 ≤ 100 | ≤ 100 | ≤ 16 | > 16 ≤ 40 | > 40 ≤ 63 | > 63 ≤ 80 | > 80 ≤ 100 |
| S420N/NL | 420 | 400 | 390 | 370 | 360 | 520 to 680 | 19 | 19 | 19 | 18 | 18 |
| S460N/NL | 460 | 440 | 430 | 410 | 400 | 540 to 720 | 17 | 17 | 17 | 17 | 16 |

[a] For plate, strip and wide flats with width ≥ 600 mm the direction transverse (t) to the rolling direction applies. For all other products the values apply for the direction parallel (l) to the rolling direction

*Table 3.2 Nominal mechanical properties for thermomechanical rolled HSS up to 100 mm thick (EN 10025-4)*

| Steel grade | Minimum yield strength $R_{eH}$ [a] [MPa] Nominal thickness [mm] | | | | | Tensile strength $R_m$ [a] [MPa] Nominal thickness [mm] | | Minimum percentage elongation after fracture [b] $L_0 = 5.65\sqrt{S_0}$ |
|---|---|---|---|---|---|---|---|---|
| | ≤ 16 | > 16 ≤ 40 | > 40 ≤ 63 | > 63 ≤ 80 | > 80 ≤ 100 | ≤ 40 | > 40 ≤ 63 | |
| S420M/ML | 420 | 400 | 390 | 380 | 370 | 520 to 680 | 500 to 660 | 19 |
| S460M/ML | 460 | 440 | 430 | 410 | 400 | 540 to 720 | 530 to 710 | 17 |
| S500M/ML | 500 | 480 | 460 | 450 | 450 | 580 to 760 | 580 to 760 | 15 |

[a] For plate, strip and wide flats with widths ≥ 600 mm the direction transverse (t) to the rolling direction applies. For all other products the values apply for the direction parallel (l) to the rolling direction.
[b] For product thickness < 3 mm for which test pieces with a gauge length of $L_0$ = 80 mm shall be tested, the values shall be agreed at the time of the order.

*Table 3.3 Nominal mechanical properties for quenched and tempered HSS (EN 10025-6)*

| Steel grade | Minimum yield strength $R_{eH}$ [MPa] Nominal thickness [mm] | | Tensile strength $R_m$ [MPa] Nominal thickness [mm] | | Minimum percentage elongation after fracture $L_0 = 5.65\sqrt{S_0}$ |
|---|---|---|---|---|---|
| | > 3 ≤ 50 | > 50 ≤ 100 | > 3 ≤ 50 | > 50 ≤ 100 | |
| S460Q/QL/QL1 | 460 | 440 | 550 to 720 | | 17 |
| S500Q/QL/QL1 | 500 | 480 | 590 to 770 | | 17 |
| S550Q/QL/QL1 | 550 | 530 | 640 to 820 | | 16 |
| S620Q/QL/QL1 | 620 | 580 | 700 to 890 | | 15 |
| S690Q/QL/QL1 | 690 | 650 | 770 to 940 | 760 to 930 | 14 |

### Hollow sections

High strength cold formed welded hollow sections of S420 and S460 are covered by EN 10219[4]. Two delivered conditions are available: N and M. The mechanical properties of the cold formed high strength hollow sections are presented in Table 3.4. The maximum wall thickness is limited to 40 mm for all hollow products.

*Table 3.4 Nominal mechanical properties for HSS cold formed hollow sections in thickness ≤ 40 mm – Feedstock material condition N and M (EN 10219)*

| Steel grade | Minimum yield strength $R_{eH}$ [MPa] Specified thickness [mm] | | Tensile strength $R_m$ [MPa] Specified thickness [mm] | Minimum percentage elongation $A$ [a, b] |
|---|---|---|---|---|
| | ≤ 16 | > 16 ≤ 40 | ≤ 40 | |
| S460NH/NLH | 460 | 440 | 540 to 720 | 17 |
| S420MH/MLH | 420 | 400 | 500 to 660 | 19 |
| S460MH/ MLH | 460 | 440 | 530 to 720 | 17 |

[a] For section sizes D/T < 15 (circular) and (B+H)/2T < 12.5 (square and rectangular) the minimum elongation is reduced by 2.
[b] For thicknesses < 3 mm, see EN 10219-2 clause 9.2.2.

High strength hot finished hollow sections in S420 and S460 are covered by EN 10210[3] in the normalized delivery condition. Their mechanical properties are presented in Table 3.5.

*Table 3.5 Nominal mechanical properties for normalized/ normalized rolled HSS hollow sections (EN 10210)*

| Steel grade | Minimum yield strength $R_{eH}$ [MPa] at specified thickness [mm] | | | Tensile strength $R_m$ [MPa] at specified thickness [mm] | Minimum percentage elongation $A$ at specified thickness ≤ 65 mm | |
|---|---|---|---|---|---|---|
| | ≤ 16 | > 16 ≤ 40 | > 40 ≤ 65 | ≤ 65 | Longitudinal | Transverse |
| S420NH/NLH | 420 | 400 | 390 | 520 to 680 | 19 | 17 |
| S460NH/NLH | 460 | 440 | 430 | 540 to 720 | 17 | 15 |

### Sheet and strip

HSS sheet products for cold forming are available from S420 to S700. These hot rolled flat products made of high yield strength steel suitable for cold forming can be delivered in thermomechanical rolled (S420MC to S700MC) and normalized rolled (S420NC) conditions. Products up to 20 mm thick are covered by EN 10149[2], with the maximum permissible thickness being related to the steel grade. The mechanical properties are presented in Table 3.6.

*Table 3.6 Nominal mechanical properties hot rolled HSS for cold forming (EN 10149)*

| Steel grade | Minimum yield strength $R_{eH}$ [MPa] a | Tensile strength $R_m$ [MPa] a | Minimum percentage elongation after fracture a<br>Nominal thickness [mm]<br>< 3<br>$L_0$=80 mm | <br><br>≥ 3<br>$L_0 = 5.65\sqrt{S_0}$ | Bending at 180° minimum mandrel diameter b, c | Permissible thickness [mm] |
|---|---|---|---|---|---|---|
| S420MC | 420 | 480 – 620 | 16 | 19 | 0.5$t$ | 1.5 to 20 |
| S460MC | 460 | 520 – 670 | 14 | 17 | 1$t$ | |
| S500MC | 500 | 550 – 700 | 12 | 14 | 1$t$ | 1.5 to 16 |
| S550MC | 550 | 600 – 760 | 12 | 14 | 1.5$t$ | |
| S600MC | 600 | 650 – 820 | 11 | 13 | 1.5$t$ | |
| S650MC | 650 d | 700 – 800 | 10 | 12 | 2$t$ | |
| S700MC | 700 d | 750 – 950 | 10 | 12 | 2$t$ | |
| S420NC | 420 | 530 – 670 | 18 | 23 | 0.5$t$ | 1.5 to 20 |

a The values for the tensile test apply to longitudinal test pieces except for S420 NC for which the values for the tensile test apply to longitudinal test pieces for product width < 600 mm and to transverse test pieces for product width ≥ 600 mm
b The values for the bend test apply to transverse test pieces
c $t$ = thickness in mm for test piece for bend test
d For thickness > 8 mm the minimum yield strength can be 20 MPa lower

## 3.1.2 Constitutive model of HSS for FE analysis

A new part of Eurocode 3, EN 1993-1-14 is under preparation with a likely publication date of 2025, which covers the design of steel structures assisted by finite element analysis[27]. EN 1993-1-14 specifies the following constitutive material models for HSS up to S700:

- For HSS exhibiting a sharply defined yield point and yield plateau, the material model for hot-rolled steels may be adopted. It is recommended to use a multi-linear elastic-plastic model for HSS S420 and S460.
- For HSS that exhibit a more rounded stress-strain curve, typically S500–S700, the two-stage Ramberg-Osgood model can be used, which is also recommended for cold formed steels and stainless steels.

Appendix B gives the expressions for the different material models, which are based on the 2019 draft of EN 1993-1-14.

## 3.2 Physical properties

HSS have the same physical properties as conventional strength steel, as given in Table 3.7.

| Physical properties | Value |
|---|---|
| Density, $\rho$ | $7.85 \times 10^3$ kg/m$^3$ |
| Poisson's ratio in elastic range, $\upsilon$ | 0.3 |
| Modulus of elasticity, $E$ | 210000 MPa (at 20 °C) |
| Shear modulus, $G$ | $\frac{E}{2(1+\upsilon)} \approx$ 81000 MPa |
| Coefficient of linear thermal expansion, $\alpha$ | $12 \times 10^{-6}$ per K (for $T \leq 100$ °C) |
| Thermal conductivity, $\lambda_a$ | 53.3 W/mK (at 20 °C) |
| | 47.3 W/mK (at 200 °C) |
| | 40.7 W/mK (at 400 °C) |

*Table 3.7 Physical properties of steels*

## 3.3 Effects of temperature

Structural steels display a reduction in ductility as the temperature drops below ambient temperatures, while the yield and ultimate strengths increase. The elastic modulus of steels increases slightly at lower temperatures but this can be disregarded for structural design. Generally, the toughness of steels decreases as the temperature reduces and the susceptibility to brittle fracture increases. HSS (both M and Q steels) have improved toughness properties compared to conventional strength steel grades even at very low temperatures (i.e. transition to brittle fracture occurs at lower temperatures than for conventional steel grades). The selection of the appropriate steel to avoid brittle fracture at low temperature is discussed in Section 3.4.

HSS, like all steels, lose stiffness and strength at elevated temperatures. The design of HSS in fire is covered in Section 7.

## 3.4 Fracture toughness

In certain circumstances, steel can behave in a non-ductile manner – failure occurs suddenly without plastic deformation – this is commonly referred to as brittle fracture. The risk of brittle fracture in steelwork can be minimised by the specification of an appropriate steel grade with sufficient fracture toughness for the given application and service conditions.

### 3.4.1 Risk factors

The risk of a brittle fracture for an individual component or element depends on factors that cannot necessarily be precisely known – actual material toughness, actual flaw size, actual residual stresses local to the flaw – it follows that design against brittle fracture must be based on achieving a certain level of reliability (a defined probability of failure), based on calibrations for the various factors that influence the likelihood of brittle fracture. The principal risk factors are discussed below:

#### Material temperature

As mentioned previously, temperature affects the stress-strain behaviour of steel. At room temperature, most steels are able to exhibit a significant amount of plasticity beyond the elastic range, achieving stresses larger than the yield stress. At low temperatures, on the other hand, fracture may occur within the elastic range before the attainment of the room temperature yield stress, resulting in a sudden or 'brittle' failure. The difference in behaviour is expressed diagrammatically in Figure 3.2, known as the ductile-brittle transition curve.

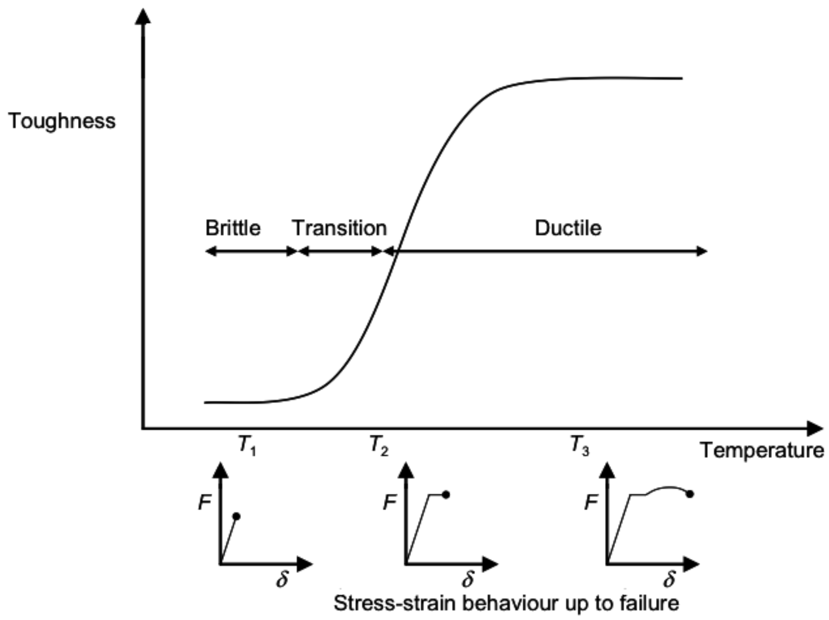



*Figure 3.2 Variation of toughness with temperature*

There is no sudden step between brittle behaviour and ductile behaviour. Instead the change from one type of behaviour to the other takes place over a temperature range known as the transition range. A transition range at lower temperatures indicates a 'tougher' material.

#### State of stress

High tensile stresses increase the risk of brittle fracture and therefore elements entirely in permanent compression are at reduced risk. Since the likelihood of brittle fracture depends on the net value of residual and applied stress, the level of residual stresses is also important.

### Material thickness

The toughness of steel generally decreases toward the middle of thick material. Additionally, the notional flaw size that is allowed for in design rules depends on thickness. The consequence is that thicker material needs to have a greater toughness for the same level of reliability against brittle fracture.

### Local details

Local detail geometries leading to high stress concentration influence both the initial flaw size that needs to be considered and the local stress level at the detail. Generally, details that would have a lower fatigue life are also more susceptible to brittle fracture. For example, welded details are more susceptible than bolted details. Sharp corners or 'hard points' at the connection of one member to another may also increase the susceptibility to brittle fracture.

### Cold forming

If material is cold formed (curved, rolled or pressed into different cross-sections or member shapes), the material is taken beyond the yield stress as part of the forming process. At high strain levels, this reduces the local plastic strain capacity between yield and fracture, thus increasing the susceptibility to brittle fracture.

### Strain rate

At high strain rates, the susceptibility to brittle fracture increases. For parts subject to strain rates higher than $4 \times 10^{-4}$ $sec^{-1}$ which is commonly assumed in 'static' design, an allowance must be made for this effect. Although high strain rates are uncommon in steel buildings, there are some elements which might be subject to high strain rates due to, for example impact loading, etc.

In the context of HSS, the brittle fracture risk factors are equally applicable.

## 3.4.2 Specification of toughness

A convenient measure of the fracture toughness of steel is given by the Charpy V-notch impact test (hence the commonly used terms 'notch toughness' and 'Charpy value'). The Charpy toughness requirements given in the European product standards for HSS are given below:

### EN 10025-3, EN 10025-4, fine grain steels to EN 10210-1 and EN 10219-1

Steels to EN 10025-3 and EN 10025-4 and hollow sections to EN 10210-1 and EN 10219-1 may be one of two sub-grades. The required impact energy for each toughness grade is:

- M, N minimum 40 J impact energy at -20 °C (equivalent to 27 J at -30 °C)
- ML, NL minimum 27 J impact energy at -50 °C

### EN 10025-6

HSS steels to EN 10025-6 may be one of three toughness grades. The required impact energy for each toughness sub-grade is:

- Q minimum 30 J impact energy at -20 °C
- QL minimum 30 J impact energy at -40 °C
- QL1: minimum 30 J impact energy at -60 °C

The 30 J requirement at the test temperature is taken, for design purposes, to be the same as the 27 J requirement for other sub-grades given in the different product standards.

The range of possible toughness designations is summarized in Table 3.8.

*Table 3.8 Steel toughness designations*

| Sub grade | Energy absorption (minimum) | |
|---|---|---|
| Q | 30 J at -20 °C | |
| M, N | 40 J at -20 °C / 27 J at -30 °C | |
| QL | 30 J at -40 °C | Increasing resistance to brittle fracture |
| ML, NL | 27 J at -50 °C | |
| QL1 | 30 J at -60 °C | |

## 3.4.3 Design basis for brittle fracture

In Europe the design requirements to avoid brittle fracture are set out in EN 1993-1-10[28]. Brittle fracture is considered to be an ‘accidental combination’ of actions and the effects of actions appropriate to that combination are expressed in EN 1993-1-10, clause 2.2 as:

$$E_{\mathrm{d}} = E\{A[T_{\mathrm{ED}}] + \sum G_{\mathrm{K}} + \psi_1 Q_{\mathrm{K1}} + \sum \psi_{2,i} Q_{\mathrm{K}i}\}$$

where ‘+’ means ‘combined with’

This combination of actions should be read as the combined effect of:

- The leading action $A$ which is $T_{\mathrm{ED}}$, the reference temperature (which influences the toughness of the material and might lead to effects due to restraint of movement)
- The characteristic value of permanent actions, $G_{\mathrm{K}}$
- $\psi_1$ × the characteristic value of the leading variable action, $Q_{\mathrm{K1}}$
- $\psi_{2,\mathrm{i}}$× the characteristic value of any accompanying variable actions, $Q_{\mathrm{Ki}}$

The combination factors $\psi_1$ and $\psi_{2,\mathrm{i}}$ should be in accordance with EN 1990[29].

The stress under this combination is calculated as an indicator of the susceptibility to brittle fracture.

### 3.4.4 Design methodology

Codes of practice for the design of structural steelwork generally aim to reduce the risk of brittle fracture for the given service conditions through the specification of steel sub-grades that exhibit a sufficiently high level of fracture toughness.

EN 1993-1-10 gives two possible approaches:

- Maximum permitted thickness values, or
- Fracture mechanics evaluation

The maximum permitted thickness approach is a simplified design methodology based upon fracture mechanics principles.

Fracture mechanics considers the interaction between the applied tensile stress, the size of any flaw present and the material's fracture toughness, which enables engineers to assess the risk of brittle fracture for the given service conditions. There are a number of equivalent fracture mechanics based approaches:

- The Energy Criterion approach, which states that crack extension (i.e. fracture) occurs when the energy available for crack growth is sufficient to overcome the resistance of the material, which might include surface energy, plastic work or other type of energy dissipation associated with crack propagation, or
- The stress intensity ($K$) approach, which characterises the stress intensity at the tip of a crack in an elastic material. If it is assumed that failure occurs at some critical combination of stress and strain, it follows that fracture must occur at a critical stress intensity, $K_{Ic}$. As such, the critical stress intensity is a measure of the material's fracture toughness.

In its simplest form stress intensity $K_I$ can be expressed as follows:

$$K_I = Y\sigma\sqrt{\pi a}$$

where:

$K_I$ is the applied (mode I, crack opening) stress intensity factor
$Y$ is a stress intensity correction
$\sigma$ is the stress, which includes the influence of both the applied primary stress and residual secondary stresses
$a$ is a measure of the crack size dependent upon the nature of the crack being considered; e.g. for a surface breaking flaw $a$ is the crack depth or half the height for an embedded flaw.

Failure occurs when $K_I$ is equal to $K_{Ic}$. In this case, $K_I$ is the driving force for fracture and $K_{Ic}$ is a measure of the material's resistance. As a consequence, $K_{Ic}$ is also sometimes referred to as $K_{mat}$.

The procedures adopted in EN 1993-1-10 utilise a modified approach of the fracture mechanics procedures set out in BS 7910[30],[31], in which a component can resist the effect of a crack or flaw, as long as the fracture toughness of the material is greater than the stress intensity factor. This may be expressed as:

$$K_{\mathrm{appl,d}} \leq K_{\mathrm{mat,d}}$$

where:

$K_{\mathrm{appl,d}}$ is the applied stress intensity factor

$K_{\mathrm{mat,d}}$ is a measure of material toughness, in compatible units

This fundamental relationship is transformed within the Eurocode to allow the verification to be based on temperature, $T$, such that:

$$T_{\mathrm{Ed}} \leq T_{\mathrm{Rd}}$$

where the subscripts Ed and Rd indicate 'design effect' and 'design resistance', respectively.

## 3.4.5 Eurocode procedures for sub-grade selection

Within the Eurocode, selection of an appropriate steel sub-grade is made in accordance with clause 2.2 of EN 1993-1-10 and is summarised in the following sections.

### Table 2.1 of EN 1993-1-10

Table 2.1 of EN 1993-1-10 (reproduced in Table 3.9) and Table 4 of EN 1993-1-12[32] (reproduced in Table 3.10) can be used for the selection of an appropriate steel sub-grade for steels up to S700, or the maximum permissible value of the member thickness for a given steel sub-grade. The tables give the reference stress level $\sigma_{\mathrm{Ed}}$ associated with the reference temperature $T_{\mathrm{Ed}}$, providing limiting thicknesses for each steel sub-grade. For a combination of stress and reference temperature, it is necessary to ensure that the limiting thickness is larger than the actual thickness of the member (which typically will be a plate thickness, or the thickness of a flange).

| Steel grade | Sub-grade | KV at T [°C] | KV $J_{min}$ | Reference temperature $T_{Ed}$ [°C] | | | | | | | | | | | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | | | | $\sigma_{Ed} = 0{,}75\, f_y(t)$ | | | | | | | $\sigma_{Ed} = 0{,}50\, f_y(t)$ | | | | | | | $\sigma_{Ed} = 0{,}25\, f_y(t)$ | | | | | | |
| | | | | 10 | 0 | -10 | -20 | -30 | -40 | -50 | 10 | 0 | -10 | -20 | -30 | -40 | -50 | 10 | 0 | -10 | -20 | -30 | -40 | -50 |
| S235 | JR | 20 | 27 | 60 | 50 | 40 | 35 | 30 | 25 | 20 | 90 | 75 | 65 | 55 | 45 | 40 | 35 | 135 | 115 | 100 | 85 | 75 | 65 | 60 |
| | J0 | 0 | 27 | 90 | 75 | 60 | 50 | 40 | 35 | 30 | 125 | 105 | 90 | 75 | 65 | 55 | 45 | 175 | 155 | 135 | 115 | 100 | 85 | 75 |
| | J2 | -20 | 27 | 125 | 105 | 90 | 75 | 60 | 50 | 40 | 170 | 145 | 125 | 105 | 90 | 75 | 65 | 200 | 200 | 175 | 155 | 135 | 115 | 100 |
| S275 | JR | 20 | 27 | 55 | 45 | 35 | 30 | 25 | 20 | 15 | 80 | 70 | 55 | 50 | 40 | 35 | 30 | 125 | 110 | 95 | 80 | 70 | 60 | 55 |
| | J0 | 0 | 27 | 75 | 65 | 55 | 45 | 35 | 30 | 25 | 115 | 95 | 80 | 70 | 55 | 50 | 40 | 165 | 145 | 125 | 110 | 95 | 80 | 70 |
| | J2 | -20 | 27 | 110 | 95 | 75 | 65 | 55 | 45 | 35 | 155 | 130 | 115 | 95 | 80 | 70 | 55 | 200 | 190 | 165 | 145 | 125 | 110 | 95 |
| | M,N | -20 | 40 | 135 | 110 | 95 | 75 | 65 | 55 | 45 | 180 | 155 | 130 | 115 | 95 | 80 | 70 | 200 | 200 | 190 | 165 | 145 | 125 | 110 |
| | ML,NL | -50 | 27 | 185 | 160 | 135 | 110 | 95 | 75 | 65 | 200 | 200 | 180 | 155 | 130 | 115 | 95 | 230 | 200 | 200 | 200 | 190 | 165 | 145 |
| S355 | JR | 20 | 27 | 40 | 35 | 25 | 20 | 15 | 15 | 10 | 65 | 55 | 45 | 40 | 30 | 25 | 25 | 110 | 95 | 80 | 70 | 60 | 55 | 45 |
| | J0 | 0 | 27 | 60 | 50 | 40 | 35 | 25 | 20 | 15 | 95 | 80 | 65 | 55 | 45 | 40 | 30 | 150 | 130 | 110 | 95 | 80 | 70 | 60 |
| | J2 | -20 | 27 | 90 | 75 | 60 | 50 | 40 | 35 | 25 | 135 | 110 | 95 | 80 | 65 | 55 | 45 | 200 | 175 | 150 | 130 | 110 | 95 | 80 |
| | K2,M,N | -20 | 40 | 110 | 90 | 75 | 60 | 50 | 40 | 35 | 155 | 135 | 110 | 95 | 80 | 65 | 55 | 200 | 200 | 175 | 150 | 130 | 110 | 95 |
| | ML,NL | -50 | 27 | 155 | 130 | 110 | 90 | 75 | 60 | 50 | 200 | 180 | 155 | 135 | 110 | 95 | 80 | 210 | 200 | 200 | 200 | 175 | 150 | 130 |
| S420 | M,N | -20 | 40 | 95 | 80 | 65 | 55 | 45 | 35 | 30 | 140 | 120 | 100 | 85 | 70 | 60 | 50 | 200 | 185 | 160 | 140 | 120 | 100 | 85 |
| | ML,NL | -50 | 27 | 135 | 115 | 95 | 80 | 65 | 55 | 45 | 190 | 165 | 140 | 120 | 100 | 85 | 70 | 200 | 200 | 200 | 185 | 160 | 140 | 120 |
| S460 | Q | -20 | 30 | 70 | 60 | 50 | 40 | 30 | 25 | 20 | 110 | 95 | 75 | 65 | 55 | 45 | 35 | 175 | 155 | 130 | 115 | 95 | 80 | 70 |
| | M,N | -20 | 40 | 90 | 70 | 60 | 50 | 40 | 30 | 25 | 130 | 110 | 95 | 75 | 65 | 55 | 45 | 200 | 175 | 155 | 130 | 115 | 95 | 80 |
| | QL | -40 | 30 | 105 | 90 | 70 | 60 | 50 | 40 | 30 | 155 | 130 | 110 | 95 | 75 | 65 | 55 | 200 | 200 | 175 | 155 | 130 | 115 | 95 |
| | ML,NL | -50 | 27 | 125 | 105 | 90 | 70 | 60 | 50 | 40 | 180 | 155 | 130 | 110 | 95 | 75 | 65 | 200 | 200 | 200 | 175 | 155 | 130 | 115 |
| | QL1 | -60 | 30 | 150 | 125 | 105 | 90 | 70 | 60 | 50 | 200 | 180 | 155 | 130 | 110 | 95 | 75 | 215 | 200 | 200 | 200 | 175 | 155 | 130 |

*Table 3.9 Maximum permissible values of element thickness in mm for steel grades S235 to S460 in accordance with EN 1993-1-10*

The reference stress level $\sigma_{Ed}$ is given as a proportion of the nominal yield strength $f_y$ ($t$), which may be taken as the minimum yield strength in the relevant product standard. Alternatively, $f_y$ ($t$) may be calculated as $f_y$ ($t$) = $f_{y,nom}$ - 0.25$t$ (MPa) where $f_{y,nom}$ is the nominal yield strength of the steel in MPa and $t$ is the thickness of the plate in mm.

Table 2.1 of EN 1993-1-10 or Table 4 of EN 1993-1-12 provide limiting thicknesses for three different stress levels: 0.75$f_y$ ($t$), 0.5$f_y$ ($t$) and 0.25$f_y$ ($t$). Linear interpolation can be used between the different stress levels. Most applications require stress levels between 0.75$f_y$ ($t$) and 0.5$f_y$ ($t$), while 0.25$f_y$ ($t$) is mainly given for interpolation purposes.

*Table 3.10 Maximum permissible values of element thickness in mm for steel grades S500 to S700 in accordance with EN 1993-1-12*

| Steel grade | Sub-grade | KV | | Reference temperature $T_{Ed}$ [°C] | | | | | | | | | | | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | | at T [°C] | $J_{min}$ | 10 | 0 | -10 | -20 | -30 | -40 | -50 | 10 | 0 | -10 | -20 | -30 | -40 | -50 | 10 | 0 | -10 | -20 | -30 | -40 | -50 |
| | | | | $\sigma_{Ed} = 0{,}75\ f_y(t)$ | | | | | | | $\sigma_{Ed} = 0{,}50\ f_y(t)$ | | | | | | | $\sigma_{Ed} = 0{,}25\ f_y(t)$ | | | | | | |
| EN 10025-6 | | | | | | | | | | | | | | | | | | | | | | | | |
| S500 | Q | -20 | 30 | 65 | 55 | 45 | 35 | 30 | 20 | 15 | 105 | 85 | 70 | 60 | 50 | 40 | 35 | 170 | 145 | 125 | 105 | 90 | 80 | 65 |
| | QL | -40 | 30 | 100 | 80 | 65 | 55 | 45 | 35 | 30 | 145 | 125 | 105 | 85 | 70 | 60 | 50 | 200 | 195 | 170 | 145 | 125 | 105 | 90 |
| | QL1 | -60 | 30 | 140 | 120 | 100 | 80 | 65 | 55 | 45 | 200 | 170 | 145 | 125 | 105 | 85 | 70 | 205 | 200 | 200 | 195 | 170 | 145 | 125 |
| S550 | Q | -20 | 30 | 60 | 50 | 40 | 30 | 25 | 20 | 15 | 95 | 80 | 65 | 55 | 45 | 35 | 30 | 160 | 140 | 120 | 100 | 85 | 75 | 60 |
| | QL | -40 | 30 | 90 | 75 | 60 | 50 | 40 | 30 | 25 | 135 | 115 | 95 | 80 | 65 | 55 | 45 | 200 | 185 | 160 | 140 | 120 | 100 | 85 |
| | QL1 | -60 | 30 | 130 | 110 | 90 | 75 | 60 | 50 | 40 | 185 | 160 | 135 | 115 | 95 | 80 | 65 | 200 | 200 | 200 | 185 | 160 | 140 | 120 |
| S620 | Q | -20 | 30 | 55 | 45 | 35 | 25 | 20 | 15 | 15 | 85 | 70 | 60 | 50 | 40 | 30 | 25 | 150 | 130 | 110 | 95 | 80 | 65 | 55 |
| | QL | -40 | 30 | 80 | 65 | 55 | 45 | 35 | 25 | 20 | 125 | 105 | 85 | 70 | 60 | 50 | 40 | 200 | 175 | 150 | 130 | 110 | 95 | 80 |
| | QL1 | -60 | 30 | 120 | 100 | 80 | 65 | 55 | 45 | 35 | 170 | 145 | 125 | 105 | 85 | 70 | 60 | 200 | 200 | 200 | 175 | 150 | 130 | 110 |
| S690 | Q | -20 | 30 | 50 | 40 | 30 | 25 | 20 | 15 | 10 | 80 | 65 | 55 | 45 | 35 | 30 | 20 | 140 | 120 | 100 | 85 | 75 | 60 | 50 |
| | QL | -40 | 30 | 75 | 60 | 50 | 40 | 30 | 25 | 20 | 115 | 95 | 80 | 65 | 55 | 45 | 35 | 190 | 165 | 140 | 120 | 100 | 85 | 75 |
| | QL1 | -60 | 30 | 110 | 90 | 75 | 60 | 50 | 40 | 30 | 160 | 135 | 115 | 95 | 80 | 65 | 55 | 200 | 200 | 190 | 165 | 140 | 120 | 100 |
| EN 10149-2 | | | | | | | | | | | | | | | | | | | | | | | | |
| S500 | MC | -20 | 40 | 80 | 65 | 55 | 45 | 35 | 30 | 20 | 125 | 105 | 85 | 70 | 60 | 50 | 40 | 195 | 170 | 145 | 125 | 105 | 90 | 80 |
| S550 | MC | -20 | 40 | 75 | 60 | 50 | 40 | 30 | 25 | 20 | 115 | 95 | 80 | 65 | 55 | 45 | 35 | 185 | 160 | 140 | 120 | 100 | 85 | 75 |
| S600 | MC | -20 | 40 | 70 | 55 | 45 | 35 | 30 | 20 | 15 | 105 | 90 | 75 | 60 | 50 | 40 | 35 | 180 | 155 | 130 | 110 | 95 | 80 | 70 |
| S650 | MC | -20 | 40 | 65 | 50 | 40 | 30 | 25 | 20 | 15 | 100 | 85 | 70 | 55 | 45 | 35 | 30 | 170 | 145 | 125 | 105 | 90 | 75 | 65 |
| S700 | MC | -20 | 40 | 60 | 45 | 35 | 30 | 25 | 20 | 15 | 95 | 80 | 65 | 50 | 45 | 35 | 30 | 165 | 140 | 120 | 100 | 85 | 70 | 60 |

### Reference temperature

The following expression is given in EN 1993-1-10 for calculating the reference temperature $T_{Ed}$ which takes into account different effects, such as the strain rate or the degree of cold forming, that increases the risk of brittle fracture.

$$T_{\mathrm{Ed}} = T_{\mathrm{md}} + \Delta T_{\mathrm{r}} + \Delta T_{\sigma} + \Delta T_{\mathrm{R}} + \Delta T_{\dot{\varepsilon}} + \Delta T_{\varepsilon_{\mathrm{cf}}}$$

where:

$T_{md}$ is the lowest air temperature.

$\Delta T_r$ is an adjustment for radiation loss.

$\Delta T_\sigma$ is an adjustment for stress and yield strength of material, crack imperfections and member shape and dimensions.

$\Delta T_R$ is a safety allowance, as given in the National Annex of the member state.

$\Delta T_{\dot{\varepsilon}}$ is an adjustment for strain rate.

$\Delta T_{\varepsilon_{cf}}$ is an adjustment for the degree of cold forming.

Added together, the two terms $T_{md}$ + $\Delta T_r$ give the minimum service temperature, which in the UK is taken as -5 °C for internal steelwork in buildings and as -15 °C for external steelwork in buildings.

$\Delta T_{\dot{\varepsilon}}$ = 0 for strain rates $\dot{\varepsilon}$ no greater than 4 × $10^{-4}$ $sec^{-1}$

$\Delta T_{\varepsilon_{cf}}$ = 0 when the degree of cold forming $\varepsilon_{cf}$ is no greater than 2 %.

When using the tabulated values within the Eurocode, $\Delta T_{\sigma}$ may be taken as 0 °C.

Hot finished hollow sections according to EN 10210 are covered by these rules with no temperature shift required due to the production method. However, special guidance on the choice of steel to avoid brittle fracture has been published for cold formed hollow sections to EN 10219 and any cold-formed cross-section in accordance with EN 1993-1-3 in a JRC report[33] (because of the high degrees of cold-forming involved in making these sections). The report gives a conservative assessment procedure for considering the effect of cold-forming based on an effective strain which depends on the wall thickness and inner radius of the cold bend. This approach will be included in the next revision of EN 1993-1-10.

### Material toughness, $K_{mat}$

The material toughness is derived using Charpy impact data. An estimate of $K_{mat}$ is determined using what is known as the Master curve correlation (Wallin-Sanz correlation).

### Eurocode scope of application

Clause 2.1(2) of EN 1993-1-10 states that the Eurocode rules are applicable to 'tension elements, welded and fatigue stressed elements in which some portion of the stress cycle is tensile'. However, EN 1993-1-1[5] requires that for components under compression, a minimum toughness property should be selected and refers to the National Annex which gives information on the selection of toughness properties for members in compression. For these type of members, the use of Table 2.1 of EN 1993-1-10 with $\sigma_{Ed}$ = 0.25 $f_y$ $(t)$ is recommended.

### UK National Annex to EN 1993-1-10

In the UK National Annex to EN 1993-1-10[28], the value of $\Delta T_R$, the safety allowance, is given by the following expression:

$$\Delta T_R = \Delta T_{RD} + \Delta T_{Rg} + \Delta T_{RT} + \Delta T_{R\sigma} + \Delta T_{Rs}$$

where:

$\Delta T_{RD}$ is a temperature adjustment for the detail type used; e.g. welded or unwelded.

$\Delta T_{Rg}$ is an adjustment for the effect of gross stress concentrations.

$\Delta T_{RT}$ is an adjustment to restrict steels being used too far below their $T_{27J}$ test temperature.

$\Delta T_{R\sigma}$ is an adjustment for the applied stress level.

$\Delta T_{Rs}$ is an adjustment for the strength grade.

The UK National Annex refers to a complementary Published Document, PD 6695-1-10[34]. This provides an extension of EN 1993-1-10 Table 2.1 to include a wider range of reference temperatures and the thickness limits have been re-organised in line with the UK National Annex to make it easier to use when designing the internal and external steelwork of buildings, and bridges.

### 3.4.6 Implications for HSS

It will be noted from the table that as strength increases the maximum permissible thickness decreases. The decreasing permissible plate thickness for increasing steel grades is not necessarily a disadvantage for HSS as they commonly require thinner plates compared with conventional strength steel. The increase in yield strength may therefore compensate in most cases for the reduced permissible thickness.

The maximum permissible plate thicknesses have been determined using the minimum Charpy impact energy required for the material and an assumed crack size near a welded detail with an allowance for residual stresses, which is subsequently loaded in fatigue.

For either non-welded elements or those where fatigue might not be a significant design issue, there may be scope to increase the permissible element thickness through a more detailed fracture mechanics-based assessment using the principles described in BS 7910[30]. In doing so, it is recommended that expert advice is sought both in undertaking the fracture mechanics, but also in consideration of the design flaw size being assumed. It is essential that any changes to the design flaw being considered should only be made on the understanding that the proposed non-destructive testing is capable of reliably detecting such a flaw and, if so, the particular element is subject to post fabrication inspection to ensure that critical defects are identified and addressed.

There is some allowance in the EN 1993-1-10 procedures for a degree of fatigue crack propagation, this should however not be at the expense of good practice with regards to fatigue design. A safe life approach to fatigue design would always be considered sensible.

For HSS of thickness less than 10 mm, testing with standard sized Charpy specimens is not possible. In such circumstances the testing may be conducted using sub-sized specimens. As reported by Wallin[35], the difficulty lies in extrapolating the result from a sub-sized specimen to correspond to a result from a standard sized specimen. Annex J of BS 7910 provides a procedure (based upon Wallin's work) for the treatment of sub-sized Charpy specimens, which can then be used with the Master correlation to estimate $K_{\mathrm{mat}}$.

As the thickness of the material decreases, the stress conditions tend towards plane stress which reduces the susceptibility to brittle fracture, and therefore, very thin materials might generally be expected to behave in a ductile manner.

# 3.5 Through-thickness properties

## 3.5.1 Introduction

The specification of through-thickness (z-direction) properties for steel plate relates to the risk of lamellar tearing, which can occur beneath the weld if the steel plate has poor through-thickness ductility. This is in turn associated with a high concentration of elongated inclusions which are oriented parallel to the surface of the rolled plate material. The general principles are illustrated in Figure 3.3 which shows the transverse weld shrinkage strains acting in the through-thickness direction of the cross-member and the fusion boundary of the weld roughly parallel to the plane of inclusions.

Through-thickness properties are a measure of the through-thickness ductility, expressed formally as a ‘Quality class’, but commonly known as a ‘Z-grade’. Through-thickness properties are assessed by testing in accordance with EN 10164[36], which specifies where material samples are to be taken, the dimensions of the samples and the testing procedure. The tensile test examines the capacity of the steel to ‘neck’ before fracture, which is a measure of material ductility in the z-axis.

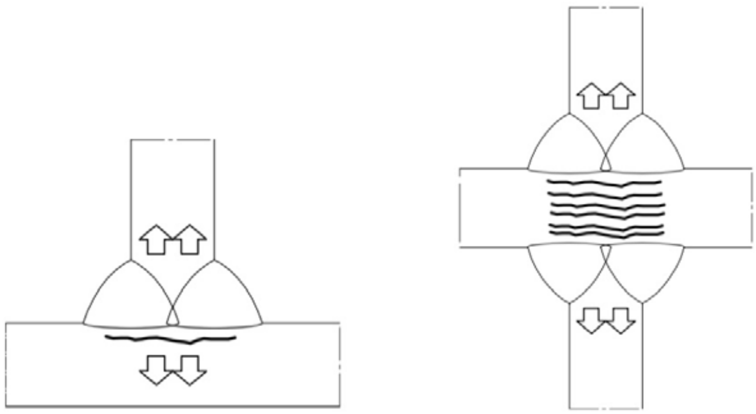


*Figure 3.3*
*Lamellar tearing*

With respect to the base material, lamellar tearing is sensitive to the anisotropy of the material and z-direction properties. In particular, the type, size, volume fraction and morphology of inclusions such as manganese-sulphide (MnS) (elongated) will increase the risk of tearing. The overall matrix properties of the steel will also contribute as higher strength steels will inherently have less ductility. Furthermore, the presence of heavy ferrite-pearlite banding of the microstructure can also initiate and support propagation of lamellar tears. The latter can be improved with steels having lower carbon contents and also Q steels.

It is important to note that all modern structural steels are fully-killed with aluminium during steelmaking, so will have low levels of inclusions present and furthermore, having sulphur contents < 0.005 % will significantly reduce the risk of lamellar tearing. If required, further steps can be taken during steel production to reduce these levels even further.

### 3.5.2 Eurocode requirements

Guidance on where improved through-thickness properties are required is given in EN 1993-1-1 and EN 1993-1-10. EN 1993-1-10 also gives a procedure for assessing the susceptibility of a welded connection to lamellar tearing. The procedure is essentially a scoring system based on a number of contributing factors. Factors that increase the risk are awarded a higher score (or Z-value) and those that reduce the risk are given a lower or a negative Z-value. The required Z Quality class must be greater than the summation of the individual Z-values.

The Eurocode notes that:

- The strain through the thickness of the material arises as welds to the surface cool and shrink. If that shrinkage is restrained by other stiff parts of the assembly, the possibility of lamellar tearing increases,
- Larger welds increase the possibility of tearing,
- Thoughtful weld detailing can reduce the risk of lamellar tearing, for example by avoiding fusion faces which are parallel to the surface of the steel,
- The sulphur content in the steel is important, as lower levels improve the through-thickness properties of the steel.

## 3.6 Durability

### 3.6.1 Introduction

The successful adoption of HSS for structural steelwork applications requires an understanding of the risks associated with corrosion and the appropriate measures that can be readily applied to control these risks. In practice the approaches taken for S355 should also be largely applicable for HSS. This section provides an overview of some of the issues relating to durability.

Section 4 of EN 1993-1-1 addresses durability, which in turn cross-references the requirements set out in EN 1090-2[6].

### 3.6.2 Good practice

The risk of corrosion can be reduced by simple, but effective design practice. For example, detailing the structure to prevent the risk of standing water, entrapment of moisture or dirt. Contact with other materials that present a corrosion risk should be avoided. For example, there may be a risk of bimetallic corrosion between steel and some other metals in corrosive environments.

Where HSS are used for applications involving fatigue, and it may be necessary to utilise post-weld improvement techniques for the weld toe, it is essential that the resulting surface is provided with appropriate corrosion protection. Failure to do so may eliminate the benefit of the weld toe improvement technique.

### 3.6.3 Corrosion protection

Effective measures to provide adequate corrosion protection should be readily achievable provided the factors affecting durability are recognised during the design process. A typical starting point would be to define the corrosivity of the environment to which the structure will be exposed. For example, atmospheric environments and general corrosion rates are classified in EN ISO 12944-2[37]. Five different corrosivity categories are defined with the associated corrosion risk, together with examples of internal and external atmospheric environments for each category.

Based upon the nature of the environment and the perceived corrosion risk, there are two primary approaches to corrosion protection for building structures that are generally adopted:

- the application of an appropriate paint corrosion protection system, or
- the application of a metallic coating which is more reactive to the environment than the steel thereby protecting the underlying steelwork.

In the context of building structures, CIRIA 174[38] provides guidance on the selection, application and specifications for paint in the general steel fabrication industry. Alternatively, EN ISO 12944-5[39] provides guidance on the selection of paint systems available for different atmospheric environments (corrosivity classification), different types of surface preparation and the anticipated durability of the system.

The most common method of protecting steelwork against corrosion using a metallic coating is by hot dip galvanizing (HDG). The specification requirements for HDG are set out in EN ISO 1461[40], further guidance can be found in EN ISO 14713-2[41]. With regards to HSS it should be noted that consideration should be given to the risk of hydrogen embrittlement associated with acid pickling pre-treatment prior to galvanizing where the steel hardness exceeds 34 HRC (Rockwell C hardness scale), 340 HV (Vickers hardness scale) or 325 HB (Brinell hardness scale). These levels of hardness approximate to a tensile strength of around 1100 MPa.

Also associated with strength is the risk of distortion cracking. EN ISO 14713-2 notes that hardened and/or high tensile steels, defined as steel with a yield strength above 650 MPa, may contain internal stresses of such magnitude that pickling and HDG may increase the risk of the steel cracking in the bath. It is recommended that specialist advice should be sought in such circumstances.

Other examples where HSS may suffer from hydrogen embrittlement include offshore applications associated with the use of immersed HSS subject to Cathodic Protection, or where corrosion at the surface of a HSS has occurred. In these particular circumstances it is recommended that specialist guidance is sought as to the appropriateness of the steel being considered for the given application.

### 3.6.4 Weathering steels

Weathering steels are low-alloy steels, which in certain environments exhibit improved corrosion resistance without the need for additional corrosion protection. They contain small amounts of alloying elements such as chromium, copper, nickel and phosphorous. Weathering steels have been found to perform best in rural temperate environments where the steel experiences regular wet-dry cycles. Like conventional strength steels, on exposure to air they form a rust layer. The difference with weathering steels is that the rust layer is more adherent and provides a protective patina which reduces the corrosion rate over time.

Until recently weathering steels were limited to grades with comparable strength to S355, but in the latest revision of EN 10025-5 the strength grades have increased to include S420 and S460. Two new toughness sub-grades are included with Charpy impact energies of 27 J at -40 and -50 °C, denoted J4 and J5 respectively, which could be of interest for low temperature applications.

These higher strength weathering steel grades, in particular plates, are produced under very similar processing conditions and meet the same requirements as their ‘non-weathering’ counterparts (i.e. according to EN 10025-3 and -4). However, they are typically made by the thermo-mechanically rolled route (designated ‘M’ or ‘ML’ delivery conditions).

Given that the improved corrosion resistance of weathering steels can be attributed to the formation of a stable rust patina, it follows that designers should incorporate an appropriate corrosion allowance in their designs. The UK National Annex to EN 1993-2[42] provides guidance for appropriate corrosion allowances for weathering steels in bridge applications. Similar corrosion allowance guidance is also given in BD 7/01[43], a section of the UK Design Manual for roads and bridges. The latter also provides clear guidance on the limitations of weathering steels, notably they are not recommended in marine environments where the structure would be affected by chlorides, nor in highly polluted environments. In both cases the prevailing atmospheric conditions interfere with the formation of the stable rust patina.

# SUSTAINABILITY

Growing awareness of the consequences of global warming and climate change is focussing attention on the environmental impacts of buildings and construction materials. In addition, growing demand for finite, non-renewable resources is forcing a rethink about how products are designed so that the value of resources is retained through circular, rather than linear, consumption models; the so-called circular economy.

This section introduces the subject of environmental assessment with a focus on aspects relating to HSS and their use within buildings and the wider construction sector.

Steel is energy intensive in production with high, associated greenhouse gas (GHG) emissions. Globally, steel production accounts for around 6 % of total GHG emissions. Conversely, steel's strength to weight ratio means that a 'little steel goes a long way' and its inherent recyclability means that steel can be recycled, or reused, several times without loss of properties. The quantification and balance of these impacts and benefits is a fundamental, and often controversial, undertaking.

## 4.1 Life cycle assessment and environmental product declarations

First developed in the 1960s, Life Cycle Assessment (LCA) is the most widely used and highly regarded tool for quantifying the environmental impacts of products and services. Despite being conceptually quite straightforward, LCA can be very complex with many important, often material-specific, assumptions that can significantly influence the outcome. LCA is the tool used to develop Environmental Product Declarations (EPDs) which are sets of standardised environmental information based on a common set of rules called Product Category Rules (PCRs). EPDs are increasingly being used by construction product manufacturers to provide robust, quantified environmental data for their products.

In order to compare EPDs of different products, the EPDs must have been developed using the same PCR, to ensure that the scope, methodology, data quality and indicators are the same. This is a frequent limitation or failing of many comparative LCA studies.

LCA involves the collection and evaluation of quantitative data on the inputs and outputs of material, energy and waste flows associated with a product over its entire life cycle so that its whole-life environmental impacts can be determined. In the context of steel-making, inputs include iron ore, coke, limestone and scrap steel, and outputs include emissions to air (including carbon dioxide) and solid waste such as slag and the steel product itself.

The scope of an LCA study generally includes a range of different environmental impacts including global warming potential, acidification, eutrophication, toxicity, etc. Of these, the most common impact considered is global warming potential (GWP) reflecting the importance of addressing the threat of global warming. GWP is also commonly referred to as the embodied carbon or carbon footprint of a product.

In simple terms, therefore, the environmental impact of a product is calculated by multiplying the weight of a product or material by the unit environmental impact of that product/material. So, for example, a low weight (strong), high impact material can have a smaller environmental impact than a heavier (less strong), lower impact material. This is very relevant in the context of HSS since weight can be reduced in many applications but the unit environmental impact (of HSS) is likely to be higher than for conventional strength steel grades (see Section 4.2). It is therefore important that a comprehensive LCA study is undertaken to assess the relative benefits of using HSS.

In the context of buildings and construction LCA, the standards developed by CEN TC 350 are important. (CEN TC350 *Sustainability of Construction Works* is the standards committee which is responsible for the development of voluntary horizontal standardized methods for the assessment of the sustainability aspects of new and existing construction works and for standards for the environmental product declaration of construction products.) These include EN 15804[44] which provides core rules for producing EPDs for construction products and EN 15978[45] which provides a calculation method for assessing the environmental performance at the whole building level.

Although it is generally understood that a well-designed HSS structure will be lighter than an equivalent structure of conventional strength steel, there have been relatively few rigorous studies estimating how much weight can be saved in practical structures subject to typical loading scenarios. There is also a paucity of reliable data about the environmental impact savings achievable through the use of HSS. A recent European research project was conducted to determine the weight and cost savings associated with a welded I-beam, box column and tubular trusses fabricated in HSS when compared to the regular S355[46].

## 4.2 Environmental impact of HSS

Structural steel grades differ in their chemical composition and/or heat treatment processes. It seems logical therefore that the environmental impact of different steel grades varies.

The steel sector has invested heavily in LCA and published many EPDs for structural steel products. These EPDs either refer to conventional strength steel grades (generally S355) or make no mention of the scope of the steel grades covered by the EPD.

An exception is the EPD published by Bauforumstahl[47] which is arguably the most commonly used EPD for structural steel in Europe. The data provided are average data based on the production routes of four large EU producers of steel plate and sections. As such, the EPD data presented includes both basic oxygen steelmaking (BOS) and electric arc furnace (EAF) steelmaking routes, i.e. an average. The EPD applies to one tonne of structural steel (sections and plates) and covers steel products of grades S235 to S960 rolled into structural sections, merchant bars and heavy plates.

ArcelorMittal has published an EPD for their HISTAR section range (see Section 2.6.2)[48]. However, since the EPD only covers steel production via the EAF route, it is not possible to compare the environmental impact of HISTAR with conventional strength steel grades and hence infer the impact of the additional Quenched and Self-Tempered process.

The most relevant research investigating the environmental impact of HSS structures is the Steel Eco-Cycle programme which was undertaken by the Swedish Steel industry between 2004 and 2012[49]. Of the range of projects conducted under the Steel Eco-Cycle programme, the project entitled ‘The environmental value of HSS structures’[50] specifically addressed the development and validation of LCA techniques to assess HSS structures. This included a number of ‘active’ structures, such as vehicles, in addition to ‘passive’ building structures. It is noted that ‘light-weighting’ of active structures, like vehicles, is likely to result in greater whole life savings than for passive structures since the operational impacts in passive structures are smaller. For active structures, over 90 % of the savings from the use of HSS are related to the use phase where HSS can enable an increase in payload or lower fuel consumption due to a reduction in weight of the structure, resulting in lower operating costs, increased competitiveness and lower $CO_2$ emissions.

LCA on the production of different steel grades was performed and relationships between environmental impact, steel type, steel strength and chemical composition established. Methods for LCA of HSS structures were developed and the potential for reducing environmental impact was exemplified via a range of case studies. Real life upgrading cases were evaluated, and reductions in weight related to increases in steel strength were established. The analysis of environmental impact from steel production is reported in detail[51].

Experience from the Steel Eco-Cycle programme shows that there is a link between the environmental impact value for different steel grades and the steel yield strength or chemical analysis. This relationship was determined through regression analysis of the environmental impact value, carbon dioxide equivalents [$CO_2$e] per tonne of steel and steel strength. The Eco-Cycle programme concluded that, for the steels studied, as the steel grade increases from S355 to S900, there is an increase in GWP impact of approximately 6 % for plates and 10 % for sections. However, depending on the production processes adopted, some higher strength steels demonstrate a smaller, or even negligible, increase in GWP impact.

The environmental benefits of using HSS in buildings have also been studied by Stroetmann[52] who presents the relative environmental impacts of higher strength heavy plates relative to S235. Weighting the different environmental impact categories considered, Stroetmann calculated the required weight savings (relative to S355J2) to compensate for the additional environmental impact of manufacturing HSS. The results are shown in Table 4.1 and indicate achievable weight savings.

*Table 4.1 Required weight saving (%) relative to grade S235J2*

| | S355J2 | S420N | S460N | S420M | S460M | S460Q | S500Q | S550Q | S690Q |
|---|---|---|---|---|---|---|---|---|---|
| Heavy plates | 6.6 | 9.3 | 10.6 | 3.3 | 4.1 | 10.2 | 11.1 | 13.4 | 17.1 |
| Rolled sections | 6.7 | 8.8 | 9.9 | 3.4 | 4.2 | N/A | N/A | N/A | N/A |

## 4.3 Friends Arena case study

The Friends Arena is a multi-purpose stadium with a retractable roof and capacity for up to 65000 people. Four 17 m deep space trusses span the 162 m width of the stadium and carry the load of the retractable roof. These trusses were identified as having most potential to benefit from the use of HSS (Figure 4.1).

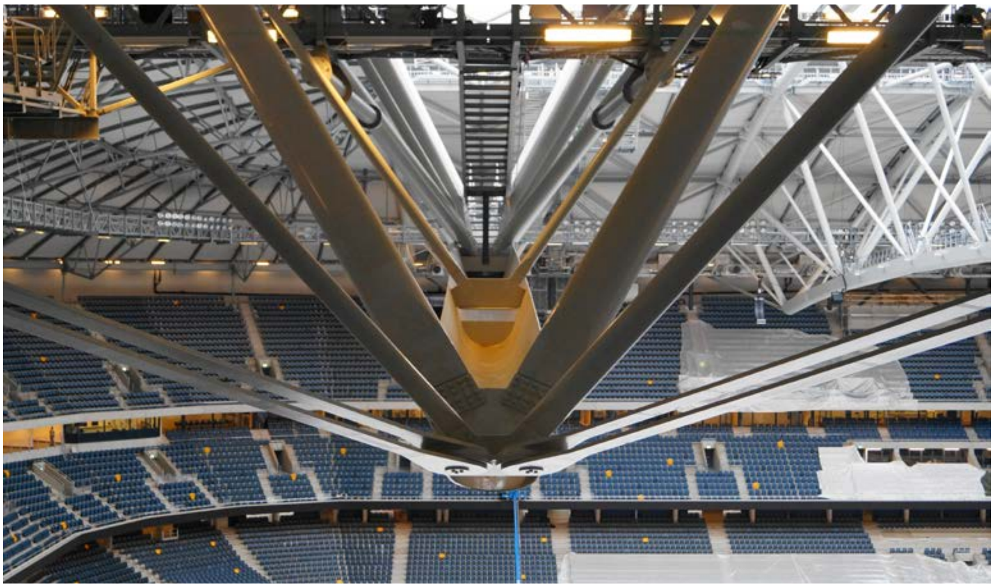


*Figure 4.1 Friends Arena roof truss, Stockholm*

The key elements of the trusses offering the greatest potential to exploit the advantages of HSS, were the top and bottom chords and the outer diagonal member closest to the support points. The top chords consist of S460 steel tubes with a diameter of just over one metre. The designers were able to use HSS in the bottom chords and the outer diagonals (S690 and S900 respectively) as these are predominantly subject to tensile loading. The bottom chord was a U profile to simplify welded connections and the outer diagonal members consisted of flat plates.

The use of HSS led to a 17 % reduction in the weight of the roof compared to a roof made from conventional S355 structural steel[53]. Although HSS is slightly more expensive than conventional strength structural steels, the reduced tonnage and cost of fabrication (mainly due to the reduced welding required) led to an overall cost saving of 14.5 %, which was over €2 million.

For the roof structure, the three aspects of the life cycle assessment (LCA) which were identified as having the greatest environmental impact were steel production, transportation and recycling at end of life. The environmental impact due to the fabrication of the structure and during its operating lifetime was assumed to be negligible. Figure 4.2 compares the environmental impact of the HSS roof with a roof fabricated from conventional strength steel. The data is presented as carbon dioxide equivalent [$CO_2e$] and shows that HSS led to reduction of the global warming potential of 17 % (895675 kg of $CO_2e$).

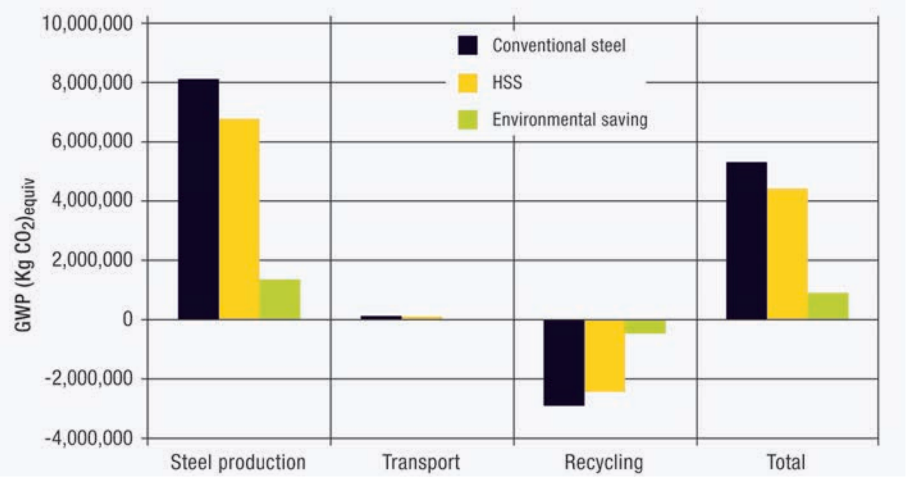



*Figure 4.2 Global Warming Potential (GWP) comparison between conventional and high strength steel[54]*

## 4.4 Building sustainability assessment methodologies

There are many national schemes that have been developed over recent years to assess the overall sustainability performance of buildings. The leading and most widely used international schemes are BREEAM (Building Research Establishment Environmental Assessment Method) and LEED (Leadership in Energy and Environmental Design). As holistic assessment methodologies, they cover a range of environmental impact criteria. Although schemes vary and reflect climatic variations and differing national priorities, they broadly cover impacts grouped into the following categories:

1. Management practices in connection with design, construction, commissioning, and handover of the building
2. Health, wellbeing and safety of building users
3. Energy efficient building solutions, systems and equipment
4. Transport impacts including access to local amenities and to sustainable means of transport

5. Sustainable water use during the operation of the building
6. Environmental impact of materials used to construct the building
7. Waste reduction from construction and throughout the life of the building
8. Sustainable land use, habitat protection and creation, and improvement of long-term biodiversity for the building's site
9. Prevention and control of pollution including light pollution, noise, flooding and emissions to air, land and water.

Impact scores within the different categories are weighted and aggregated to give an overall score or rating for the building. Assessments are generally undertaken during the design stage and subsequently confirmed via a post-construction assessment.

As shown above, the scope of these assessments covers a wide range of impacts relating to the design, construction, operation and maintenance of buildings. The environmental impact of the materials used to construct the building is only part of this scope; for example, in BREEAM, the weighting of the materials category is only 15 %. Furthermore, the materials assessment includes all elements of the building including the frame, upper floors, roof, external walls, windows, doors, partitions, services, etc. It is therefore apparent that changing the environmental impact of the structure has a relatively minor impact on the overall building assessment score.

# STRUCTURAL ANALYSIS AND MEMBER DESIGN

## 5.1 Introduction

The obvious benefit of HSS is the increased resistance, meaning that members may have smaller cross-sections, they may be lighter, or both. The consequent opportunities are:

- Structures may be lighter, which leads to smaller foundations, which are generally cheaper and may be quicker to construct.
- Shallow floor construction reduces the building height, possibly offering the opportunity to provide additional storeys.
- Shallow floor construction leads to savings in the cladding on the building elevations.
- Smaller column cross-sections are less intrusive and maximise floor space.
- Lower permanent loads and hence lower foundation bearing capacity requirements for bridges.
- Increased spans for bridges to reduce the number of piers on land or obstruction in water.

If members with smaller cross-section and higher strength are specified, Serviceability Limit State (SLS) considerations such as deflection and vibration response become more critical.

HSS should be considered whenever member resistance is the critical design check (i.e. deflection or other SLS checks do not govern). In these circumstances there is economic benefit by using the higher strength, with further benefits as described above.

If a higher strength member is specified, higher strength connection components may also be required to maintain commensurately sized details. For example, higher strength plate components should be considered so that the weaker component does not penalise the weld strengths that may be used. For bolted connections, higher strength class bolts should also be considered to transfer the force (but see 6.3.1 also).

Certain situations are ideally suited to gain benefit from HSS:

- Columns in multi-storey frames,
- Beam or column sections of low to medium slenderness,
- Fabricated members such as beams, where the flanges carry the majority of the applied bending moment,
- Components in tension.

As mentioned previously, the current scope of EN 1993-1-1[5] covers steel grades from S235 to S460, while EN 1993-1-12[32] gives supplementary rules to extend the scope up to S700. The next revision of EN 1993-1-1 will cover steel grades from S235 to S700, with a new version of EN 1993-1-12 under development for steel grades up to S960.

## 5.2 Ductility requirements

The general ductility requirements are given in Table 5.1, although they may be modified by the National Annex.

*Table 5.1 Ductility requirements for structural steels*

| | S235–S460 (EN 1993-1-1) | S500–S700 (EN 1993-1-12) |
|---|---|---|
| $f_u/f_y$ (tensile to yield strength ratio) | 1.10 | 1.05 |
| $\varepsilon_f$ (elongation at failure) | ≥ 15 % | ≥ 10 % |
| $\varepsilon_u$ (uniform elongation, i.e. elongation at the maximum load) | ≥ 15 $f_y/E$ | ≥ 15 $f_y/E$ |

In the next edition of EN 1993-1-1, the ductility requirements are expected to be modified as follows:

For plastic global analysis: $f_u/f_y \geq 1.10$ and elongation at failure not less than 15 %.

For elastic global analysis: $f_u/f_y \geq 1.05$ and elongation at failure not less than 12 %.

The requirement for uniform elongation will be removed. The next edition of the Eurocode also states that all steels conforming to any of the grades listed in Table 3.1 to Table 3.6 may be assumed to satisfy the minimum ductility requirements for elastic global analysis. Steels conforming to one of the grades up to and including S460 listed in Table 3.1 to Table 3.6 may be assumed to satisfy the minimum ductility requirements for plastic global analysis.

## 5.3 Structural analysis

EN 1993-1-12 specifies two restrictions related to the reduced ductility of HSS. Firstly, it prohibits the use of global elastic-plastic analysis, as this demands plastic hinges to form and rotate. Secondly, it also prohibits the limited plastic redistribution of moment in continuous beams permitted by clause 5.4.1(4) of EN 1993-1-1. However, this redistribution has limited practical significance, and even when designing with conventional strength steel is generally not utilised. The prohibition of using plastic analysis also does not constitute a significant inconvenience for HSS because global plastic analysis is generally limited to portal frame structures. For this type of structure, the bending moment obtained from a plastic analysis does not significantly differ from that obtained from an elastic analysis (especially when the length of the haunch is increased). Also, for portal frame structures, deflections are often critical, making the selection of HSS unlikely anyway.

Although plastic hinge analysis is not permitted for steel grades higher than S460, EN 1993-1-12 permits the use of non-linear plastic analysis considering the partial plastification of members in plastic zones. This type of analysis can thereby allow for some limited amount of stress re-distribution, although not to the same extent as with the plastic hinge analysis. Non-linear plastic analysis is not commonly used in practice due to its complexity which makes it almost exclusively suitable for Finite Element (FE) software (i.e. shell or solid FE). Therefore, when designing a structure made of HSS, the most likely type of structural analysis to be used is elastic analysis, as with structures made of conventional strength steel.

Irrespective of the type of analysis, all structures must be assessed for the significance of second order effects in accordance with clause 5.2.2 of EN 1993-1-1. This verification is an assessment of the stiffness of the structure, which tends to reduce when members with smaller cross-sections and higher strengths are adopted. Designers using HSS should therefore expect the resulting structure to be more sensitive to second order effects.

## 5.4 Design of steel members to Eurocode 3

### 5.4.1 Cross-sectional resistance

The first step in designing a member is to classify its cross-section, as subsequent calculations depend on the cross-section class. Classification is based on the width to thickness ratio of the parts of the cross-section subject to compression, which are compared to limiting values. The limiting values generally depend on $\varepsilon$, which is defined as:

$$\varepsilon = \sqrt{\frac{235}{f_y}} \quad \text{where } f_y \text{ is the design strength} \tag{5.1}$$

Thus, for higher strength steels, $\varepsilon$ reduces and the classification limits become more onerous.

Unless cross-sectional instabilities take place, the cross-sectional resistance of a steel member is directly related to its yield strength.

For steels up to S460, the yield strength and ultimate strength are given in Table 3.1 of EN 1993-1-1. The use of this table is subject to national choice, with the National Annex for several European countries specifying that the steel strengths must be taken from the product standard. Compared to Table 3.1 of EN 1993-1-1, the product standard introduces additional steps in the material thickness where the strengths must be reduced. Certain National Annexes specify that for the ultimate strength, the lowest value in the range tabulated in the product Standard must be adopted.

EN 1993-1-12 provides values for yield strength and ultimate strength of steel grades above S460 and up to S700. These strengths match those given in the product standard and are not subject to national determination.

For rolled members, the strength of the entire cross-section is often based on the maximum thickness of its constitutive elements – generally the flange.

### Local buckling

The susceptibility of a steel cross-section to local buckling is closely related to the grade of the steel. This is because the local slenderness of a cross-section is proportional to $\sqrt{f_y}$, as shown by the following equation, which is given in EN 1993-1-5[55] to calculate the plate slenderness of any flat element of a cross-section subject to compressive stress.

$$\bar{\lambda}_p = \sqrt{\frac{f_y}{\sigma_{cr}}} = \frac{\bar{b}/t}{28{,}4\varepsilon\sqrt{k_\sigma}} \tag{5.2}$$

where $\sigma_{cr}$ is the local buckling stress and $k_\sigma$ is the plate buckling coefficient

In the Eurocode, local buckling of a cross-section is accounted for by replacing the gross area (for members in compression) or section modulus (for members in flexure) with a reduced effective area or reduced effective section modulus. According to clause 4.4 of EN 1993-1-5, for cross-sections formed of flat elements, the effective area (or effective section modulus) is calculated based on the effective width of each element, which in turns depends on the plate slenderness. It should be noted that the current version of EN 1993-1-1 does not give provisions for determining the resistance to local buckling (i.e. effective area or effective section modulus) of circular hollow sections (CHS), However, these provisions will be included in the next version of EN 1993-1-1.

As a result of local buckling, the gain in cross-section resistance that may be achieved with HSS due to the increased yield strength is reduced. This reduction is illustrated in Figure 5.1 and Figure 5.2 for internal and outstand flat elements, respectively, as a function of the width-to-thickness ratio of the element, $\bar{b}/t$, and in Figure 5.3 for circular hollow sections as a function of diameter-to-thickness ratio, $D/t$ (the local buckling resistances used to develop Figure 5.3 are based on the provisions that will be included in the next version of EN 1993-1-1). In the figures, the resistance corresponding to steel grades S700 and S460 is compared to that corresponding to steel grade S355 for the case of pure compression. The range of $\bar{b}/t$ currently covered by standard Universal Columns (UC) and European wide flange columns (HD) is indicated in the figures by dark shading, while the range of $\bar{b}/t$ currently covered by square hollow sections (SHS) is indicated in Figure 5.1 by the light and dark shading. For CHS, the range of $D/t$ is indicated in Figure 5.3 by dark shading.

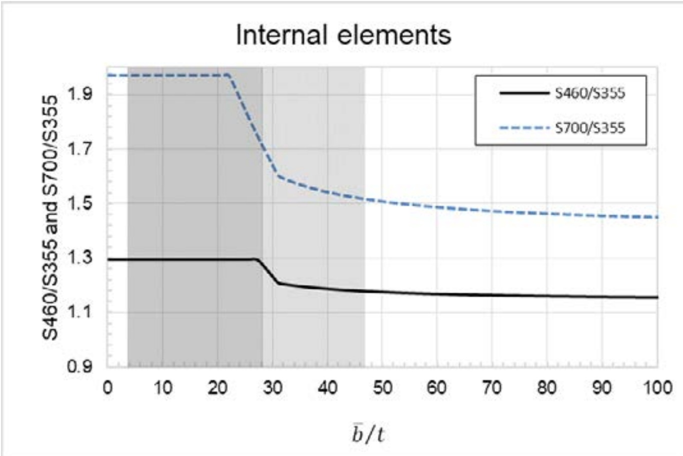



*Figure 5.1*
*S700, S460:S355 comparative local buckling resistance: internal element*

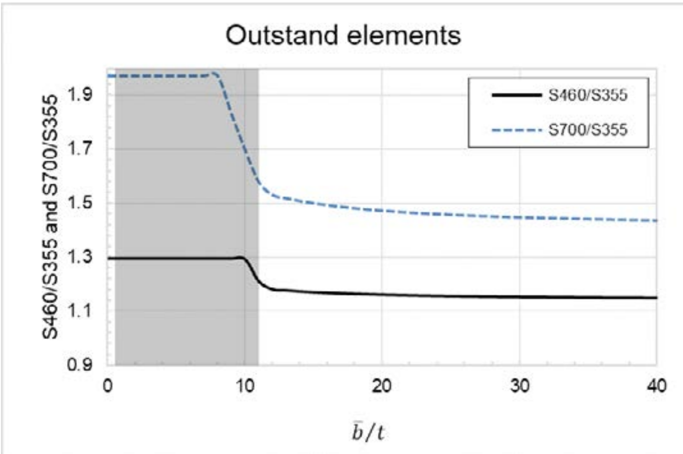



*Figure 5.2*
*S700, S460:S355 comparative local buckling resistance: outstand element*

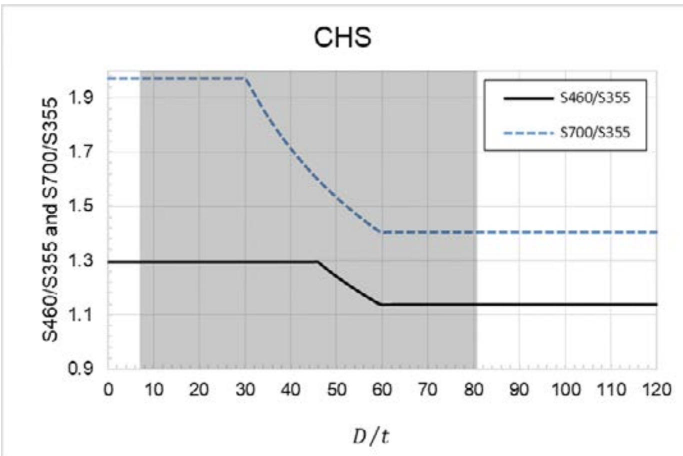



*Figure 5.3*
*S700, S460:S355 comparative local buckling resistance: CHS*

### Design resistance of a net section

Specific provision is made for the tensile resistance of a cross-section which is reduced due to holes. In addition to verifying the gross section, the net section must be verified.

In EN 1993-1-1, the resistance of the net section, $N_{u,Rd}$ is given by:

$$N_{u,Rd} = \frac{0.9 A_{net} f_u}{\gamma_{M2}} \tag{5.3}$$

The value for $\gamma_{M2}$ is recommended as 1.25, but subject to national determination.

In EN 1993-1-12, for steel grades above S460 and up to S700, the resistance of the net section, $N_{t,Rd}$ is given by:

$$N_{t,Rd} = \frac{0.9 A_{net} f_u}{\gamma_{M12}} \tag{5.4}$$

The value for $\gamma_{M12}$ is recommended as 1.25, but subject to national determination.

The partial safety factors used in the two Eurocode Parts may therefore be different. For example, in the UK, $\gamma_{M2}$ = 1.1 while $\gamma_{M12}$ = 1.25.

### 5.4.2 Member resistance (global buckling)

For columns, the non-dimensional slenderness for flexural buckling $\bar{\lambda}$ is given by

$$\bar{\lambda} = \sqrt{\frac{f_y A}{N_{cr}}} \tag{5.5}$$

while for beams the non-dimensional slenderness for lateral torsional buckling $\bar{\lambda}_{LT}$ is given by

$$\bar{\lambda}_{LT} = \sqrt{\frac{f_y W_y}{M_{cr}}} \tag{5.6}$$

In both cases, an increased design strength $f_y$ leads to an increase in non-dimensional slenderness by a factor of $\sqrt{f_y}$. The effect of the yield strength on the reduction factors $\bar{\lambda}$ or $\bar{\lambda}_{LT}$ is difficult to generalise due to the form of the buckling curve. However, generally speaking it can be said that at higher slenderness the effect is very small, while at lower slenderness the effect becomes more noticeable.

For members with high slenderness there is no benefit in using HSS, as in this case the resistance of the member is dominated by elastic buckling with little to no influence of the yield strength of the material. Also, when elastic buckling takes place the maximum stress in the member is significantly below the yield strength. Therefore, the strength of the material is not utilised efficiently. For members with low to intermediate slenderness the benefit of using HSS becomes more noticeable.

#### Flexural buckling

The use of HSS for members subject to compression is particularly advantageous for certain types of cross-section (hot-rolled I-sections), primarily because the Eurocode permits the use of a higher buckling curve for HSS. This advantage is because residual stresses in higher strength steel have been shown to have less significance on the buckling resistance[56]. For example, for a typical rolled I-section column with $h/b \leq 1.2$ and $t_f \leq 100$ mm buckling about the minor axis, the selection of HSS warrants an improvement of two buckling curves (from curve c to curve a). It should be noted, however, that in the next version of EN 1993-1-1, the buckling curve for HSS is expected to be reduced from a to b. For welded or cold-formed cross-sections, the advantage of using HSS only comes from the higher yield strength, and reduces as the slenderness of the member increases.

The buckling curves specified in EN 1993-1-1 and EN 1993-1-12 for different type of cross-sections are presented in Table 5.2.

| Cross-section | Limits | | Buckling axis | Buckling curve: S275 S355 S420 | Buckling curve: S460 to S700 inclusive |
|---|---|---|---|---|---|
| Rolled sections (beams, columns) | $h/b > 1.2$ | $t_f \leq 40$ mm | y – y<br>z – z | a<br>b | $a_0$<br>$a_0$[1)] |
| | | $40 < t_f \leq 100$ mm | y – y<br>z – z | b<br>c | a<br>a[2)] |
| | $h/b \leq 1.2$ | $t_f \leq 100$ mm | y – y<br>z – z | b<br>c | a<br>a[2)] |
| | | $t_f > 100$ mm | y – y<br>z – z | d<br>d | c<br>c |
| Welded I sections | $t_f \leq 40$ mm | | y – y<br>z – z | b<br>c | b<br>c |
| | $t_f > 40$ mm | | y – y<br>z – z | c<br>d | c<br>d |
| Hollow sections | hot finished | | any | a | $a_0$ |
| | cold formed | | any | c | c |
| Welded box sections | generally (except as below) | | any | b | b |
| | thick welds: $a > 0.5t_f$<br>$b/t_f < 30$<br>$h/t_w < 30$ | | any | c | c |

1) In the next version of EN 1993-1-1, curve $a_0$ is reduced to curve a.
2) In the next version of EN 1993-1-1, curve a is reduced to curve b.

*Table 5.2 Selection of flexural buckling curve*

The ratio between the flexural buckling resistance of a Universal Column UC made from steel grade S460 compared to one made from S355 is shown in Figure 5.4, Figure 5.5 and Figure 5.6 for three different cross-section sizes. In each case the ratio between the yield strength of steel grades S460 and S355 is indicated for comparison with a horizontal line, and the typical range of column lengths used in multi-storey frame construction (between 3 m and 5 m) is indicated by shading. These figures illustrate the advantage of using a higher buckling curve for HSS I-sections – the increased resistance is greater than the simple effect of increased strength.

The same comparison of buckling resistances is shown for rectangular hollow (RHS) and circular hollow (CHS) sections. Figure 5.7 compares the hot finished and cold formed S460 RHS columns. The higher buckling curve (curve $a_0$) used for hot finished hollow sections can be seen to result in a larger increase in the buckling resistance. The increase of resistance of high strength cold formed CHS columns is shown in Figure 5.8.

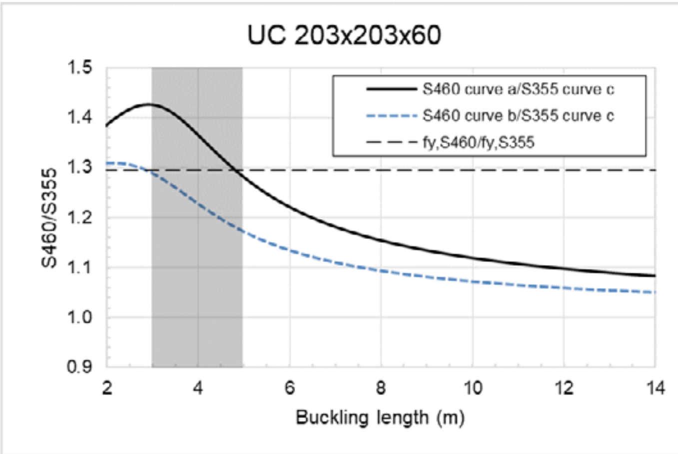



*Figure 5.4 S460:S355 comparative flexural resistance: UC 203x203x60*

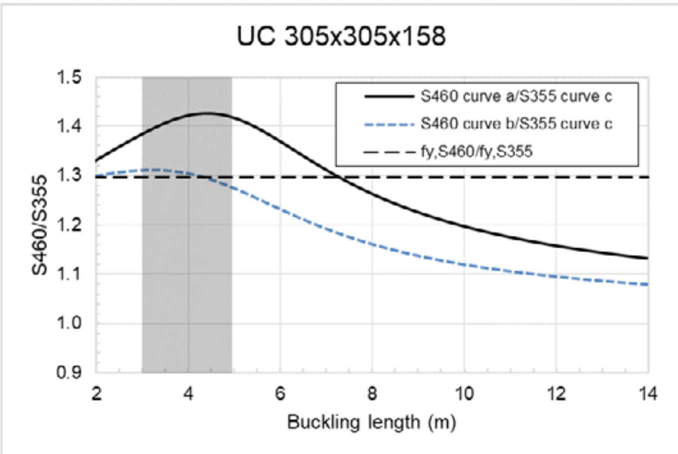



*Figure 5.5 S460:S355 comparative flexural resistance: UC 305x305x158*

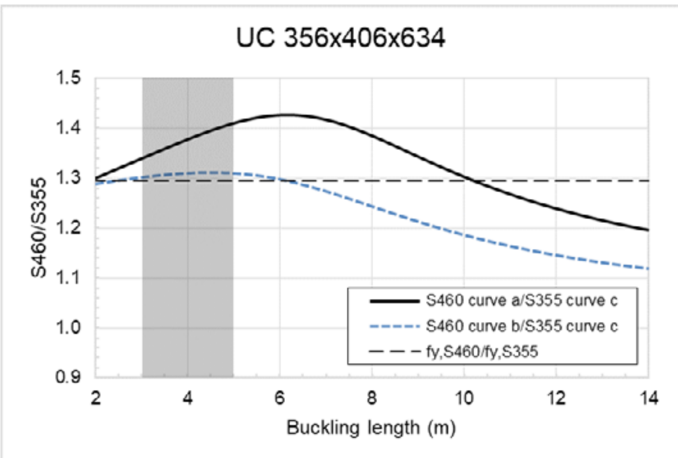



*Figure 5.6 S460:S355 comparative flexural resistance: UC 356x406x634*

Figure 5.7 S460:S355 comparative flexural resistance: Hot finished and cold formed RHS 300x200x12.5

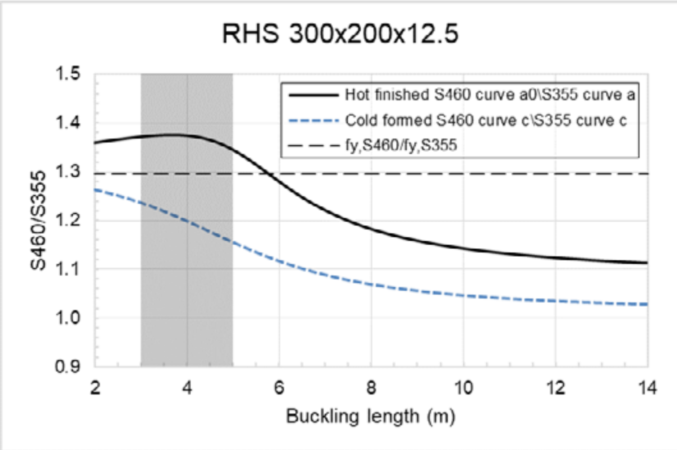
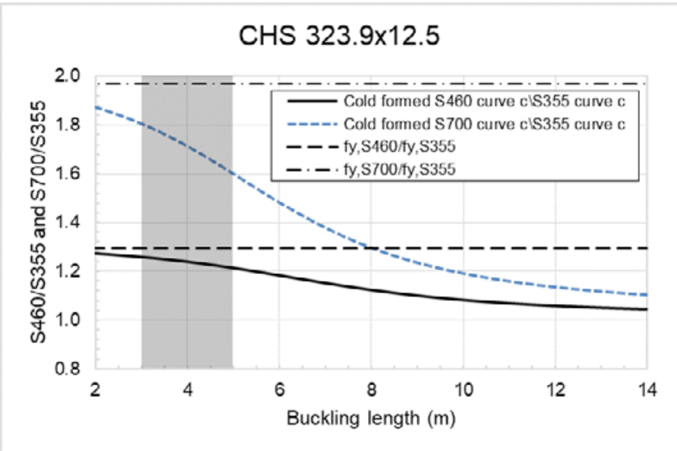



Figure 5.8 S700, S460:S355 comparative flexural resistance: Cold formed CHS 323.9x12.5




### Lateral torsional buckling

When calculating the lateral torsional buckling (LTB) resistance of a member, the selection of the buckling curve is identical for all grades. Therefore, when selecting a higher strength steel, there are only two factors affecting the LTB resistance of the beam, which are always present and oppose each other. On the one hand, the higher yield strength offered by the HSS leads to an increase in the LTB resistance. However, this benefit is rapidly counteracted by an increase in the non-dimensional slenderness (by a factor of $\sqrt{f_y}$), and consequently a reduction in the resistance. A third factor which may also limit the benefit of HSS is the potential change in the cross-section classification when the yield strength increases. This may lead, for example, to having to calculate the LTB resistance based on the elastic section modulus rather than the plastic section modulus if the cross-section moves from being Class 1 or 2 to Class 3.

Figure 5.9 compares the ratio between the LTB resistances of a UB 533x210x92 made from S460 steel to that of an identical beam but made from S355. At low slendernesses, the resistance of the beam is not affected by LTB, and the increase in resistance corresponds to the increase of the yield strength (solid curve coincides with dashed horizontal line indicating the ratio between the yield strength of steel grade S460 and S355). However, the increase in resistance resulting from the higher yield strength of HSS decreases as inelastic LTB becomes more dominant, and there is no increase in resistance at high slenderness, where failure is governed by elastic LTB.

This shows that the benefit of the higher yield strength in HSS can be better exploited for beams if LTB is prevented or minimized (for example, by having more lateral supports).

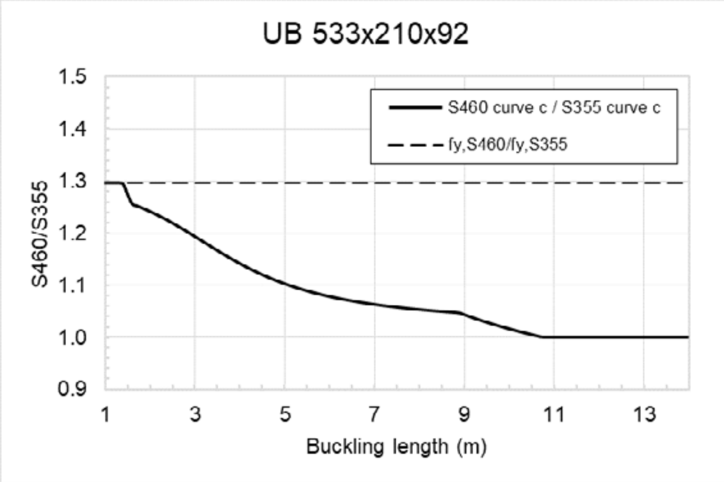



*Figure 5.9 S460:S355 comparative LTB resistance: UB 533x210x92*

### 5.4.3 Serviceability

The use of HSS is likely to result in the selection of a member with smaller cross-section, which leads to an increase in deflection. In general, the following comments may be made:

- Deflection is unlikely to be a limiting feature in unrestrained beams,
- Deflection may be a critical check for restrained beams,
- Both deflection and vibration response are likely to be critical checks in composite beam design with S355 steel, therefore care is needed in specifying higher strength (smaller) steel members. However, the dynamic response of the complete floor system is not especially sensitive to the stiffness or mass of the beams (and hence to the strength), it is more sensitive to the slab depth and the span of the primary and secondary beams.
- The dynamic response of footbridges is likely to govern design and limit the benefit of HSS.

## 5.5 Design of composite members to Eurocode 4

EN 1994-1-1[57] covers the design of composite construction using steel grades up to and including S460. The following sub-sections give guidance on how the rules could be modified for the design of composite construction in steel grades up to S700.

### 5.5.1 Concrete filled hollow sections

The scope of EN 1994-1-1 covers the design of composite columns of steel grade up to S460 and concrete grade between C20/25 to C50/60. However, it has been demonstrated that the design of concrete filled steel tubular (CFST) members in accordance with the method of EN 1994-1-1 for S460 may be extended to S550, provided the steel cross-section is not Class 4[58] and the concrete grade is selected in accordance with the recommendations given in this section. The use of even higher steel grades, although possible, is subject to additional considerations and the designer is advised to seek specialist advice. Further information on material selection is given in the following section.

#### Influence of material strength

EN 1994-1-1 does not provide specific recommendations in terms of compatible concrete and steel grades. However, it is recognised that plastic design using a rectangular stress block can lead to an overestimation of the resistance, which may be limited by the ultimate strain of the concrete. Therefore, for members in combined compression and bending, the plastic bending resistance is reduced by a factor $\alpha_{\mathrm{M}}$ which is taken as 0.9 for steel grades S235 up to and including S355 and 0.8 for grades S420 and S460. This difference in the values of $\alpha_{\mathrm{M}}$ allows for the increase in the compressive strain in the cross-section at yield as the strength increases.

Table 5.3 gives recommended ‘compatible’ concrete grades for use with different HSS grades. The matching grades of steel and concrete given in the table are based on the following simplified relationship which assumes no strength enhancement due to confinement of the concrete. The equation has been shown to be applicable to concrete strength classes up to C90/105[58]:

$$f_{\mathrm{y}} \leq 0.7E_a(f_{\mathrm{ck}} + 8)^{0.31} \qquad (5.7)$$

where

$f_{\mathrm{y}}$ is the yield strength of steel [MPa]

$E_{\mathrm{a}}$ is the elastic modulus of steel [GPa]

$f_{\mathrm{ck}}$ is the characteristic cylinder compressive strength of concrete [MPa].

*Table 5.3 Compatibility of steel and concrete materials for CFST columns (not accounting for effect of concrete confinement)*

| Concrete strength | Steel strength | | |
|---|---|---|---|
| | S420 | S460 | S500 |
| C20/25 | ✘ | ✘ | ✘ |
| C25/30 | ✓ | ✘ | ✘ |
| C30/37 | ✓ | ✘ | ✘ |
| C35/45 | ✓ | ✓ | ✘ |
| C40/50 | ✓ | ✓ | ✘ |
| C45/55 | ✓ | ✓ | ✓ |
| C50/60 | ✓ | ✓ | ✓ |

Based on the recommendations of Table 5.3, S500 grade steel should be used with a minimum of C45/55 concrete, and $\alpha_{\mathrm{M}}$ should be taken as 0.8. The use of steel up to S550 is possible when combined with at least C70/85 concrete, which is currently excluded from EN 1994-1-1. However, depending on the specific case considered (geometrical properties and relative slenderness of the CFST column), the use of steel up to S550 may be justified even for lower concrete grades than those reported in Table 5.3, if the enhancement of the concrete strength due to confinement is considered.

As an approximation, for concrete filled tubes of circular cross-section the concrete strength in the above relationship may be increased according to the following equation in clause 6.7.3.2(6) of EN 1994-1-1 which is a function of the confining stress that can be developed in the circular steel section.

$$f_{\mathrm{ck}}\left(1+\eta_{\mathrm{c}}\frac{t}{d}\frac{f_{\mathrm{y}}}{f_{\mathrm{ck}}}\right) \geq f_{\mathrm{ck}} \tag{5.8}$$

where

$t$ is the thickness of the circular hollow section

$d$ is the external diameter of the circular hollow section

$\eta_{\mathrm{c}}$ is determined from EN 1994-1-1, 6.7.3.2(6)

However, the above enhancement should only apply to cases where the relative slenderness $\bar{\lambda}$ does not exceed 0.5 and $e/d$ < 0.1 (where $e$ is the eccentricity of loading), as given in clause 6.7.3.2(6) of EN 1994-1-1. These limitations are not overly restrictive because generally circular columns with a large diameter and a low relative slenderness are those for which the use of HSS may be beneficial.

Other countries have recently developed standards that allow the design of composite columns with even higher steel grades. As an example, the new Australian / New Zealand standard (AS/NZS 2327)[59] allows the use of steel grade up to S690 and concrete of compressive strength up to 100 MPa, but the design philosophy and methods are different to the ones of EN 1994-1-1 (for example, AS/NZS 2327 adopted an elasto-plastic model instead of the rigid-plastic model adopted in Eurocode 4 for combined compression and bending).

### 5.5.2 Composite beams

Although the design of composite beams is typically governed by deflection, there are situations in which the use of HSS may still be beneficial. For example, in heavily loaded beams where the dead loads account for a large proportion of the total load, deflections may be significantly reduced by using a steel section with a pre-camber. Combining this with a steel with high yield strength may allow a larger spacing between adjacent beams, thereby reducing the weight of steel.

The strength of the concrete is often chosen during the design of the slab. Therefore, HSS composite beams can be expected to be designed using concrete of the same strength class as that used for composite beams with a steel section made of conventional strength steel.

EN 1994-1-1 permits the use of three types of analysis for calculating the bending resistance of composite beams: rigid-plastic, elastic, and non-linear. Rigid-plastic analysis can only be used for a Class 1 or 2 composite section, while elastic and non-linear analyses are permitted for all classes of composite section. Since for steel grades up to S460, the vast majority of Universal Beams (UB) and European beams (IPE, IPN and HE) are classified as either Class 1 or 2, the bending resistance of composite beams is often taken as its plastic bending resistance. It should be noted that, for steel grades up to S690, most of these sections are still classified as Class 1 or 2 in bending.

### Plastic design of composite beams

The plastic bending resistance of a composite beam is calculated based on the idealisation of rectangular stress blocks equal to the design steel strength, $f_{\mathrm{yd}}$ acting in tension or compression and to a concrete strength of $0.85f_{\mathrm{ck}}/\gamma_{\mathrm{c}}$ in the slab depth acting in compression (concrete in tension is ignored). The 0.85 coefficient is partly used to account for the difference in the shape of the real stress distribution in the concrete at a limiting strain of 0.0035, which resembles more of a rectangular-parabolic stress distribution rather than a rectangular stress block. For a conventional strength steel section, the adoption of a rectangular stress block is justified (at least for composite beams with a doubly symmetric steel section) by the ductility of the steel and its ability to development strain hardening, which are able to compensate for the unyielded elastic part near the plastic neutral axis.

As the yield strength increases, the steel section needs to be strained further in order to yield. Moreover, the increase in the yield strength also leads to lowering of the position of the neutral axis of the composite section. As a result of this, the composite cross-section may fail due to crushing of the concrete before it is able to reach its plastic resistance, or even before the most stressed fibres of the steel section on the tension side are able to reach the yield strength. For this reason, clause 6.2.1.2(2) of EN 1994-1-1 stipulates that the plastic resistance of composite cross-sections with structural steel grade S420 or S460 should be reduced by a factor $\beta$ when the distance between the plastic neutral axis and the extreme fibre of the concrete slab in compression exceeds 15 % of the overall depth of the member.

The position of the plastic neutral axis is also affected by the asymmetry of the steel section, for example if the area of the tension flange is larger than the area of the compression flange. This type of asymmetry lowers the plastic neutral axis of the composite section. For asymmetries in flange areas of over 2, the plastic neutral axis generally lies in the top flange or the top part of the web of the composite beam, whereas for asymmetries of over 3, the plastic neutral axis often lies in the lower part of the web. This means that the plastic bending resistance of composite beams with flange asymmetries exceeding 2 can be reduced due to the limiting concrete strain.

For HSS composite beams where the steel section is made of a steel grade higher than S460 and up to S690, research has shown that the bending resistance $M_{\mathrm{Rd}}$ can be safely calculated based on a larger reduction factor $\beta_{\mathrm{HSS}}$, as given in Equation (5.9)[60]. This equation is applicable to composite beams where the concrete slab has a strength class ranging between C25/30 and C50/60, the steel section has an asymmetry in the flange areas of up to 3, and the position of the plastic neutral axis measured from the top of the concrete slab relative to the overall depth of the composite cross-section $y_{\mathrm{p}}/h$ does not exceed 0.5. Equation (5.9) is also applicable to hybrid steel girders.

$$M_{\mathrm{Rd}} = \beta_{\mathrm{HSS}} M_{\mathrm{pl,Rd}} = \left(1.1 - 0.9\, y_{\mathrm{p}}/h\right) M_{\mathrm{pl,Rd}} \leq M_{\mathrm{pl,Rd}} \tag{5.9}$$

Equation (5.9) was derived for cases where full shear interaction between the steel section and the concrete slab is assumed (a theoretical case associated with no slip). For beams designed with partial shear connection, in particular, the slip at the interface between steel and concrete tends to reduce the strains in the concrete, making it less likely for the limiting strain of 0.0035 to be exceeded.

For HSS composite beams designed with partial shear connection, the bending resistance may be calculated in accordance with clause 6.2.1.3 of EN 1994-1-1. However, this should not exceed the bending resistance calculated for full interaction, taking account of the limiting strain in the concrete in accordance with Equation (5.9).

### Minimum degree of shear connection in composite beams

Clause 6.6.1.2 of EN 1994-1-1 gives rules for the minimum degree of partial shear connection in composite beams which are designed plastically at the ultimate limit state (ULS). These rules are given to ensure that the deformation capacity of the shear connectors is not exceeded. The formulae for the minimum degree of shear connection are presented in terms of the ratio ($f_{\mathrm{y}}/235$) up to the current limit of S460 grade steel. This means that the required minimum degree of shear connection is higher for HSS and is often at, or close to, 100 % shear connection. Therefore, the number of shear connectors that are required with HSS sections based on plastic design may be difficult to accommodate. This may be the case for secondary beams where the profiled sheeting is perpendicular to the beam, thereby restricting the spacing between shear connectors. For primary beams this is less of an issue as in this case the ribs of the profiled sheet run along the beam, allowing more densely positioned shear connectors.

For cases where the minimum degree of shear connection required for plastic analysis cannot be satisfied due to space limitations, elastic design may be a viable solution, particularly for un-propped beams when the ratio between the imposed to dead loads is close to 1.0. This is because when elastic analysis is used, the number of shear connectors is determined based on the longitudinal shear force resulting from the applied loads rather than that for reaching the resistance of the cross-section. For composite beams designed as un-propped during construction, the loads resulting from the self-weight of the concrete slab and steel beam are entirely resisted by the steel section (resulting in no longitudinal shear between the concrete slab and the steel section). Therefore, in this case, the shear connectors only need to resist the longitudinal shear resulting from loads applied after the construction stage.

However, it should be noted that if the number of shear connectors required is determined by an elastic analysis, the bending resistance should also be determined by an elastic analysis. This may lead to a conservative prediction of the bending resistance

## 5.6 Beams with large web openings

Steel and composite beams are often designed with large web openings in order to integrate services or to optimise the cross-section in long span structures. Castellated beams with hexagonal openings and cellular beams with circular openings are formed by cutting and re-welding UB or IPE sections so that the resulting beams are deeper and stronger than the parent section for the same section weight. Fabricated beams may also be manufactured with large rectangular or circular openings cut into the web. The depth of the openings is typically 60 to 75 % of the beam depth.

Often IPE and HE sections are combined to create highly asymmetric sections, as shown in Figure 5.10, which are efficient when used in composite construction. Fabricated beams often have large rectangular openings close to mid-span where shear forces are low, as shown in Figure 5.11, in which case the length-to-depth ratio of the openings can be as high as 3.


*Figure 5.10 Cellular beam of asymmetric shape for use in composite construction*

*Figure 5.11 Fabricated beam with rectangular and circular openings*

The potential modes of failure of composite beams with large web openings are shown in Figure 5.12, which are:

- Pure shear of the reduced web depth,
- *Vierendeel* bending resistance due to transfer of shear causing bending in the web-flange Tees, which may be horizontally stiffened,
- Web-post buckling between closely spaced openings,
- Web buckling next to widely spaced openings,
- Pure bending resistance by tension failure of the bottom Tee,
- Concrete shear or crushing.

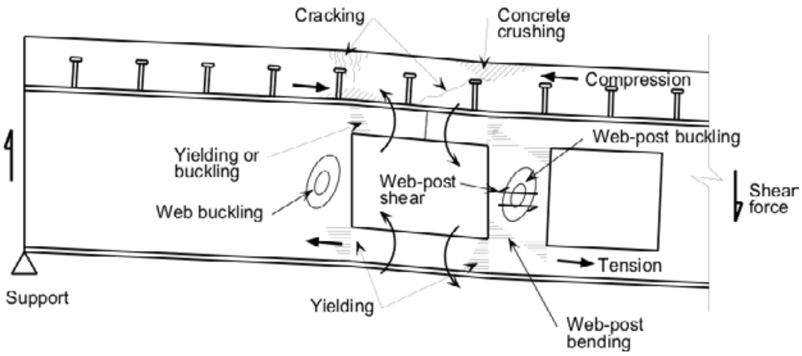



*Figure 5.12 Potential failure modes for composite beams with large web openings*

The local conditions in the web around the openings dominate the design of beams with large openings. Therefore, there is an advantage in designing the web of the beam in HSS. This has the effect of:

Increasing the local bending resistance of the web-flange Tees which increases the resistance to *Vierendeel* bending and therefore the potential size of the openings.

Increasing the shear resistance of the reduced web at the opening.

If HSS is also used for the horizontal stiffeners, the local bending resistance of the stiffened Tees increases the resistance to *Vierendeel* bending and therefore the size of the openings can be increased further.

The use of HSS for the web may also reduce the web thickness for the same opening size. However, the use of HSS has an effect on the section class of the unstiffened web around the opening, which should be taken into account. The change in steel thickness in the web has relatively little impact on the second moment of area of the steel section (as the web contributes less than 20 % to the inertia).

Work is currently underway to present codified rules for steel and composite beams with large web openings in a new part of Eurocode 3, prEN 1993-1-13, and a new annex to EN 1994-1-1 which are likely to be published in 2025. EN 1994-1-1 cross-refers to EN 1993-1-1 for its range of application and limitations. There are no restrictions on the design methods in prEN 1993-1-13 for steel up to S460 grade. For higher strength steels, the design methods are limited to the elastic resistance of the cross-section and for *Vierendeel* bending at the openings. This is because of the unknown ability of HSS to redistribute moments around a large opening, which depends on the ductility of the steel.

Guidance on the design of steel and composite beams with web openings is given in SCI publication P355[61].

## 5.7 Fabricated hybrid girders

Plate girders are commonly used for carrying high loads, and for beams with medium to long spans. A fabricated section with high strength flanges can be an ideal solution for these members, particularly if the bending resistance governs design. Since the shear resistance is often not critical, the web may be made from a lower strength plate than the flanges. When steel grades are mixed in this way, the member is described as a hybrid girder.

The design of a hybrid girder does not differ significantly from that of a homogeneous beam. The only mention of hybrid girders in the Eurocodes is given in clause 4.3 of EN 1993-1-5, which recommends that the ratio between the yield strength of the flanges and the web $f_{y,f}/f_{y,w}$ should not exceed 2. When $f_{y,f}/f_{y,w}$ is within this limit, even though some local yielding of the web may occur at the SLS, the reduction in stiffness will have a negligible effect on the deflection. The fatigue resistance of a hybrid beam with $f_{y,f}/f_{y,w} \leq 2$ is also taken as the same as that of a homogeneous beam. Clause 4.3 of EN 1993-1-5 also requires that when designing a hybrid girder, the increase of the flange stresses caused by yielding of the web should be taken into account by limiting the stresses in the web to $f_{y,w}$, and that the effective width of the web should be calculated based on the yield strength of the flange in compression. The latter requirement implies that the classification of both the web and the compression flange should be based on the yield strength of the compression flange.

The bending resistance of a hybrid cross-section can be determined following the same rationale used for a homogeneous cross-section. The only difference is that the contribution of each element has to be considered separately based on its yield strength. Figure 5.13 illustrates the typical stress distributions for a hybrid cross-section depending on whether it is classified as Class 1 or 2, Class 3 and Class 4 (where the web is Class 4), which can be used to determine the bending resistance.

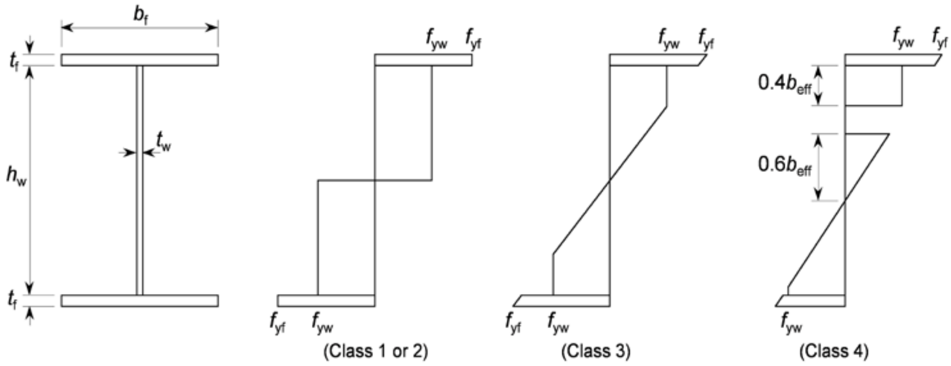



*Figure 5.13 Typical stress distribution in hybrid cross-sections classified as Class 1, 2, 3 and 4*

Regarding the resistance of a hybrid girder to lateral-torsional buckling, research has shown that yielding of the web has little influence on the lateral stability of the girder, and therefore the reduction factor $\chi_{LT}$ can be determined in accordance with clause 6.3.2.2 of EN 1993-1-1 using the same buckling curve as for welded I-sections[62]. Other checks, such as the shear resistance of the web, are identical to those required for a homogeneous beam.

Design resistance tables have been prepared by SCI for hybrid girders with steel grade up to S460[63]. The design process for a hybrid girder is illustrated in Design Example 4 in Appendix C.

## 5.8 Special considerations for design with HSS

This section recommends some methods for increasing the resistance to local or global buckling at ULS and reducing the deflections and dynamic response at SLS. Section 8 discusses methods for improving the fatigue life of welded connections.

### 5.8.1 Methods to inhibit local buckling at ULS

Susceptibility to local buckling can be reduced by controlling the width to thickness ratio ($b/t$) of elements in compression by using welded or cold formed local stiffeners. Figure 5.14 shows two other types of cross-section where folds are used to stiffen the elements susceptible to local buckling. The use of I sections with corrugated webs is another example (Figure 5.15). However, unless architectural issues dominate, it is usually cheaper to use a heavier section than to involve additional workmanship costs.

Local buckling can also be inhibited by filling a hollow section with concrete or encasing a section in concrete, or by the use of shallow floor beams or composite slabs.

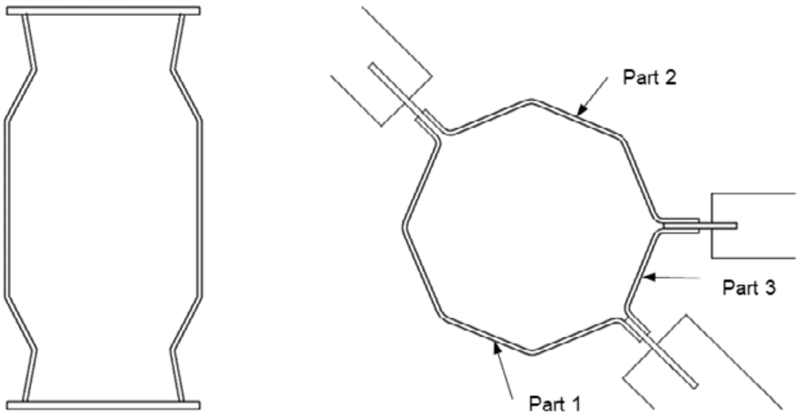



*Figure 5.14*
*Left: A beam section with stiffened webs*
*Right: A semi-closed polygonal cross-section for use as a truss chord*

*Figure 5.15*
*Sections with corrugated webs*



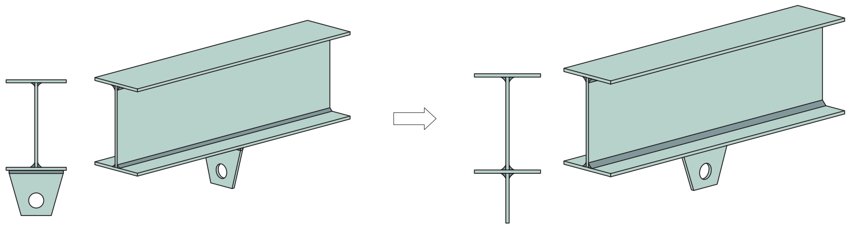
### 5.8.2 Methods to inhibit global buckling at ULS

The load-carrying capacity of high strength slender columns can be limited by global instabilities. The use of members with high h/b ratios as columns should be avoided due to their relatively low moments of inertias. The benefit of end fixity and nominal base stiffness, may enable a reduction to be made in the buckling length of the column.

With the addition of cross-arms and external pre-stressed cables, buckling displacement can be prevented and the load-carrying increased. These systems, often known as pre-stressed stayed columns, offer efficient and lightweight structural solutions[64]. However, it is important to note that pre-stressed columns are sensitive to geometric imperfection.

### 5.8.3 Load introduction

Load introduction is important in the design of some HSS structures as thinner and lighter HSS components are susceptible to unintended failure modes if the load is inappropriately introduced. For example, as shown in Figure 5.16, if an eye hook is welded to a beam in the transverse direction, the flange will be subject to high local stress which will reduce the resistance of the beam. In the second example, an angle bracket is attached to a bottom flange which results in distortion and high stresses. If the load is introduced directly to the web in both cases, the resistance of the member will not be impaired.



a) Eye hook welded crosswise results in high local stress b) Load goes directly to the web of the beam

(a) Eye hook welded to an I beam

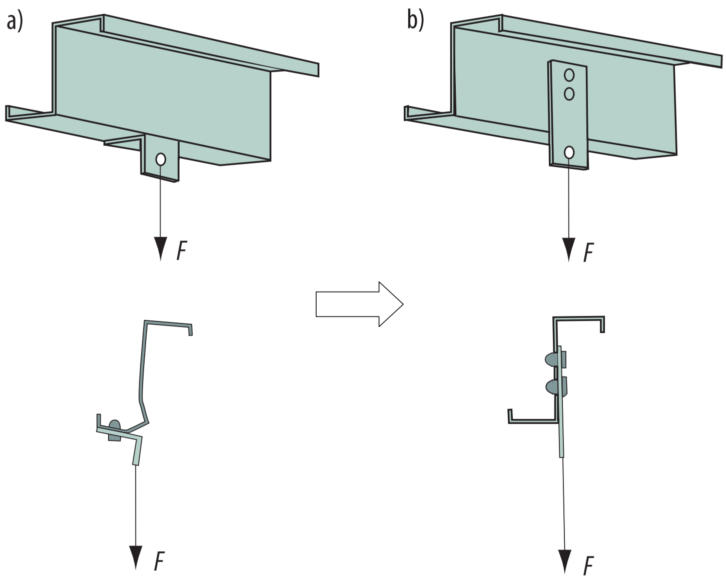



*Figure 5.16 Correct load introduction to avoid section distortion*

(b) Load introducton to a Z-section

© SSAB (Design Handbook, Structural design and manufacturing in high-strength steel)[65]

## 5.8.4 Methods to reduce deflection at SLS

HSS members are usually lighter or smaller, or both, and thus the deflections that result from reduced stiffness are more likely to govern their design. Excessive deflections can produce distortion in connections and lead to high secondary stresses.

### Increase the depth of cross-section

The stiffness of a HSS member is directly proportional to its moment of inertia. The moment of inertia can be increased by increasing the depth of the section: an increased cross-section depth and reduced wall thickness can result in both lower weight and higher stiffness. However, in many applications, minimising depth may be more important than minimising weight.

### Pre-cambering

Deflection can be partly compensated for by pre-cambering HSS beams. Generally the amount of camber applied is taken as equal to the deflection at permanent load. Some manufacturers apply pre-camber to their beams at no extra cost.

### Take advantage of end fixity

Exploiting the benefit of end fixity to reduce the distance between points of zero moment in a beam can lead to a significant reduction in deflection. Figure 5.17 shows the deflection coefficient as a function of 'support'/beam stiffness for an internal span subject to Uniformly Distributed Loading (UDL). The formula for deriving deflection from $\beta$ is shown in the figure. Zero 'support' stiffness represents the case of a simply supported beam (corresponding to a value of $\beta$ = 5 for UDL ($\delta = 5wL^4/384EI$). As the relative support stiffness increases, and the beam tends towards being built-in, the curve approaches a horizontal asymptote at $\beta$ = 1 for UDL ($\delta = wL^4/384EI$).

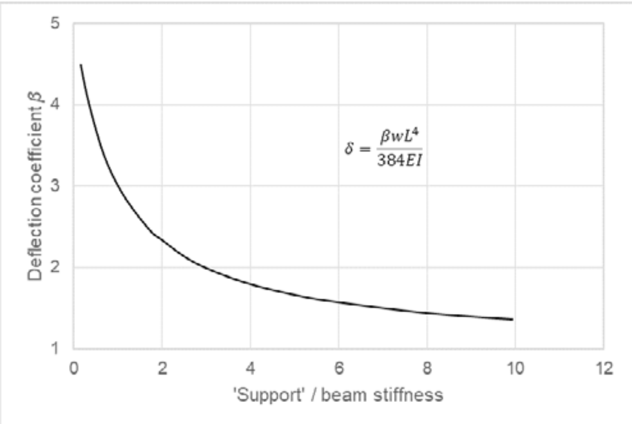



*Figure 5.17 Deflection as a function of relative stiffness – internal span with uniformly distributed load*[66]

### Composite action

In buildings and bridges, it is common to have concrete floor slabs supported by steel beams. If the steel beam is joined to the concrete slabs by shear connection, the two members then behave as one. The composite action enables the concrete slab to work with the steel beams and increases the strength and stiffness compared to when the steel and concrete acted separately, another benefit is the top flange is restrained by the concrete slab. The live load deflection can be reduced by 75 % assuming full shear interaction (infinitely stiff shear connection)[67]. Pre-camber is also often used in composite beams to compensate for the dead load of the concrete slabs. The use of HSS means that strain compatibility needs to be taken into account (see Section 5.5).

### Pre-stressed trusses

For long span trusses formed from high strength hollow sections, pre-stressing the bottom chords using post-tensioned cables can efficiently control the deflections, while bringing other benefits such as counteracting self-weight, enhancing the lateral stability and using less material. Combining the advantages of HSS and pre-stressing, pre-stressed HSS trusses can offer efficient solutions for long-span structures[64].

### 5.8.5 Methods to reduce dynamic response at SLS

A lighter floor can be susceptible to vibration under dynamic load thus causing discomfort to the occupants. Possible measures to reduce the dynamic response of floor systems are:

- increase the floor mass,
- increase the floor stiffness and, consequentially, the natural frequency,
- install extra columns to interrupt the span of flexible members,
- increase the damping of the floor system, for example by changing the placement of non-structural components such as partitions, or provision of tuned mass dampers or specialist damping materials.

However, the dynamic response of an overall floor system is far more sensitive to the slab depth and span of the beams rather than the stiffness or mass of the beams. Hence the dynamic response of a floor system with HSS beams is unlikely to differ significantly from that of an equivalent floor system with S355 beams.

Research has been carried out into the concept of Adaptive Structures[68], which are designed by separating the two engineering principles: a passive system to ensure safety and an active system to control movements, thereby maintaining comfort and usability. Any active system needs power to run, and will use some energy to control movements. In practice this means designing structures to withstand loads at the ULS, whilst being 'adaptive' at SLS under, for example, extreme wind loading, etc. The objective is to reduce embodied energy (by using less material), and on the rare occasions when extreme service loads occur, operational energy is used to provide the necessary resistance.

# DESIGN OF JOINTS

## 6.1 Introduction

The design of joints in steel up to and including S460 is covered by EN 1993-1-8[69]. Steels up to S700 are covered in EN 1993-1-12[32], which provides additional rules to supplement EN 1993-1-8 where necessary. These additional rules primarily cover welding, and are discussed in Section 6.4.1 of this guide.

Joint design in accordance with the Eurocodes is a ‘component-based’ approach, meaning that each component in the load path through a joint must be verified. The resistance of the joint is set by the weakest component.

Components in joints with HSS members will generally require to be of commensurately higher strength, otherwise the detail of the joint may become out of proportion to the connected member. It is likely that plate components will need to match the strength of the connected member, bolts will need to be of higher class (Class 10.9 typically) and welding consumables will need to be selected to deliver a higher resistance than commonly used for conventional strength steel.

The fabrication associated with joints is clearly a major contribution to the overall cost of a steel frame, so joint design and detailing should give due regard to economy. The most important contributions to delivering an economic detail are:

- Minimise welding operations. Fillet welds are preferable to butt welds, and smaller welds are the most economic. Design forces for weld verification should not be over-specified.
- Minimise the necessity for local strengthening. This is particularly true for joints between hollow sections (see Section 6.4.2), but is also true of any ‘stiffeners’ needed to reinforce a joint. Reinforcement of a joint invariably involves significant welding, and overall economy may be realised by selecting a stronger member.
- Minimise the specification of Category B and C bolted connections. These are slip-resistant connections, which involve additional measures in preparing the joint and assembly on site (see Section 6.3.3).

Standardisation of components such as end plates of standard width and thickness can be helpful, as some steelwork contractors have machinery to – for example – crop flat steel into end plates and punch standard diameter bolt holes. Other steelwork contractors cut nested plate components of any shape from a large steel sheet, so standardisation is less important. Although modern numerically controlled machinery can prepare components of infinite variety, there are advantages in familiar, standardised details.

When HSS is specified for members, in comparison to a member of conventional strength, either the member is smaller for the same design load, or higher design loads may be applied. In either scenario, bolts are typically required to carry more load than bolts connecting conventional strength steel members. Class 10.9 bolts are likely to be more appropriate, compared to the commonly used class 8.8 bolts. The potential increase in member strength when using a HSS can be significantly greater than the advantage gained by using class 10.9 bolts. If a large joint is to be avoided, larger diameter, more closely spaced bolts are likely to be required.

If the plate components used in a joint have the same high strength as the main member and appropriate high strength consumables are used (see Section 6.4.1), welded joints should not be disproportionate compared with joints fabricated in conventional strength steel.

## 6.2 Joint classification

Joints may be classified by stiffness, or strength, or both, depending on the method of global analysis. Classification by stiffness involves a consideration of the basic components of the joint. EN 1993-1-8 provides classification boundaries which may be used to identify the class of the joint.

According to EN 1993-1-8, all joints must be classified as nominally pinned, rigid or semi-rigid. This classification respects the reality that joints are never perfectly pinned or perfectly rigid. Some departure from the ideal assumption is permitted where the effects of the real joint behaviour are shown to be sufficiently small that they may be neglected.

Classification by strength requires that for a joint to be regarded as nominally pinned, this must have a moment resistance no greater than 25 % of a full strength joint and must be capable of accepting the rotations under design loads. The resistance of a full strength joint must be no less than that of the connected members.

Classification by stiffness is independent of material strength.

Classification by strength is related to the strength of the connected members. The limiting moment of 25 % of full strength will be higher (compared to conventional strength steel members), but the moment resistance of the joint is likely to be commensurately higher, if the joint uses components with a higher resistance.

Rotation is an essential characteristic of a nominally pinned joint, which has important implications for the proposed details. Joint components which are less ductile include bolts and welds. Components contributing to joint ductility are plates in flexure, bearing deformation around bolts and web panels in shear. Common practice is to ensure that welds are not the 'weak link' in a joint, and that the bearing resistance of a bolt is less than the shear resistance. This latter requirement can be difficult to achieve in practice, demanding a very thin plate, so the requirement is often met with reference to behaviour observed in tests.

## 6.3 Bolted connections

Bolted connections are categorised according to Table 3.2 of EN 1993-1-8, summarised below as Table 6.1.

*Table 6.1 Categories of bolted connection*

| Connection type | Category | Description |
|---|---|---|
| Shear | A | Bolts in clearance holes. Load transfer in shear and bearing. Some movement anticipated. |
| | B | Slip-resistant at SLS. No movement anticipated |
| | C | Slip-resistant at ULS. No movement anticipated |
| Tension | D | No preloading of bolts |
| | E | Bolts are preloaded |

### 6.3.1 Hydrogen embrittlement in HSS bolting assemblies

For hydrogen embrittlement in HSS bolts to occur, a combination of three conditions are required:

- Material that is susceptible,
- Tensile stress (typically from externally applied load or residual stresses), and
- Atomic hydrogen

As steel strength increases, it becomes harder, less ductile, exhibits a lower fracture toughness and, as a consequence, has been found to become more susceptible to hydrogen embrittlement. This is more likely to affect HSS at the upper end of the high strength range being considered by this guide. By way of example, EN ISO 14713-2[41], the European Standard for hot dip galvanizing, suggests that if steels are harder than approximately 34 HRC (Rockwell C hardness scale), 340 HV (Vickers hardness scale, or 325 HB (Brinell hardness scale) – which approximates to a tensile strength of around 1100 MPa – care must be taken to minimise hydrogen absorption during surface preparation.

Where the use of HSS is envisaged in service conditions that might promote hydrogen embrittlement, designers are advised to seek specialist advice to better understand the objective risks involved with their particular application.

Clause 3 of EN 1993-1-8 limits the highest bolt class to 10.9. Clause 5.6.1 of EN 1090-2[6] notes that the risk of hydrogen embrittlement is increased when class 10.9 bolts are electroplated or galvanised.

### 6.3.2 Category A bolted connections with HSS

Category A bolted connections are likely to be detailed with bolt class 8.8 or, more commonly, 10.9 in order to reduce the number of bolts needed and to minimise the size of the joint.

#### Bolt shear

The single shear resistance of bolt class 8.8 and 10.9 fixings is shown in Table 6.2, based on $\gamma_{M2}$ = 1.25 (the recommended value in EN 1993-1-8).

*Table 6.2 Bolt shear resistance*

| Bolt diameter | Single shear resistance [kN] | |
|---|---|---|
| | Bolt Class 8.8 | Bolt Class 10.9 |
| M20 | 94.1 | 98.0 |
| M24 | 136 | 141 |
| M30 | 215 | 224 |

#### Bearing resistance

Bearing resistance depends on bolt diameter, the ultimate strength of the steel and dimensions $e_1$, $e_2$, $p_1$ and $p_2$ as shown in Figure 6.1. These dimensions are commonly known as end distance, edge distance, pitch and gauge, respectively.

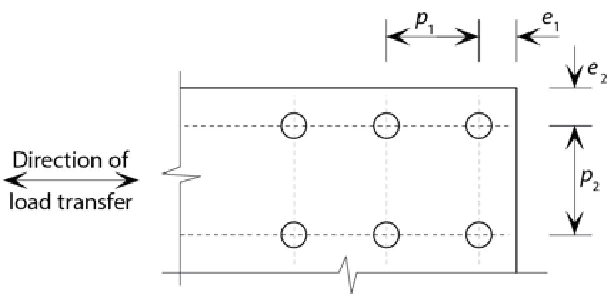



*Figure 6.1 Bolt setting out dimensions*

The calculation of bearing resistance is given in Table 3.4 of EN 1993-1-8. Table 6.3 gives bearing resistance for a typical M24 bolt in steel grade S355, S460 and S690. The assumptions made in calculating these values are shown below the table.

*Table 6.3 Bearing resistance*

| Steel grade | | Bearing resistance [kN] for plate thickness [mm] | | | | | |
|---|---|---|---|---|---|---|---|
| | Standard | 5 | 10 | 15 | 20 | 25 | 30 |
| S355 | EN 10025-2 | 72.3 | 144.6 | 216.9 | 289.2 | 361.5 | 433.8 |
| S460 | EN 10025-4 | 83.1 | 166.2 | 249.2 | 332.3 | 415.4 | 498.5 |
| S690 | EN 10025-6 | 118.5 | 237.0 | 355.3 | 473.8 | 592.3 | 710.8 |

Table based on: $e_1$ = 50 mm, $e_2$ = 40 mm, $p_1$ = 75 mm, $p_2$ = 90 mm, $d_o$ = 26 mm.
For S355, $f_u$ = 470 MPa, for S460, $f_u$ = 540 MPa, for S690, $f_u$ = 770 MPa, $\gamma_{M2}$ = 1.25

Since bearing resistance is proportional to plate thickness, it can readily be shown that for an M24 class 10.9 bolt and the tabulated bolt group geometry, the bearing resistance exceeds the shear resistance of the bolt at 9.8 mm for S355, 8.5 mm for S460 and 5.9 mm for S690. The shear resistance of the bolt will therefore often be the critical failure mode.

#### Block tearing

Block tearing is covered by clause 3.10.2 of EN 1993-1-8. In HSS, the verification is likely to be less critical than for conventional strength steel. The applied force is limited by the bolt resistance, but the higher strength of the steel increases the resistance to block tearing.

### 6.3.3 Category B and C connections

Category B and C connections are slip-resistant, at Serviceability Limit State (SLS) or Ultimate Limit State (ULS), respectively. The mating surfaces of the connection (commonly called 'faying' surfaces) are brought into close contact by pre-tensioning the bolt assemblies, and transfer the load by friction.

This form of connection is more expensive than category A, since the friction coefficient depends on preparing and protecting the faying surfaces, and because of the requirement to tension the bolts accurately.

EN 1993-1-8 provides reduction factors for preloaded bolts used in oversized or slotted holes. EN 1993-1-12 requires that for grades over S460 and bolts loaded in shear in oversized or slotted holes, only category C connections be used (i.e. with shear in normal holes, also Category A and B are permitted).

### 6.3.4 Category D and E connections

Category D and E connections are loaded in tension, and differentiate from each other by having non-preloaded or preloaded bolts, respectively. Resistance checks are identical for both non-preloaded and preloaded bolts – both the tension resistance and the punching shear resistance, as described in Table 3.4 of EN 1993-1-8, must exceed the design force. Preloading is used if the joint is subject to vibration, for fatigue resistance, for seismic loading or to increase the stiffness of the joint (although the effect of preloading is not recognised in the stiffness coefficients presented in Table 6.11 of EN 1993-1-8).

### 6.3.5 Bolted connections to hollow sections

A number of proprietary 'blind' fixings are available which allow bolted connections to be completed from one side only. These include expansion type fixings and bolts with a gravity operated anchor which rotates inside the section. Some of the proprietary fixings are covered by a European Technical Approval, and are CE marked for use in construction.

Shear resistances of the assembly are generally based on manufacturer's data. Bearing resistance in the hollow section may relate to the diameter of the fixing, or may be modified – the manufacturer will provide guidance.

Tension resistances quoted by the manufacturer are often the tension resistance of the fixing in isolation and do not account for the physical deformation of the hollow section wall. The ultimate resistance of the hollow section wall under tension is generally based on a yield line analysis[70] but this does not consider the deformation at SLS, which is likely to be critical. Finite element methods may be used to determine deformations.

## 6.4 Welded connections

Execution, coordination and inspection of welding are covered in Section 10.

### 6.4.1 Weld design

The design resistances of fillet welds and butt welds in material up to S460 is covered by clauses 4.5 and 4.7 of EN 1993-1-8, respectively. Additional guidance covering steel up to S700 is given in clause 2.8 of EN 1993-1-12.

#### Under- and over-matching weld metal

This terminology relates to the strength of the deposited weld metal. Over-matching means that the deposited weld metal is stronger than the base metal strength; under-matched means the opposite.

Under-matched weld metal can be advantageous as it is typically more resistant to hydrogen cracking (Section 10.2). The lower strength material is generally more ductile and leads to lower residual stresses. The use of under-matched weld metal is clearly not appropriate for full penetration butt joints, but may be used for partial penetration and fillet welds. The depth of penetration, or size of fillet weld may need to be increased to compensate for the reduced strength of the weld metal.

Clause 4.2(2) of EN 1993-1-8 requires the mechanical properties of the weld metal to be equivalent to, or better than those specified for the base material (i.e. over-matched). However, clause 2.8 of EN 1993-1-12 stipulates that for steels over S460 and up to S700, the weld metal may have a lower strength than the base material (i.e. under-matched). This may be restricted by the National Annex.

#### Fillet welds

The design resistance of a fillet weld depends on the throat $a$, the design shear strength of the weld $f_{\mathrm{vw,d}}$ and a correlation factor $\beta_{\mathrm{w}}$.

For welds in material up to S460 (the scope of EN 1993-1-8) the design shear strength of the weld is based on the ultimate strength $f_u$ of the parent material (the weaker part, if they are not identical), noting the requirement discussed in Section 6.4.1 that the weld metal must be at least as strong as the parent material. The correlation factor $\beta_w$ depends on the steel grade, ranging from 0.8 for S235 to 1.0 for S460.

For grades over S460, if under-matched electrodes are used, the design strength should be based on the ultimate strength of the filler metal. The correlation factor $\beta_w$ is kept as 1.0 for steel grades up to S700 (note that for S355, $\beta_w$ = 0.9).

### Butt welds

Full strength butt welds will by necessity be full penetration, and must be executed with matching or over-matched weld metal. Like fillet welds, if under-matched electrodes are used in partial penetration (and therefore partial strength) welds, the design strength should be based on the ultimate strength of the filler metal. The resistance of partial penetration welds should be based on fillet weld strengths. If partial penetration welds are reinforced with superimposed fillet welds, the weld resistance may be calculated as for a butt weld if the combined nominal throat thickness is at least the thickness of the part joined, and there is minimal unwelded gap. In other circumstances, the resistance of partial penetration butt welds reinforced with fillet welds must be calculated based on fillet weld strengths.

### Selection of weld type

The joint designer is obliged to specify the type of weld required. In the case of a fillet weld, the joint designer will specify the necessary throat (or, in many countries, the necessary leg dimension). In S460 and lower grades, since the weld metal must at least match the parent material, there is no uncertainty about the design strength of the weld.

For fillet welds in steels above S460, since it is possible to use under-matched consumable material, it is recommended that the joint designer specify fillet welds by the required throat (or leg) dimension and the ultimate strength of the consumable assumed in the calculation of weld size. This practice provides the opportunity for the steelwork contractor to propose an alternative combination of weld size and consumable strength that equally satisfies the design resistance.

'Full penetration' butt welds should only be specified by the joint designer in special circumstances, as there are a number of ways to produce a 'full strength' weld. Fillet welds, or partial penetration butt welds with superimposed fillet welds can be configured to produce a full strength connection. The joint designer should base the weld design on the actual forces at the joint and specify the assumed weld size (throat, any superimposed fillet, etc) and the ultimate strength of the weld metal assumed in reaching the specified weld size.

### 6.4.2 Welded hollow section joints

The resistance of hollow section joints is covered by section 7 of EN 1993-1-8, which, according to clause 7.1.1(4), requires a factor of 0.9 to be applied to the static design resistance of joints in steel grade S460. EN 1993-1-12 requires a reduction factor of 0.8 to be applied for steel grades between S460 and S700.

The verification of the joint requires each potential failure mode to be examined and the resistance determined. For each joint type and potential failure mode, resistance expressions are provided. The resistance expressions are based on physical tests, so it is important to observe the range of validity for the design rules.

Once the members have been selected and the geometry of the joint established, the resistance of the joint can be established. Reinforcement of a joint is possible, but expensive – and can be avoided by judicious member selection at the design stage.

In general, joint resistance is increased by selecting smaller chords with thicker walls, larger internal members, increasing the steel grade and changing the joint configuration. Overlap joints generally have a higher resistance than gap joints.

Changing the joint configuration may introduce eccentricity into the analysis model and into the joint design. Clause 5.1.5 of EN 1993-1-8 specifies when the effects of eccentricities need to be included and when they may be ignored.

The design resistance of welds between hollow sections is covered by clause 7.3 of EN 1993-1-8. Generally, a weld giving the same resistance as the member is necessary (7.3.1(4)) unless a smaller weld can be shown to be adequate, recognising the non-uniform stress distribution and paying due regard to the required deformation and rotation within the joint. Annex E of EN 1090-2 provides comprehensive examples of joint details between hollow sections, including joint preparation and weld geometry.

Welding around (and close to) the corners of cold-formed square and rectangular hollow sections is subject to some restrictions, as given in Table 4.2 of EN 1993-1-8. The cold forming strains the material at the corners, which increases the possibility of brittle fracture close to the weld. Restrictions are placed on the physical geometry of the corner radii and on the chemical composition and production quality of the cold formed steel if welding is to be carried out.

## 6.5 Mechanical fasteners

Self-drilling screws are the primary method of fastening for cold-formed steel members. Heavy duty self-drilling screws are widely available and have been used for connecting thin gauge (less than 1-2 mm thick) HSS up to S550. Above this strength grade or for thicker members, it is recommended to use a different joining mechanism such as bolting, welding or clinching.

# DESIGN AT ELEVATED TEMPERATURES

## 7.1 Structural fire resistance

Both the chemical composition and production process have an impact on the rate of degradation of mechanical properties of HSS as the temperature rises during a fire, and on the residual properties after a fire.

However, tests have shown that the strength and stiffness retention factors for conventional strength steel given in EN 1993-1-2[71] can be safely applied to HSS up to and including S700[72]. (There is considerable scatter in the elevated temperature mechanical properties for both conventional strength steel and HSS, especially for Young's modulus.) Hence EN 1993-1-12[32] states that the rules in EN 1993-1-2 are applicable for HSS up to S700 without any modification.

It should be noted that HSS members are likely to have thinner plate thicknesses, and so the rise in temperature may be quicker than for conventional strength steel of equivalent load-bearing capacity.

## 7.2 Post-fire strength properties

In general, heating up to temperatures of around 500 °C does not appear to affect the residual mechanical properties of HSS. However, cooling down from higher temperatures will have an effect on strength and ductility, which depends on the heat treatment process adopted during production. Therefore, the post-fire properties of HSS heated beyond 500 °C will differ from the expected residual properties for conventional strength steel[73],[74].

## 7.3 Fire resistant steels

The fire resistance of steels can be improved by the carefully controlled addition of key alloying elements and a low carbon content. Fire resistant steels are generally defined as steels that retain at least 67 % of their room temperature design strength at 600 °C, compared to conventional steels which retain around 47 % of their room temperature strength at 600 °C. The largest market and usage of these steels is in Japan, typically for multi-storey car parks, exterior steel frames and sports facilities. Fire resistant steels are made in design strengths up to 460 MPa in the as-rolled plate condition or as-rolled sections, although availability in Europe is limited[75] .

# FATIGUE RESISTANCE

## 8.1 Introduction

Fatigue is a damage mechanism that results from the cumulative action of repeated or cyclic stresses. In general terms fatigue damage is divided between two distinct stages of development; crack initiation and crack propagation. Subject to the continued application of the stress-history, crack propagation will continue to a point where the crack reaches a critical size and either general yielding occurs (where the applied stresses exceed the material's tensile properties because of the reduced cross-section), or brittle fracture occurs (at a point where the applied stress intensity exceeds the material's fracture toughness for the given service conditions). This section discusses fatigue and fatigue design in the context of HSS and the associated implications for the use of HSS in fatigue loaded structures.

It is important to recognise that a distinct difference in fatigue performance is seen between welded and unwelded material[76]. The fatigue performance of welded components can be much lower than that of unwelded components. For unwelded components the fatigue life is largely governed by crack initiation, and the period of crack initiation generally increases with tensile strength. However, this is not the case for welded material where the presence of pre-existing weld features and high tensile residual stresses influences the fatigue behaviour such that the majority of the fatigue life of the component is associated purely with crack propagation, and fatigue crack propagation is largely independent of material strength. Hence for most structures which are joined together by welded connections, HSS offer no additional benefit compared to that which may be obtained from S355 steel. The reality is slightly more complicated as it is explained below. It should be noted that a higher strength can be beneficial for structures where the permanent loading is much more significant than the variable loading, or where the number of high variable load cycles is small.

## 8.2 Design rules

The majority of fatigue design rules used around the world rely on empirical fatigue test data which set out the relationship between the applied stress range $S$ and the endurance or number of cycles to failure $N$ in terms of $S$-$N$ curves for different construction details. In this respect EN 1993-1-9[77], the Eurocode for the fatigue design of steel structures, is consistent with the general approach. Figure 8.1 shows the fatigue strength curves contained in EN 1993-1-9. The family of *S-N* curves are derived from constant amplitude fatigue tests conducted using different construction fabrication details, such as transverse double-sided full penetration butt welds. The data is then subject to statistical analysis and plotted on log-log curves.

Figure 8.2 is an extract from EN 1993-1-9 illustrating the fatigue 'Detail category' (effectively the applicable *S-N* curve) for a typical constructional detail. It should be noted that the achievement of the detail category is not only dependent upon the weld type, geometry and loading direction, but also on a number of additional explicit requirements set out in the table. For example, detail category 112 requires all welds to be ground flush, etc. Deviations from these requirements may have a detrimental effect on the fatigue strength. As such, designers should be aware of the limitations and ensure that the requirements for particular detail categories are realised in practice through their project specifications and engineering drawings.

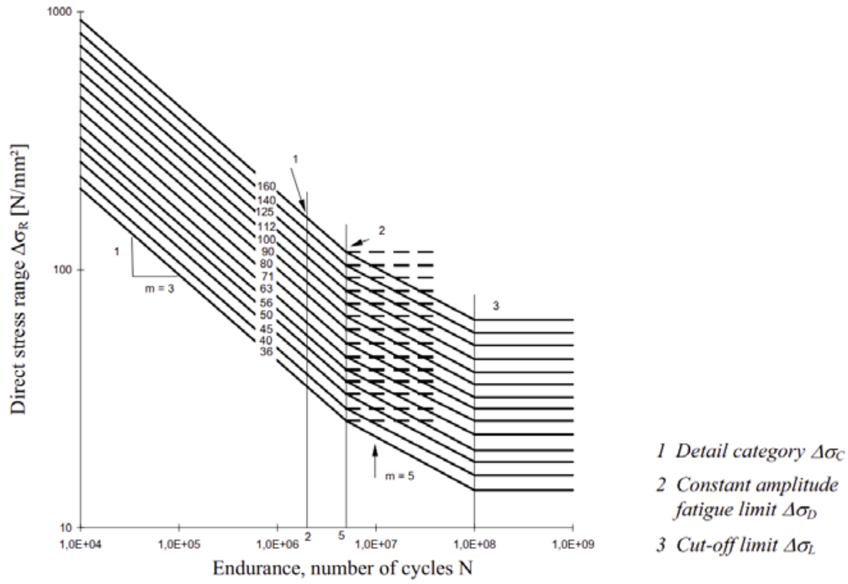



Figure 8.1
EN 1993-1-9 fatigue strength curves for direct stress range

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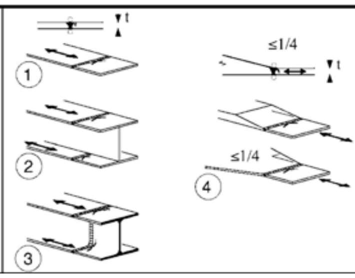
| Detail category | Constructional detail | | Description | Requirements |
|---|---|---|---|---|
| 112 | size effect for t>25mm: $k_s=(25/t)^{0,2}$ |  | Without backing bar:<br>1) Transverse splices in plates and flats.<br>2) Flange and web splices in plate girders before assembly.<br>3) Full cross-section butt welds of rolled sections without cope holes.<br>4) Transverse splices in plates or flats tapered in width or in thickness, with a slope ≤ ¼. | - All welds ground flush to plate surface parallel to direction of the arrow.<br>- Weld run-on and run-off pieces to be used and subsequently removed, plate edges to be ground flush in direction of stress.<br>- Welded from both sides; checked by NDT.<br>Detail 3):<br>Applies only to joints of rolled AC2 sections, cut and welded. AC2 |

Figure 8.2
Extract from EN 1993-1-9, Table 8.3: Transverse butt welds

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The fatigue strength curves presented in EN 1993-1-9 vary slightly from those used in other design standards and codes of practice. The constant amplitude cut-off limit, the stress range below which no fatigue damage is expected to occur in tests under constant amplitude stress conditions, is assumed to be 5 x $10^6$ cycles. This is in contrast to many other design standards which adopt $10^7$ cycles for the constant amplitude cut-off limit; for example BS 7608[78] and DNV-RP-C203[79] . Following the Eurocode approach, it is assumed that there is no fatigue damage under constant amplitude loading if the stress range does not exceed the stress range $S$ for a particular detail category for an endurance $N$ of 5 x $10^6$ cycles.

The constant amplitude nominal stress fatigue strength can be determined using the following expression (from clause 7.1 of EN 1993-1-9):

$\Delta\sigma_{\mathrm{R}}^{m}\ N_{\mathrm{R}} = \Delta\sigma_{\mathrm{C}}^{m}\ 2 \times 10^6$, where $m$ = 3 for $N \leq 5 \times 10^6$

Where:

$\Delta\sigma_{\mathrm{R}}$ is the direct stress range [MPa]

$m$ is the slope of the fatigue strength curve

$N_{\mathrm{R}}$ is the design life time expressed as a number of cycles related to a constant stress range

$\Delta\sigma_{\mathrm{C}}$ is a reference value of the fatigue strength at $N_{\mathrm{C}}$ = 2 million cycles

The process of welding introduces residual stresses, as a result, the fatigue life for welded structures and components is independent of mean stress and depends directly on the applied stress range. This applies even if the stress range is compressive. Standard fatigue design practice is therefore, based upon nominal stress range. BS 7608 does include provisions for assessing the benefit of stress relief (clause 16.3.6), but these potential benefits can only be realised if the nature and magnitude of the residual stresses are quantified. This is rarely the case, and so the benefit of residual stress relief is typically ignored.

In the real world (as opposed to the determination of empirical fatigue strength data), truly constant amplitude loading is rare. It is far more likely that the steel structure or component is subjected to variable amplitude loading. This is commonly addressed using the Miners cumulative damage sum where the component of each stress range $S$ and the associate endurance $N$ are determined and summed:

$$^{n_1}/_{N_1} + ^{n_2}/_{N_2} + ^{n_3}/_{N_3} + \cdots + ^{n_n}/_{N_n} = \sum {}^{n}/_{N}$$

Failure is deemed to occur under a sequence of different stress ranges $S$ when the sum of all the ratios $n/N$ equals unity, where $n$ is the applied number of cycles and $N$ is the fatigue endurance associated with the stress range $S$.

The use of a Miners cumulative damage sum being equal to unity is generally accepted. However, under some variable amplitude loading, including those involving fully-tensile stress cycling about a high tensile mean stress or where there is little variation in the maximum applied tensile stress, fatigue tests have shown that fatigue failure can occur at values less than unity. This issue is discussed in BS 7608, which gives guidance on where a designer might consider using a lower value. If there is any uncertainty about the nature of the applied stress-history, or for critical applications, BS 7608 recommends limiting the Miners sum to 0.5.

EN 1993-1-9 gives specific rules for determining stresses from fatigue actions for lattice girders made of hollow sections based on a simplified truss model with pinned connections. It also gives a simplified model for determining the design value of the modified nominal stress range for welded joints of hollow sections. Table 8.6 and Table 8.7 of EN 1993-1-9 give fatigue details for hollow sections and lattice girder node joints (where the slope of the fatigue curve, $m$ = 5), respectively.

## 8.3 High strength steels

The fatigue assessment procedures given in EN 1993-1-9 are applicable to all grades of structural steels, provided the materials conform to the toughness requirements contained in EN 1993-1-10[28]. Toughness requirements are given in EN 1993-1-10 for S420, S460 and S690 grades of steel in the N, M and Q supply conditions. EN 1993-1-9 therefore is applicable to HSS.

The primary advantage of HSS in static structures is to take advantage of the increased strength to reduce section size. This in turn reduces weld size, welding time and gives rise to more efficient and potentially more economic structures. Paradoxically, the efficiency gained for static structures leads to a reduction in fatigue performance for dynamically loaded structures.

By way of a simple example, if it is assumed that the increase in yield strength from S355 to S460 is associated with a comparable increase in fatigue stress range $S$ (as a result of a reduced section thickness), given that the fatigue endurance $N$ is proportional to $1/\Delta\sigma^m$ (where $m$ = 3, which is the slope of the $S$-$N$ curve), the increase in stress range $S$ results in an approximate 54 % reduction in endurance $N$. It is evident that the higher the strength, the greater the reduction in fatigue endurance for a comparable increase in fatigue stress range to static strength. In such circumstances it is clear that fatigue is more likely to govern the overall design for HSS when dynamic loading is a consideration.

It is a well-established phenomenon that the fatigue strength decreases with increasing plate thickness. As a consequence, the majority of fatigue design codes introduce a thickness correction factor. This reduction in fatigue strength is understood to be associated with the so-called $M_k$ factor, the weld toe stress intensity magnification factor, having a greater influence in thicker plates than in thin ones. EN 1993-1-9 includes a fatigue strength reduction factor for materials in excess of 25 mm (see Figure 8.2). As a consequence, the thickness reduction effect may potentially be less for HSS as they may have a reduced section size, although this benefit is expected to be modest.

## 8.4 Good fatigue design and fabrication practice

Good fatigue design practice applies equally to HSS as it does to conventional strength steels, and perhaps even more so given the potential sensitivity of HSS welded structures to fatigue. Designers should attempt to minimize fatigue loads wherever possible. More specifically, the use of construction details with a poor fatigue strength should be avoided wherever possible. This is a straightforward task where the relative performance of different construction details is readily assessed by reference to the different detail categories and corresponding $S$-$N$ curves. Given the relatively poor fatigue performance of welded joints, it will be advantageous in terms of fatigue design if welded connections can be made in areas of lower stress, thereby decreasing the sensitivity of the joint to fatigue.

In using HSS, designers might consider the use of a Miners cumulative damage sum less than unity, which would introduce a further degree of conservatism beyond that of the design code. This should only be considered on a project by project basis with a careful consideration of the consequences associated with fatigue failure.

It also follows that good fatigue design practice can be undone by poor quality fabrication. More care and attention should be given to ensure the quality of HSS fabrication. This requires careful control of the welding operations to achieve welds with good penetration and overall weld geometry to reduce local stress concentrations. Equally, the better the weld quality in terms of weld penetration, weld profile and non-destructive testing and inspection, the better the fatigue performance. This places the onus of responsibility on both the steelwork designer and fabricator. The designer should ensure that the project specification adequately covers the quality requirements to achieve the desired fatigue performance and the fabricator should ensure that the specification requirements are implemented.

## 8.5 Methods for improving fatigue resistance of welded joints

The area of most interest to the fatigue design of welded joints in HSS perhaps relates to methods of improving the fatigue resistance of the welded joints. In this respect there have been some significant improvements in the understanding of the various techniques available and the benefits that they may offer. Much of this research work has been conducted and documented in association with the International Institute of Welding (IIW).

Post-weld improvement techniques can be divided into two categories, those that seek to effectively improve the weld toe geometry and those which are focussed on changing the prevailing residual stresses. In both cases these techniques are intended to improve the fatigue strength of welded joints with respect to fatigue failure initiating from the weld toe. They offer no benefit to detail categories where the fatigue cracks initiate and propagate from other locations, such as from the weld root for fillet or partial penetration welds. The designer should always be aware of potential fatigue crack initiation sites and ensure their design adequately covers each possibility.

Geometry modification aims to reduce stress peaks, generate lower stress concentration factors (SCF) and improve the surface quality. Fatigue cracks initiate at weld toes partly because of the severity of the local stress concentration effect, but also because of the presence of very small crack-like features which are of the order of 0.1 mm deep. By removing the crack-like feature and reducing the SCF an improvement in fatigue life can be realised. Geometry modification techniques include burr grinding and TIG weld dressing.

Residual stress techniques aim to introduce a high local compressive residual stress in the region of the weld toe. Some of the techniques are also capable of improving the surface quality and reducing the SCF in a similar manner to the geometric modification techniques. Techniques included within this category include hammer peening, needle peening and high frequency impact treatments (HFMI).

High frequency mechanical impact (HFMI) covers a range of technologies, such as ultrasonic impact treatment (UIT) and pneumatic impact treatment (PIT). Through extensive testing these techniques have been demonstrated to provide a reliable, effective and user-friendly method for post-weld fatigue strength improvement technique for welded structures.

The cost benefit of weld toe improvement techniques will inevitably have to be justified through a life cycle cost approach. It can be assumed that the techniques will add to the initial fabrication cost. However, these additional initial costs are likely to be insignificant in comparison to major costs that might be incurred with addressing fatigue cracking later in the life of the structure. A good example in this respect would be a major highway bridge where the costs of any intervention during the life of the structure are likely to be considerable.

IIW-2142-10 *Recommendations on methods for improving the fatigue strength of welded joints*[80] reviews and comments on these different techniques for improving fatigue resistance. A reported improvement of up to 30 % in fatigue strength may be realised with the geometry techniques of burr grinding or TIG dressing. This corresponds to an improvement of 2.2 times the original (untreated) fatigue life. Further, for steels with a yield strength greater than S355, a reported improvement of 50 % in fatigue strength may be realised for the residual stress techniques of hammer or needle peening. This corresponds to an improvement of 3.4 times on fatigue life.

More recently, IIW *Recommendations for HFMI treatment for improving the fatigue resistance of welded joints*[81] focussed specifically on HFMI techniques. It has been proposed that the fatigue design methods for HFMI welds are based on an assumed $S$-$N$ curve slope of $m$ = 5 and fatigue strength factors defined at endurances of $N$ = 2 x

$10^6$ cycles. The range for which the benefit can be utilised excludes non-welded details and details where the fatigue lives are not governed by weld toe fatigue crack failures. The guideline is applicable to steel structures of plate thicknesses of 5 to 50 mm and for steel grades ranging from S235 to S960. For HSS grades greater than S355, the reported proposed fatigue strength benefit following HFMI treatment is an upgrade of four fatigue classes. This is based upon the IIW *S-N* curves which are different to the *S-N* curves in EN 1993-1-9. This improvement is significant and opens the opportunity to benefit from HSS in fatigue applications.

The benefit of the types of procedure described above should not be used to compensate for poor fatigue design practice or in any way compensate for the use of poor fabrication practice. The fatigue strength improvement procedures should be considered complementary. Equally, with any technique of this nature, care has to be taken to ensure that the application of the technique is reliable and repeatable. In this context, both IIW guidelines present recommendations on adequate treatment procedures and quality control measures. There are reported examples where poor application of the technique has not achieved the desired improvement in fatigue performance, but has resulted in a reduction relative to the as-welded condition.

# EXECUTION

## 9.1 Introduction

This section gives recommendations for good practice concerning the execution of HSS structures, discussing the distinctive fabrication characteristics of N, M and Q type steels. It is intended that the information provided here could enable a preliminary assessment to be made of the suitability of a fabricator to perform work on these steels.

It is essential that HSS components are clearly marked to identify the grade if differing grades and/or qualities of constituent products are in circulation together. EN 1090-2[6] does not permit the use of hard stamped, punched or drilled marks on steels above S500.

The fabrication processes applicable to HSS components do not differ significantly from fabrication of conventional strength steel. However, in general it is considered that a more rigorously controlled execution process is essential in order to avoid any potential deterioration of the mechanical properties and/or hydrogen (cold) cracking issues. Naturally, the most common source of hydrogen is from moisture, which can come from humidity in the local environment. Other potential sources arise from mill scale (oxides), greases, oils, paints and other coatings which may have been applied to the base steel. The filler metal itself (i.e. electrode / wires) can also be a source of hydrogen as these can be susceptible to moisture pick-up if stored and handled incorrectly, leading to the introduction of hydrogen into the weld. Furthermore, improper shielding materials and set-up can also lead to unwanted hydrogen pick-up.

Great attention should be paid to thermal fabrication processes with HSS, as uncontrolled exposure to elevated temperatures may alter the microstructure of the material and consequently compromise its properties (particularly for M and Q steels since their high strength is obtained by controlled cooling and heat treatment).

Tooling for working with HSS is usually more expensive than that needed for conventional strength steel. Harder cutting tools made from ‘better’ cutting steels are generally employed to ease the cutting process and prolong the life of the tool. Normal steel cutting blades and drill bits can be used, however their performance and longevity can be compromised when used on HSS. Equally, cold forming will require high capacity equipment.

The structural form of HSS components may be dictated by product availability, which is more limited than that for conventional strength steel. For example, hot rolled open sections are only available up to grade S460. I-sections of higher steel grades are fabricated by welding.

## 9.2 EN 1090 *Execution of steel structures and aluminium structures*

In Europe, the fabrication and erection of steel structures is carried out in accordance with the harmonised standard EN 1090 which specifies requirements for the execution of steel structures in order to ensure adequate levels of mechanical resistance, stability, serviceability and durability. (In the UK, the National Structural Steelwork Specification[82] is widely used for buildings, the Highways Specification[83] for road bridges and the Network Rail Specification[84] for rail bridges.)

Part 1 of EN 1090 is *Requirements for Conformity Assessment of Structural Components*[85]. This Part describes how manufacturers can demonstrate that the components they produce meet the declared performance characteristics (the structural characteristics which make them fit for their particular use and function).

Part 2 of EN 1090 is *Technical Requirements for Steel Structures*. This Part specifies the requirements for the execution of steel structures and determines the performance characteristics for components that the manufacturer must achieve and declare through the requirements of Part 1. It is applicable to structural components in buildings and other similar structures. It covers technical requirements for a wide range of carbon steel and stainless steel structures, dealing with both hot rolled steel up to and including S690, and cold formed product forms up to and including S700. It can also be used for HSS up to grade S960, though no specific guidance is given for these stronger steels. Work is underway preparing an annex to EN 1090-2 with special rules for HSS stronger than S700 and up to S960.

## 9.3 Execution class

An execution class must be specified in accordance with the normative Annex C of EN 1993-1-1. There are four execution classes in EN 1090-2: Execution Class 1 gives the lowest set of requirements and Execution Class 4 gives the highest, most stringent set of requirements. The main reason for giving four execution classes is to provide a level of reliability against failure that is matched both to the consequence of failure for the structure, component or detail, and to the requirements of execution. Each class relates to a set of requirements for fabrication and in-situ construction which are given in Annex A.3 of EN 1090-2. The execution class is used by steelwork contractors to put in place a set of manufacturing process controls that form part of a certified factory production control system for CE marking fabricated steelwork. This limits the structures that each steelwork contractor is permitted to fabricate. For example, a steelwork contractor with an EXC2 certified system can only fabricate EXC1 and EXC2 structures. Clients, specifiers and main contractors can therefore use the execution class to identify steelwork contractors with the correct level of quality and assurance controls as a preliminary assessment. Fabricators with experience in fabricating HSS would be a second level of appraisal that could also be considered. The execution class is also used by designers to determine the appropriate level of quality and assurance controls required during fabrication to meet their design assumptions.

The execution class can be specified for the works as a whole, or for an individual component or even a detail of a component. In some cases the execution class for the structure, the component and the details will be the same while in other cases the execution class for the components and the details may be different from that for the whole structure. Over-specification of the execution class should be avoided wherever possible, to prevent unnecessary costs being introduced.

The factors governing the selection of the execution class are:

- The required reliability (based on either the required consequence class or the reliability class or both, as defined in EN 1990[29]),
- The type of structure, component or detail,
- The type of loading for which the structure, component or detail is designed (static, quasi-static, fatigue or seismic).

While each building needs to be considered on its own merits, EXC2 will be appropriate for the majority of buildings constructed in non-seismic zones. EXC4 should be applied to structures with extreme consequences of a structural failure. The National Annex to EN 1993-1-1[5] gives guidance on the selection of execution classes.

It is recommended that execution of HSS components should be classified at least as EXC2.

## 9.4 Thermal cutting

HSS can be cut by the same thermal cutting methods (flame, plasma and laser) used for cutting steel of conventional strength. Other than welding, thermal cutting is the only execution process in EN 1090-2 that involves localised melting of the steel.

Due to the amount of heat generated during cutting, the microstructure and properties of the surrounding metal, known as the heat affected zone (HAZ), change so that they differ from that of the base material. The properties of the HAZ depend on the delivery condition of the steel, the chemical composition of the steel and the impact of the exposure to elevated temperatures from the cutting process. These changes in properties are usually undesirable, resulting in the properties in the HAZ becoming inferior to the base metal leading to residual stress, welds cracks (both hot cracking and cold cracking), increased brittleness, lowered material strength and a reduced corrosion resistance. As a result of this, the HAZ is often the area where failures occur.

Thermal cutting processes that operate at higher temperatures and slow speeds tend to create a larger HAZ, while lower temperature or higher speed cutting processes tend to reduce the HAZ size. The extent of the HAZ is determined by the cutting process: flame cutting has the highest thermal impact followed by plasma cutting and laser cutting. Under normal conditions, the total HAZ from flame cutting will extend only 2 - 3 mm into the plate. However, this value is thickness dependent and may be in excess of 10 mm, particularly if slow cutting speeds are employed.

Cracks in the cut surface may become visible between 48 hours up to several weeks after thermal cutting. Cut edge cracking is a phenomenon that is closely related to hydrogen (cold) cracking in welds, and depends on the hydrogen content and residual stresses in the steel. For steel grades S460 up to S700, EN 1090-2 limits the hardness of free edge surfaces to 450 (HV10). It is therefore beneficial to reduce the hydrogen content as well as the residual stresses, for example by pre or post heating the cut area, reducing the cutting speed or allowing slow cooling. The hard edges can be mechanically removed by grinding or cutting. This is especially important for bridges and other fatigue sensitive structures as hard brittle edges are susceptible to crack initiation.

It is recommended that the thermal cutting of HSS should only be undertaken using procedure specifications (TCPSs – Thermal Cutting Procedure Specifications), and that these must be supported by approval records (TCPARs – Thermal Cutting Procedure Approval Records). The TCPARs should be independently ‘qualified’ using similar criteria for macro examination and hardness checks in HAZ as are used for the welding approval WPAR (Welding Procedure Approval Record). The TCPAR could be undertaken at the same time as the WPAR if it is assumed to have no time limit.

### 9.4.1 Flame cutting (oxy-fuel cutting)

HSS plates can be easily cut by the flame cutting process. Although it is possible to cut relatively thin material, flame cutting is generally used to cut material above 20 mm thick. There is no upper limit on the thickness of steel which can be flame cut.

Typically no preheating is required when steel grades S420 and S460 are flame cut at an ambient temperature of 15 °C or above. Preheating the surrounding area to be flame cut to 50 °C is recommended if the steel is wet, or the temperature is below 0 °C. Preheating may be necessary for flame cutting thick steel plates above S460 - advice from the steel producer should be sought.

If the cut edges are expected to undergo cold forming for further processing, it is recommended to preheat a zone approximately 100 mm wide within the forming area to a temperature between 120 °C and 200 °C.

The European project HIPERCUT[86] investigated cut edges for a range of HSS from S355M to S890Q using oxy-fuel, plasma and laser cutting facilities and developed a set of guidelines and recommendations for optimising cut edge quality and fatigue and fracture resistance.

### 9.4.2 Plasma and laser cutting

HSS can be cut by plasma and laser cutting methods using the same process parameters as conventional strength steel. The maximum plate thickness which can be cut is approximately 25 mm for laser cutting and 50 mm for plasma cutting. Preheating is not required for plasma and laser cutting.

## 9.5 Hot forming

Hot forming is permitted between 960 °C and 750 °C for normalized HSS, followed by natural cooling in air. The cooling rate should be controlled to prevent hardening and excessive grain coarsening.

M or Q steels are not suitable for hot forming as the microstructure of the steels will be altered and properties impaired – it is very difficult to regain the original properties of these steels.

## 9.6 Flame straightening

Flame straightening is a local heating technique used to correct distortion or warping which has occurred during welding or manufacture. This technique can be used on HSS components: advice on maximum temperatures should be sought from the steel producer in order to avoid adversely affecting the steel's strength or toughness. It follows that steelwork fabricators should employ suitable controls to ensure that these temperature limits are not exceeded.

EN 1090-2 requires that a documented procedure be developed for flame straightening steels stronger than S355, which includes:

- maximum steel temperature and procedure of cooling allowed;
- method of heating;
- method used for temperature measurements;
- identification of workers entitled to apply the process.

The procedure shall be qualified based on the results of tensile, impact and hardness tests.

As part of the European project OPTISTRAIGHT, flame-straightening of a range of as-rolled, N, M and Q steels up to S690 in accordance with EN 10025[1] were investigated[87]. For as-rolled, N and M steels up to S460, an upper limit of 900 °C for surface (superficial) heating and up to 700 °C for full section heating were recommended, but this was time dependent. For S690 Q steels an upper limit of 800 °C for surface (superficial) heating and up to 30 °C below the tempering temperature of the product was recommended for full section heating.

## 9.7 Cold forming

Cold forming HSS sheet and plate can be carried out successfully with care and appropriate bending parameters. Guidance on minimum forming radii can be obtained from steel producers. Some guidance is also given in product standards. Designers and fabricators should be aware that when bending HSS there is a risk of losing contact between the tool and the material being formed, which results in a smaller inside radius than the tooling radius. Notches increase the risk of a crack forming and should be removed from the forming area.

Designers should be aware that cold forming causes a degree of hardening of the steel, which in turn results in a reduction of the toughness. The effects of cold forming should be considered when addressing the design against brittle fracture (Section 3.4).

In general, the force required for bending and the springback increase with the strength of the steel. The yield strength has the greatest influence on springback. To compensate for the greater springback of HSS, the die should be shaped in such a way as to allow over-bending without coining the material (applying a sufficiently high stress to induce plastic flow on the surface). Trials are recommended as the springback of HSS is very difficult to predict accurately.

Flame cut edges should be dressed by grinding or machining prior to forming to ensure that hardened edges are removed completely. Sheared edges should also be dressed by grinding.

EN 1090-2 requires that the stress relief treatment for HSS after cold forming should be carried out at temperatures between 530 °C and 580 °C (or 40 °C below the tempering temperature for Q steels). The holding time should be 2 min/mm of material thickness, with a minimum time of 30 minutes. Stress relief treatment at more than 580 °C, or for over an hour, may lead to deterioration of the mechanical properties. If it is intended to stress relieve S420 to S700 steels at higher temperatures or for longer times, the required minimum values of the mechanical properties should be agreed in advance with the product manufacturer.

## 9.8 Cold cutting

Shearing and punching can only be used to cut HSS in moderate thicknesses (usually up to 10 mm). However, abrasive water jet (AWJ) cutting can successfully cut HSS with no limit on thickness.

## 9.9 Machining

HSS can be machined without any difficulty using the same methods as those used for conventional strength steels. Depending on the type of machining work, sufficient cooling should be applied because an interruption in the coolant and lubricants may lead to overheating the cutting edge. Overheating may cause increased tool wear, and in extreme cases lead to tool breakage.

## 9.10 Erection and handling

There is no fundamental difference between the handling of HSS and conventional strength steels. HSS components are likely to be lighter and have smaller moments of inertia than equivalent members made of conventional strength steel. This is usually an advantage during handling and shipping, but may result in the requirement for additional bracing for stability during transportation and erection.

# WELDING

## 10.1 Introduction

HSS are generally weldable. The welding characteristics of HSS vary slightly according to the production route and the individual characteristics of the HSS grade. Compared to conventional strength steels, there are no significant differences in the applicable welding processes – all common fusion welding methods can be used. However, given the additional complexities associated with HSS, it is essential that any welding operations are more carefully controlled and operated under the supervision and direction of an appropriately qualified welding engineer.

Where flaws are generated as a result of the welding operations, they will be located either in the weld metal or in the parent metal adjacent to the weld (HAZ). The chemical composition of the filler material is therefore very important in achieving the desired strength and toughness of the weld. Flaws arising from the thermal cycle associated with the welding can be avoided by strict adherence to an approved welding procedure.

Site welding should be completed to the same high standard as welding in the workshop, with identical requirements for Welding Procedure Specifications and qualified welders as discussed in subsequent sections. Site welding is likely to demand additional provisions, for safe access and weather protection, for example, but no reduction in weld quality should be allowed. In the UK, it is common practice that welding should not be carried out when ambient temperature is below 5 °C. Guidance on welding HSS is given in BCSA Publication 63/20[88].

## 10.2 Weldability

### 10.2.1 Carbon equivalent value (CEV) and material supply condition

The Carbon Equivalent Value (CEV) is often referred to when assessing the weldability of steels. It is actually a measure of the steel's hardenability which is determined by the chemical composition. There are several CEV formulae in existence. Equation (10.1) below is the International Institute of Welding (IIW) definition of CEV, included in many codes of practice for the welding of steels, such as EN 1011-2[89], which is used throughout Europe. However, it is not suitable for steels containing boron.

$$\mathrm{CEV} = \mathrm{C} + \frac{\mathrm{Mn}}{6} + \frac{(\mathrm{Mo} + \mathrm{Cr} + \mathrm{V})}{5} + \frac{(\mathrm{Ni} + \mathrm{Cu})}{15} \quad [\%] \tag{10.1}$$

In general, a higher CEV leads to a higher hardenability and a greater risk of hydrogen (cold) cracking in the welded joint. The maximum permitted CEVs for HSS plates and hollow sections in European product standards are given in Table 10.1 to Table 10.4.

The lower CEVs of M type steels mean they are readily weldable with regards to the avoidance of HAZ hydrogen cracking. It should however be noted that M type steels are likely to experience some degree of HAZ softening, and therefore restrictions on the heat input, so as not to degrade the joint properties, are typically recommended.

By contrast, the Q type steels have higher CEVs and hardenability and hence are more susceptible to the risk of HAZ hydrogen cracking. Equally, welding heat input restrictions may also be applied to avoid the risk of over-tempering the steel leading to a degradation of the mechanical properties. HAZ softening is less pronounced with Q steels.

*Table 10.1 Maximum specified CEV for Normalized HSS (EN 10025-3)*

| **Steel grade (N and NL)** | **Max. CEV in % for nominal product thickness [mm]** | | |
|---|---|---|---|
| | **≤ 63** | **> 63 ≤ 100** | **> 100 ≤ 250** |
| S420 | 0.48 | 0.50 | 0.52 |
| S460 | 0.53 | 0.54 | 0.55 |

*Table 10.2 Maximum specified CEV for Thermomechanical rolled HSS (EN 10025-4)*

| **Steel grade (M and ML)** | **Max. CEV in % for nominal product thickness [mm]** | | | |
|---|---|---|---|---|
| | **≤ 16** | **> 16 ≤ 40** | **> 40 ≤ 63** | **> 63 ≤ 150** |
| S420 | 0.43 | 0.45 | 0.46 | 0.47 |
| S460 | 0.45 | 0.46 | 0.47 | 0.48 |
| S500 | 0.47 | 0.47 | 0.47 | 0.48 |

*Table 10.3 Maximum CEV for Quenched and Tempered HSS (EN 10025-6)*

| **Steel grade (Q, QL and QL1)** | **Max. CEV in % for nominal product thickness [mm]** | | |
|---|---|---|---|
| | **≤ 50** | **> 50 ≤ 100** | **> 100 ≤ 200** |
| S460 | 0.47 | 0.48 | 0.50 |
| S500 | 0.47 | 0.70 | 0.70 |
| S550 | 0.65 | 0.77 | 0.83 |
| S620 | 0.65 | 0.77 | 0.83 |
| S690 | 0.65 | 0.77 | 0.83 |

*Table 10.4 Maximum CEV for HSS hollow sections (EN 10210 and EN 10219)*

| Max. CEV in % for nominal product thickness [mm] | | | | |
|---|---|---|---|---|
| **Steel grade (NH and NHL)** | **≤ 16** | **> 16 ≤ 40** | **Steel grade (MH and MHL)** | **≤ 40** |
| S420 | 0.50 | 0.52 | S420 | 0.43 |
| S460 | 0.53 | 0.55 | S460 | 0.46 |

CEVs for commercially available steels can be lower than these maximum values. For this reason, when ordering HSS it may be possible to specify more onerous requirements in terms of weldability. This should be done in consultation between the welding coordinator and the material producer to ensure that the requirements are both directly beneficial and realistic.

### 10.2.2 Welding consumables

Welding consumables are chosen based on the strength and toughness requirements for the weld. Recommendations for consumables for HSS are available from both steel producers and consumable manufacturers. Typically, such recommendations include the use of under-matched strength welding consumables and, to reduce the risk of hydrogen cold-cracking, the use of low hydrogen consumables and welding practices. They are usually more expensive than consumables for conventional strength steel. Consumables are classified in European (EN) and American (AWS) standards, based on the strength classes and welding process used, with specific standards for different processes. The CEV of consumables for welding HSS are higher than consumables for welding conventional strength welds. Whereas the parent material gains its high strength from careful control of microstructure and grain refinement, consumables gain their strength only from microalloying. This means that the filler metal for HSS could equally suffer from hydrogen cracking before the parent material. Parameters may have to be controlled based on the filler metal rather than the parent metal as is usually the case for conventional strength steel.

If the stresses acting on the weld are expected to be high and the tensile strength of the weld needs to be equal to or close to the strength of the parent metal, matching filler metals are adopted. The use of under-matched filler metal reduces the risk of cracking and therefore should be considered if the weld is in a low-stress area.

For N and M type steels with guaranteed impact properties of 40 J at -20 °C, a standard Carbon-Manganese filler wire is usually adequate. For the NL and ML steels, or where there is any doubt that the specified properties can be achieved, a 1 – 3 % nickel-based consumable may be selected.

It is essential that consumables are used in accordance with manufacturers' recommendation.

## 10.3 Welding coordination

Ideally a welding coordinator should be appointed who is based at the manufacturing site and responsible for overseeing welding operations. Standard or comprehensive knowledge of EN ISO 14731[90] is required, in accordance with Table 14 of EN 1090-2[6]. As a minimum, this requires that the welding coordinator has the level of technical knowledge as that described in EN ISO 3834-5[91].

## 10.4 Qualification of welding procedure and welding personnel

It is recommended that the execution of HSS is undertaken using a welding procedure specification (WPS) backed up by an independently certified welding procedure qualification record (WPQR) containing test results for strength, hardness, toughness, material and calibration certificates, etc.

The preparation and testing of specimens used to establish the WPQR should address the key issues which may lead to impairment of properties in the HAZ of the parent metal. Issues such as joint preparation, consumables, energy input and the link between diffusible hydrogen and preheat levels should all be covered. The WPQR may also address joining dissimilar materials and the use of under-matched electrodes.

Wherever possible, it is recommended that WPQRs are qualified using project specific material, i.e. is the steel used for the steel test pieces the same as that which will be used in the actual construction?

The qualification of the welding procedure for welding process 111 (manual metal arc welding), 114 (self-shielded tubular-cored arc welding), 12 (submerged arc welding), 13 (gas shielded metal arc welding) and 14 (gas shielded arc welding with non-consumable electrode)  is given in Table 12 of EN 1090-2 with additional requirements for Q steels according to EN 10025-6[1].

If a qualification procedure is required for fillet welds in steel grades of S460 and above, EN 1090-2 requires that a cruciform tensile test be performed in accordance with EN ISO 9018[93]. Alternatively, if the fillet weld throat of an under-matched consumable is increased to compensate, then an all weld metal tensile test can be performed and compared with the actual tensile strength declared for the consumable.

It is equally important that welders and welding operators are appropriately qualified to demonstrate their ability to produce sound welds in HSS. Minimum requirements are set out in EN 1090-2, i.e. welders in accordance with EN ISO 9606-1[94] and welding operators in accordance with EN ISO 14732[95].

## 10.5 Joint preparation and execution of welding

Thermal cutting and machining are the most common methods used for joint preparation (see Sections 9.4 and 9.9). All conventional methods for joint preparation can be used with HSS[96]. During thermal cutting, a thin oxide film may form on the joint surface which should be removed before welding. For thin plates, ordinary shearing can be used in joint preparation. It is advisable to remove the primer for best results. For steel grades higher than S460, EN 1090-2 requires that cut areas be descaled by grinding and verified to be free from cracks by visual inspection, dye penetrant or magnetic particle testing. Visible cracks should be removed by grinding and the joint geometry corrected as necessary.

EN 1090-2 also requires that precautions be taken to avoid stray arcing, and if stray arcs do occur outside the weld fusion face, the surface of the steel should be lightly ground and checked. Visual checking for steel grades S460 and above should be supplemented by penetrant or magnetic particle testing. Weld spatter should be removed for steel grades S460 and greater.

Where possible, temporary attachments should be avoided. Where this is not possible the location of the temporary attachment should be agreed with the engineer and/or designer. EN 1090-2 requires that the removal of temporary welded attachments by cutting, gouging or chipping should be carried out in such a way that the parent metal is not damaged and shall subsequently be carefully ground smooth. The removal locations should be visually inspected and for steel grades S355 and above, subjected to NDT. Chipping and gouging are not permitted on steel grades S460 and above, unless otherwise specified.

More distortion during welding is to be expected with HSS compared to conventional strength steel. To overcome this, tighter control is required over sequencing of welds and more jigging and clamping may be necessary.

## 10.6 Mechanical properties of welds

In order to reach the required mechanical properties of the welded connection, both the weld metal and the HAZ must have sufficient strength and toughness. The strength of the welded joint depends on several factors such as the filler material (matched or under-matched, see Section 6.4.1), chemical composition (CEV), heat input and preheat and interpass temperature. (The preheat temperature is the temperature to which the surfaces to be welded are heated before welding starts and the interpass temperature is the temperature at which subsequent weld runs are deposited.)

The strength of the weld metal is mainly determined by the filler material used whereas the strength of HAZ is determined by the cooling time ($t_{8/5}$).

### 10.6.1 Heat input

The heat input ($Q$) in fusion welding is calculated by Equation (10.2) below. It depends on the voltage, current and travel speed. Although higher heat input leads to higher productivity for conventional welding methods, reduced heat input is preferred because it leads to better toughness, increased strength, reduced deformation (particularly for thinner HSS materials), lower residual stress and a narrower HAZ. Guidance on appropriate heat input controls for different grades and supply conditions can be sought directly from the steel producers.

$$Q = k \times E \tag{10.2}$$

$$E = \frac{U \times I \times 60}{v \times 1000}$$

where:

$E$ is the arc energy

$U$ = voltage [V]

$I$ = current [A]

$v$ = welding speed [mm/min]

$k$ = thermal efficiency = 0.8 (MAG, MMA); 1.0 (SAW); 0.6 (TIG)

### 10.6.2 Cooling time $t_{8/5}$

The temperature-time cycles during welding have a significant effect on the mechanical properties of a welded joint. The cooling time ($t_{8/5}$) refers to the time it takes in seconds for the weld to cool down from 800 °C to 500 °C. This cooling rate is critical, because it is during cooling between these temperatures that the microstructure of the material reforms and the toughness, strength and hardness of the HAZ are established.

In practice, there are two welding parameters that can be used to adjust $t_{8/5}$: heat input and preheat temperature. Increasing the heat input and preheat temperature leads to slower cooling and thus a longer cooling time $t_{8/5}$. Knowing the welding parameters and geometry, the cooling time $t_{8/5}$ can be determined according to EN 1011-2[89]. If no curves for the relationship of impact energy, impact transition temperature and hardness as a function of $t_{8/5}$ are available, welding procedure tests in accordance with EN ISO 15614-1[97] and EN ISO 15613[98] are recommended. An upper and lower limit of $t_{8/5}$ is required to obtain good mechanical properties in the weld and HAZ. When the cooling time ($t_{8/5}$) is short (e.g. low heat input, thick plate or lower working temperature), the hardness of the HAZ may exceed limiting values with the risk of introducing hydrogen induced cracks. A long cooling time results in poor strength and toughness in the HAZ. Acceptable properties can normally be obtained with a cooling time between 5 to 20 seconds for HSS up to S700 and reduced to 5 to 15 seconds for higher strengths.

### 10.6.3 Softening in the HAZ

When HSS are welded, soft zones can be formed in the HAZ as a result of changes in the microstructure. The width and hardness of the soft zone are primarily determined by the material thickness and the heat input. Thin plates and high heat input lead to a wider zone and lower strength. It is possible to achieve the minimum specified strength of the parent material if a suitable heat input is used.

### 10.6.4 Hydrogen (cold) cracking

Cracks may occur at temperatures below 200 °C in the HAZ after welding if the microstructure has inadequate toughness, and hydrogen and high residual stresses are present in the weld. The risk of crack formation in the weld can be associated with the alloying content of the steel: the lower the CEV, the lower the risk of cracking after welding.

Hydrogen may be present in the welded area originating from the welding consumable or from impurities on the joint surface (rust or frost, etc.). Residual stresses occur in the weld due to shrinkage when the material cools down.

As noted previously, the low CEV of M type steels means that they generally exhibit a lower sensitivity to hydrogen cracking. By comparison, the higher CEV of Q type steels means the opposite is generally the case, and they tend to be more sensitive to the risk of hydrogen cracking. In any event, appropriate precautions to avoid hydrogen cracking should be taken. Annex C of EN 1011-2 provides guidance on hydrogen cracking and its avoidance by adjusting the minimum preheating temperature.

## 10.7 Preheat and interpass temperature

Preheating reduces the risk of hydrogen cracking of weld joints. It leads to a delay in the cooling of the component after welding so that the hydrogen has enough time to diffuse out, which mainly takes place in the temperature range between 100 °C and 300 °C.

As well as preheating, the interpass temperature has to be maintained during welding. The heated area should extend at least 100 mm on either side of the seam. The interpass temperature and heat input should be kept relatively low to help prevent grain growth (lowering toughness) and over-tempering (lowering strength).

The need for preheating is determined according to Annex C of EN 1011-2. For example, when the CEV of the steel, the combined thickness of the joints to be welded, the hydrogen scale of the welding consumable and the heat input are known, recommended preheat and interpass temperatures are given in Annex C of EN 1011-2, Method A.

Steel producers will also be able to give advice on preheat and interpass temperatures.

## 10.8 Welding inspection

Inspection and testing of HSS welds should be carried out according to EN 1090-2, as relevant to the Execution Class. The proposed post-welding inspections should be evaluated as part of the welding procedure specification (WPS).

Hydrogen cracking is often referred to as ‘delayed cracking’, as it can occur as post weld cracks. The supplementary NDT of a HSS weld shall not be completed until the minimum hold time after welding. The minimum hold times vary between 8 – 48 hours and are given in Table 10.5.

| Hold time [hours] | | | |
|---|---|---|---|
| **If preheat is applied in accordance with Method A of EN 1011-2:2011, Annex C** | | | |
| **Weld size [mm] *** | **Heat input Q [kJ/mm]** | **S420 – S460** | **Above 460** |
| $a$ or $s \leq 6$ | All | Cooling period only | 24 |
| $6 < a$ or $s \leq 12$ | ≤ 3 | 8 | 24 |
| | > 3 | 16 | 40 |
| $a$ or $s > 12$ | ≤ 3 | 16 | 40 |
| | > 3 | 24 | 48 |
| **If preheat is applied in accordance with Method B of EN 1011-2:2001, Annex C** | | | |
| **Weld size [mm] *** | **S420 – S690** | | |
| $a$ or $s \leq 20$ | Cooling period only | | |
| $a$ or $s > 20$ | 24 | | |

* Size applies to the nominal throat $a$ of a fillet weld or the nominal material thickness $s$ of a full penetration weld. For individual partial penetration butt welds the governing criterion is the nominal weld depth $a$, but for pairs of partial penetration butt welds welded simultaneously it is the sum of the nominal weld throats $a$.

*Table 10.5 Minimum hold times for HSS (adapted from Table 23, EN 1090-2)*

As noted, the above hold times are the minimum requirements given in EN 1090-2. For critical applications, consideration should be given to applying a more onerous requirement. For example, all HSS might be subject to a blanket 48 hour hold time prior to NDT, which might also be linked to the CEV of the steel in question. Additionally, consideration should be given to applying increased levels of inspection. For example, the UK typically requires 100 % inspection of all site welds. See also Section 10.1.

## 10.9 Post-weld heat treatment

Post-weld heat treatments (PWHTs) are mainly used to reduce residual stresses or allow hydrogen to diffuse out of the steel to avoid cracking (this is generally carried out at around 200 °C). Where PWHT is required by the code of practice, guidance should be sought from the steel producer to ensure that the PHWT does not inadvertently adversely affect the mechanical properties of the steel.

The fabricator should demonstrate the effectiveness of the chosen post-weld heat treatment procedures for HSS.

Guidance on stress relieving is given in Section 9.7.

## 10.10 Welding of dissimilar materials

When welding HSS to conventional strength steel, the weldability of both steel types must be considered. All the procedures regarding total heat input, care and maintenance of consumables, and control of consumable and weld metal hydrogen content should be followed as if welding were being performed on HSS only, whilst the strength of consumable should be based on the lower strength steel. However, the weld metal does not need to have a tensile strength higher than that of the lowest strength material in the joint. Hydrogen control is still essential for welding of dissimilar steels.

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# APPENDIX A PRODUCTION OF HSS

## A.1 Introduction

The European structural steel standard for hot-rolled structural steel products, EN 10025, consists of six parts, which include requirements for the technical delivery conditions of conventional strength structural steel flat products through to high strength structural steels in the quench and tempered condition. Typically, it is assumed that the delivery conditions are at the discretion of the producer if no specific delivery condition is ordered. EN 10025 uses the following symbols to describe the delivery conditions.

| | |
|---|---|
| S | Structural steel |
| +AR | Supplied in the As Rolled condition |
| +N | Supplied in the Normalized or Normalized Rolled condition |
| +CR(NR) | Controlled Rolling (Normalized Rolled) |
| TM | Thermo-Mechanically Rolled |
| M | Longitudinal Charpy V-notch impacts at a temperature not lower than -20 °C |
| ML | Longitudinal Charpy V-notch impacts at a temperature not lower than -50 °C |
| N | Normalized or Normalized Rolled with longitudinal Charpy V-notch impacts at a temperature not lower than -20 °C |
| NL | Normalized or Normalized Rolled with longitudinal Charpy V-notch impacts at a temperature not lower than -50 °C |
| DQ | Direct Quenched |
| Q | Quenched and Tempered with longitudinal Charpy V-notch impacts at a temperature not lower than -20 °C |
| QL | Quenched and Tempered with longitudinal Charpy V-notch impacts at a temperature not lower than -40 °C |
| QL1 | Quenched and Tempered with longitudinal Charpy V-notch impacts at a temperature not lower than -60 °C |
| QST | Quench and Self-Tempered |

# A.2 Production routes

## A.2.1 HSS plate

In comparison to conventional strength steel plates (≤ 350 MPa yield strength), the production of HSS plates differs in: (i) the steel chemical composition, and; (ii) the conditions of hot-rolling and subsequent cooling. As with all steels, the combination of (i) and (ii) determines the final steel microstructure, which ultimately defines the mechanical properties of the steel. In general, compared to the production of conventional strength steels, producing high strength steels requires additional alloying elements and typically stricter control practices in rolling, as well as a more aggressive cooling to a lower final temperature followed by a heat treatment or tempering step.

In essence, the microstructure of high strength steels tends to be much finer (refined) and because they are produced at a lower temperature, it tends to be much harder (and thus have higher strengths) due to greater quantity of dislocations that are generated within. Consequently, a heat-treatment or tempering step is often required to soften the microstructure to restore some appreciable level of ductility whilst not losing too much overall strength.

Figure A.1 provides a schematic diagram of the various processing routes that are typically used in the production of HSS plates. The following sections describe the differences between the various routes and highlights some of the critical process and steel alloy design factors to be taken into consideration when selecting steel plate grades.

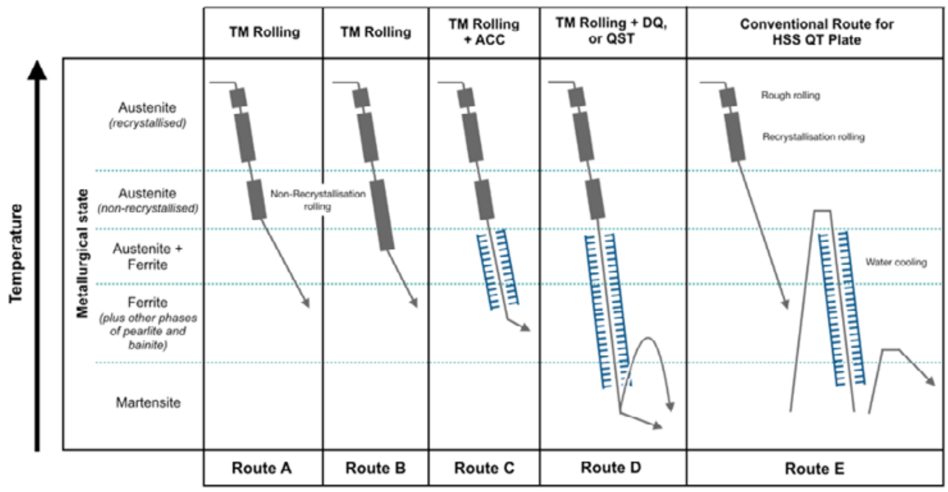


*Figure A.1 Schematic of various processing routes typically used in production of high strength plates*

**Routes A and B** encapsulate the starting point in the production of most HSS plate products, that being the process route of Thermo-Mechanical (TM) rolling. This is a specific rolling practice which is designed to improve the final mechanical properties by controlling the individual hot deformation process steps in producing the final external shape of the product. Unlike conventional rolling practices used for As-Rolled (AR) or Normalized (N) routes where the plate is essentially hot rolled only for external shape control and finished rolled in the fully recrystallised austenite phase, TM rolling equally focuses on refining the internal microstructure of the steel, leading to a structure with a fine effective grain size. A finer final ferrite grain size, transformed from the higher temperature austenite microstructural phase, does not only give higher strengths but also improve low temperature impact properties. It is important to recognise here that grain refinement is the only metallurgical mechanism that leads to both higher strengths and improved toughness. Other available mechanisms can contribute towards higher strength, but they are detrimental to other desired properties such as toughness and or weldability.

However, in order to TM roll and produce HSS plate, and other products, a change in alloy design is necessary. It is here where microalloying elements, such as niobium (Nb), vanadium (V) and titanium (Ti) are employed at very low levels ranging from 0.010 to 0.10 wt. %. Synonymous with TM plate rolling is the use of niobium at typical levels ranging from 0.015 to 0.040 wt. %. Amongst the commonly used microalloys, niobium is overwhelmingly preferred because it is able to retard the recrystallisation of austenite in between rolling passes at temperatures which are compatible with the plate rolling process. This metallurgical process is crucial as it allows the benefits of strain accumulation and the potential for a greater ferrite grain nucleation from the austenite structure with each rolling pass due to the formation of nano-sized niobium precipitates. It is this process, which is referred to as TM rolling, or sometimes referred to as ‘pancaking’ the austenite structure. Consequently, on transformation, greater number of finer ferrite grains are produced leading to higher strengths and improved toughness.

The added advantage of employing niobium is that this metallurgical process accordingly permits a reduction in the overall alloy design of the steel, i.e. in particular reducing the carbon (C) content. This is very important because steels that contain lower amounts of carbon exhibit high toughness values, better through-thickness properties and improved weldability at identical or higher yield and tensile strengths. Therefore, on occasions it is beneficial to use a lower carbon microalloyed TM rolled plate rather than a N-rolled plate, as the latter will tend to have much higher carbon content for the equivalent strength.

In principle, Routes A and B share the same starting rolling practices but differ in their final stages of rolling deformation (i.e. the finish rolling sequence). In general, the rolling of plate steels can be split into a number of key stages: (i) sizing, to get the starting slab to the intended plate width by cross rolling; (ii) roughing, to break-down the internal austenite microstructure from the as-cast condition through a process of repeated recrystallisation; (iii) finishing, to reduce the plate down to the final thickness. Similar to the first two stages, the finishing stage is mainly carried out in the high temperature austenite microstructural phase (i.e. above the $Ar_3$ temperature, which is the temperature at which the austenite grain starts to transform to ferrite). However, for some applications the rolling can also be undertaken into the two-phase, austenite and ferrite region. This generates a mixed structure consisting of equiaxed grains and sub-grains after transformation and, thereby, it increases further the strength and toughness. The development of this type of microstructure can be difficult to control to ensure homogeneous properties through the plate thickness and so is not widely practised. Thus, the main difference between Routes A and B is that in the former the steel is finished rolled above the $Ar_3$, i.e. in the fully austenite condition, whereas in the latter, it is finished rolled below the $Ar_3$, partly into the ferrite phase. In both cases the as-rolled plate is cooled in air to room temperature.

The fabrication of steel plates, irrespective of the route chosen, starts from a slab that has been reheated ('soaked') in a furnace to a predetermined temperature. For conventional strength steels, the temperature of the slab is primarily set to allow ease of deformation for the duration of rolling. Naturally, the higher the temperature, the easier it will be to hot-roll and the less demands are put onto the rolling mill (lower mill loads and torque). However, this needs to be balanced amongst other factors with the cost of fuel and time (lost productivity) spent heating the slabs and also the time required to roll the slab to the final plate thickness. The thinner the plate, the longer the rolling times. Furthermore, as not all steel plants and plate mills are the same, some older mills may have limitations on the amount of deformation per rolling pass they can do, and the starting slab size may vary.

These physical constraints gain in metallurgical significance as the thickness of the final plate increases. In general, most plate mills work based on a rule of thumb that says that the starting slab thickness should be at least three times greater than the final plate thickness being produced. For example, a 50 mm plate should have a starting slab thickness of at least 150 mm. However, in reality more is preferable as normally the starting slab is sized to the correct width before the 'real' metallurgical rolling takes place, and these initial rolling passes will reduce the slab thickness. In addition, to obtain adequate z-directional properties and better low temperature toughness properties, it is recommended that the maximum possible per-pass reduction during the initial stage (roughing passes) are made as this increases the developed strain at the core of the slab and consequently the plate. This is sometimes referred to as high shape factor rolling and has proven to be beneficial to the final properties.

As TM rolled steels are microalloyed, the final slab temperature is also of great importance to ensure that the required dissolution of the microalloying elements takes place. This is of particular relevance for niobium which again has a temperature range that works well to the generally practiced furnace operating temperatures. For vanadium microalloyed steels, the dissolution temperature is lower and so all the microalloy addition will be in solution at typical slab reheating temperatures.

With respect to the delivery state, due to the variation in TM practices, TM rolled plates are marked either as "M' indicating that the minimum values for impact energy have been defined at temperatures no lower than –20 ºC, or 'ML', indicating the minimum values for impact energy have been defined at temperatures no lower than –50 ºC.

**Route C** is effectively an extension of Routes A and B, coupled together with a post-rolling water cooling stage. Application of water cooling is referred to as 'accelerated cooling - ACC' and sometimes plates produced in this way are also referred to as TMCP produced plate (Thermo-Mechanically Controlled Processing), although this term is also often interchangeably used to describe TM rolled plates.

These plates are controlled cooled using pressurised water applied from top and underside (bottom) of the plate on exiting the rolling mill stands. The water is applied either by an arrangement of top and bottom nozzle header units, or via the plate passing through a specialised accelerated water cooling unit capable of generating much higher rates of controlled cooling. In all cases the cooling rate is specifically defined and carefully controlled depending on the plate alloy-design, thickness and the target final properties. It is important to note that Route C does not cool the plate to room temperature, but to a temperature between $Ar_3$ and $M_s$. ($M_s$ is the temperature below which a martensite microstructure will form from any remaining untransformed austenite phase, it is typically below 500 ºC, but is composition dependent.)

There are number of benefits to be gained from the application of controlled cooling, especially after TM rolling: (i) developing a much finer ferrite grain size from austenite in comparison to an air-cooled plate and thus higher strengths with improved toughness; (ii) developing other lower temperature transformation microstructures such as refined pearlite and bainite which also contribute to higher strengths; (iii) generating additional strength by forming nano-sized precipitates from the microalloys of niobium and/or vanadium; (iv) to reduce the risk of lamellar tearing by alleviating the presence of an internal banded microstructure, i.e. alternate layers of ferrite and pearlite, and; (v) allowing a much lower carbon content to be  and thus improving weldability as well as toughness.

As in the rolling stages, temperature control and temperature uniformity across the plate width, length and through thickness is important, as ultimately this has a very strong influence on the metallurgy. This becomes even more critical when cooling with water, as the mill must account for temperature gradients along the plate from the front (head) to the back (tail) of the plate, and from the surfaces to the centre (core). If the correct cooling rate is not applied, then there is a greater risk of developing unwanted microstructures leading to inconsistent mechanical properties along the plate and even poor shape (flatness). For plate up to a certain thickness, a poor shape can be rectified at the plate mill by immediately hot-levelling, although not all plate mills have hot levelling facilities. However, this does not resolve inconsistencies in mechanical properties. Furthermore, hot-levelling induces residual stresses that may not be apparent until the plate enters fabrication. Therefore, and as mentioned before, it is important for the engineer to understand how the final plate was actually rolled and cooled from a given mill.

The majority of plate grades from S355 to S460 can be easily produced via the TMCP (TM+ACC) route by most plate mills. However, not all mills have the capability of producing the thicker (heavier) plates at the higher strengths.

**Route D** is effectively a variant of Route C, but with the application of a much faster cooling rate post TM-rolling achieved by directly quenching the plate below the $M_s$ temperature and subsequently tempering in a secondary heat-treatment step. Plates produced via this route are also referred to as 'Quench and Tempered' or "QT' steels. The advantage of being able to do this directly from TM-rolling is that a refined martensite structure is developed and can be tempered resulting in enhanced impact properties when compared to a QT steel produced via the conventional route. Typically, higher strength plates up to S960 and even S1100 can be made at minimum values for impact energy at temperatures of -20 ºC (S960Q and S1100Q). However, it should be noted that only a limited number of plate producers are actually able to follow Route D due to equipment limitations, as it requires the direct quenching unit to be in-line with the existing rolling mill. In this case, the actual plate thickness will be limited by the online direct quenching capability, therefore for higher strength, heavy plate a conventional QT route must be adopted.

**Route E** outlines the conventional process taken when producing high strength QT plates, whereby plates are rolled in the fully austenitic condition, without the need for any TM-rolling treatment, and air cooled to room temperature. The plates may still contain some microalloying elements as they are required to develop the correct microstructures and thus properties.

Following the quenching process, plates are heat-treated in a roller hearth furnace back into the fully austenitic recrystallised region and then water quenched rapidly to room temperature, developing a harder microstructure predominantly consisting of martensite and low temperature bainite. However, as these structures have poor ductility and are brittle in nature, the plates are subject to a secondary low temperature heat-treatment step in another roller hearth furnace, typically at temperatures near 600 ºC for a specific period of time based on the steel alloy-design, plate thickness and final property requirements. For example, a given steel plate can be tempered to produce different products from the same chemical composition to give yield strengths of 600 to 1000 MPa. This is all possible by altering the steel microstructure from the tempering process. Therefore, understanding the effects of the various microstructures developed during tempering on the mechanical properties is important to both the optimum design of the material and industrial application. Typically, tempering coarsens the martensite laths, reduces dislocation densities, and grows carbide precipitates including those formed from niobium and vanadium microalloys.

Overall, quenching and tempering achieves an extremely fine-grained and homogeneous microstructure which is characterised by high strength and good ductility. For example, this route is used to make S500 plate with minimum impact values down to -60 ºC, at thicknesses up to 150 mm (S500QL1), S960 plate with minimum impact values down to -40 ºC, at thicknesses of 100 mm (S960QL) and recently even S1100 plate at thicknesses up to 50 mm (S1100QL). All QT plates produced, irrespective of process route, are marked as 'Q'.

## A.2.2 HSS Hollow structural sections

HSS hollow sections are supplied in either a hot finished (in accordance with EN 10210) or cold formed condition (in accordance with EN 10219), and either in circular, square, rectangular or even elliptical shapes at strengths ranging from 420 MPa up to 700 MPa in yield strength. Cold formed hollow structural sections up to S960 are also available; though outside the current scope of the product standard, they meet its requirements where applicable. Although HSS cold formed sections are commonly used in structural applications, there is increasing use of HSS hot finished sections, both welded and seamless. For the latter, outer diameters of up to 660 mm can be sourced with yield strengths of 480 MPa, with wall thicknesses up to 50 mm.

The initial production routes for hot-finished and cold-formed HSS sections are essentially the same, whereby HSS strip material that is designed for tubular manufacture is supplied in slit coil form. The coiled strip steel is de-coiled into an accumulator and the edges are milled to prepare them for welding. The strip is then formed into a tube by progressing through a series of forming rolls, where on exiting the edges (seam) are welded. The welding process can take the form of Electric Resistance Welding (ERW), which includes low, medium or high frequency welding techniques, or welding using a consumable electrode as found in Submerged Arc Welding (SAW). All the welding process is continuous in nature.

The most common welding process in the manufacture of most hollow sections is via ERW using high frequency welding (HFI) (typically ≥ 70 kHz). A major advantage is the high productivity attainable due to the high welding velocities inherent with the HFI process. High velocity means high heating and cooling rates. This kind of fast thermal cycle favours the formation of bainitic structures over martensitic structures and results in better post-processing properties. The strip width depends on the tube diameter, and very tight tolerances apply. The longitudinal strip edges are heated electrically and pressed together to achieve binding.

Once formed into a tube, it is either cold formed into the final shape by passing through another set of shaping rollers (i.e. cold formed) or heat-treated (i.e. normalized by heating the tube to around 860 ºC) and then finished rolled into the desired final shape and controlled cooled (i.e. hot finished). In some facilities tubes are not heat-treated to high normalizing temperatures and are instead ‘warm’ formed, but conventional practice is to normalize whereby the tubes are processed or heat-treated in the normalizing temperature range (i.e. in the austenite microstructure); these are designated ‘NH’. Note that the longitudinal weld can be annealed and controlled cooled (water or air cooling) after welding, or in the case of hot-finished, is full body annealed prior to forming into the final shape.

Due to this heat-treatment step, HSS hot finished hollow sections have to maintain sufficient strength and toughness properties from the original starting high strength strip material. As the steel is effectively fully austenitized, the initial fine-grained structure will become coarser resulting in reduced properties. Consequently, the supplied HSS strip contains higher levels of microalloying elements such as niobium and in particular vanadium, together with additions of molybdenum, chromium and even nickel for the very higher strength levels. Suitable combinations of microalloys with molybdenum provide enhanced fire resistance by maintaining strength for longer time periods when exposed to the temperature range of 550 °C to 650 °C. For modern HSS hot finished sections, the main challenge is not necessarily meeting the strength, but rather achieving the very low temperature toughness property (when required), especially at higher strengths and large wall thicknesses.

### A.2.3 HSS long products

As with HSS plates, the production route for HSS long products (i.e. hot rolled open sections used for beams and columns) will generally follow Routes A, C and D, as schematically shown in Figure A.1. In addition, due to the differences in the physical rolling equipment of rolled plates versus long products, the latter can also use a conventional rolling procedure, as shown in the first part of Route E, where all of the hot rolling is undertaken in the fully austenitic condition and air cooled or water cooled to room temperature.

As with HSS plate products, the production of HSS long products differs from lower strength products in: (i) the steel chemical composition, and; (ii) the conditions of hot-rolling and subsequent cooling. As with all steels, the combination of (i) and (ii) determines the developed final steel microstructure, which ultimately defines the mechanical properties of the steel. In general, to achieve higher strengths requires the addition of specific alloying elements, typically stricter control practices in rolling, and a more aggressive cooling to a lower final temperature followed by a tempering step (QST). Unlike HSS plate products, long products are generally not subjected to further offline heat-treatment processes. Furthermore, they are subjected to shorter processing times in order to maximise the productivity through the production line. This means that unlike plate rolling, pieces cannot be held back (delayed) to allow for temperatures to naturally drop for TM-rolling.

A key distinction in the rolling of flat products versus long products is that the starting stock for the former are from continuously cast slabs, and for the latter, are blooms or beam-blanks. Blooms are square or rectangular cross-section casts typically from 220 x 220 mm up to 750 x 850 mm. These are used to make both H-beams, I-beams and other sections. However, medium and heavy H-beams are typically rolled from continuously cast beam blanks. These are near-net shape casts that resemble a dog-bone shape ranging from 250 x 200 x 80mm to 1150 x 490 x 130 mm in size.

## A.3 Availability

### A.3.1 HSS plate

In recent years, the operational capability of some of the world leading plate mills has significantly improved, in particular the ability to cast and roll much thicker starting steel slabs. This has opened up the possibility to produce much heavier (thicker) plates with improved low temperature Charpy V-notch impacts and higher strengths. Furthermore, with the installation of upgraded online direct quenching facilities, there are more plate mills capable of supplying a wider range of QT plates. Figure A.2 provides some guidance on the availability of HSS plate.

### A.3.2 HSS hollow structural sections

In recent years the operational capability of some of the world leading hollow section mills has significantly improved, in particular the ability to process higher strength strip grades at greater wall thicknesses. Furthermore, increased capability at some oil and gas seamless pipe producers has also meant greater availability of higher strength, excellent low temperature Charpy V-notch impacts and very thick walled seamless hot-finished tubes for structural applications. Figure A.3 provides an example of the availability of HSS cold-formed circular sections. For reference, the figure also shows the general dimensional matrix for S355. It is worth noting that most producers can supply beyond that shown in Figure A.3 and therefore enquiries should be sought if required.

### A.3.3 Long products

The operational capability of some of the world's leading producers of long products has significantly improved in recent years. In particular, the ability to cast and roll much heavier pieces, and the addition of QST capabilities, has meant that much heavier (thicker) beams with improved low temperature Charpy V-notch impacts and improved weldability are possible at higher strengths using lower carbon equivalent steels. Today, as-rolled H-sections are readily available in a variety of dimensions and steel grades up to S460. The largest currently available are extra heavy rolled wide flange sections with up to 140 mm flange thickness and weighing up to 1377 kg/m in S460M in accordance with EN 10025. Naturally, thicker flanges and even higher strength sections can be fabricated directly from plate.

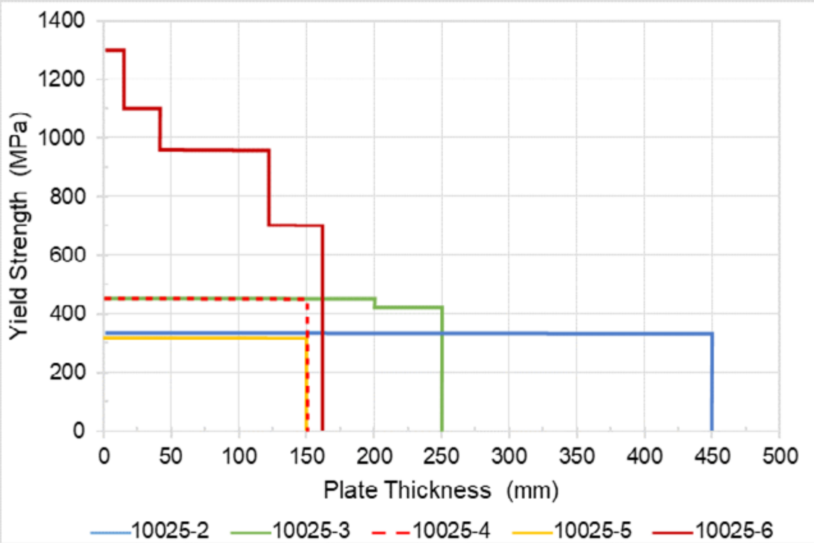


*Figure A.2 Availability of HSS plate in the various EN 10025 delivery conditions with respect to yield strength and maximum plate thickness*

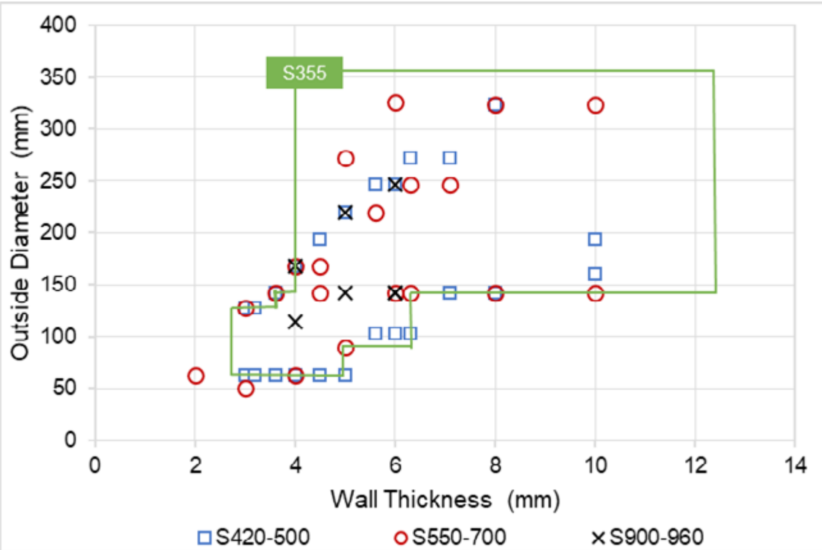


*Figure A.3 Typical availability of HSS cold-formed circular hollow sections with respect to yield strength, wall thickness and outer diameter*

# APPENDIX B CONSTITUTIVE MODEL OF HSS FOR FE ANALYSIS

## B.1 Material model for HSS with a sharply defined yield point

Depending on the type of analysis, and its requirements in terms of accuracy and allowable strains, the following models of material behaviour may be used for HSS, see Figure B.1:

- linear elastic – perfectly plastic material model without strain hardening,
- linear elastic – perfectly plastic material model with a nominal plateau slope,
- linear elastic – linear hardening plastic material model (quad-linear material model with strain hardening),
- linear elastic – non-linear hardening material model based on coupon-test results using engineering or true stress-strain curve.

The Young's modulus of the steel may be assumed to be according to EN 1993-1-1 clause 5.2.5(1). Where test results are being modelled, the measured Young's modulus may be adopted (EN ISO 6892-1 covers tensile testing methods for steel).

The quad-linear material is given by Equation (B.1) and shown in Figure B.1(c).

$$f(\varepsilon) = \begin{cases} E\varepsilon, & \text{for } \varepsilon \leq \varepsilon_y \\ f_y, & \text{for } \varepsilon_y < \varepsilon \leq \varepsilon_{sh} \\ f_y + E_{sh}(\varepsilon - \varepsilon_{sh}), & \text{for } \varepsilon_{sh} < \varepsilon \leq C_1\varepsilon_u \\ f_{C_1\varepsilon_u} + \dfrac{f_u - f_{C_1\varepsilon_u}}{\varepsilon_u - C_1\varepsilon_u}(\varepsilon - C_1\varepsilon_u), & \text{for } C_1\varepsilon_u < \varepsilon \leq \varepsilon_u \end{cases} \tag{B.1}$$

where:

$f_y$ is the yield stress,

$\varepsilon_y = f_y/E$ is the yield strain,

$f_u$ is the ultimate stress,

$\varepsilon_{sh}$ is the strain hardening strain, which is given by:

$$\varepsilon_{sh} = 0.1\frac{f_y}{f_u} - 0.055, \qquad \text{but } 0.015 \leq \varepsilon_{sh} \leq 0.03 \tag{B.2}$$

$\varepsilon_u$ is the ultimate strain, which is given by:

$$\varepsilon_u = 0.6\left(1 - \frac{f_y}{f_u}\right), \qquad \text{but } 0.06 \leq \varepsilon_u \leq A \tag{B.3}$$

where:

$A$ is the elongation after fracture defined in the relevant material specification,

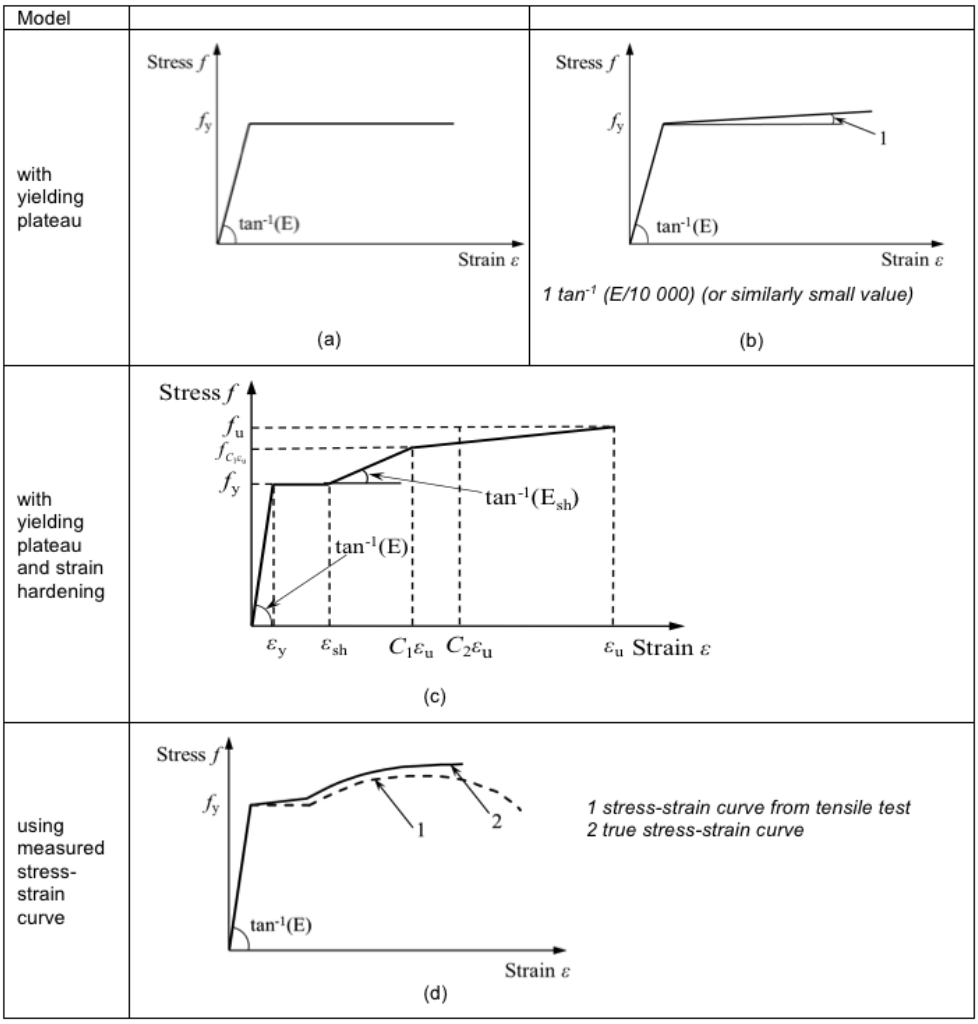


*Figure B.1 Modelling of hot-rolled steels*

$C_1$ is a material coefficient that is given by:

$$C_1 = \frac{\varepsilon_{sh} + 0.25(\varepsilon_u - \varepsilon_{sh})}{\varepsilon_u} \tag{B.4}$$

$E_{sh}$ is the strain hardening modulus that is given by:

$$E_{sh} = \frac{f_u - f_y}{C_2\varepsilon_u - \varepsilon_{sh}} \tag{B.5}$$

where:

$C_2$ is a material coefficient that is given by:

$$C_2 = \frac{\varepsilon_{\mathrm{sh}} + 0.4(\varepsilon_{\mathrm{u}} - \varepsilon_{\mathrm{sh}})}{\varepsilon_{\mathrm{u}}} \tag{B.6}$$

## B.2 Material models for HSS with a rounded stress-strain curve

For HSS materials that exhibit a rounded stress-strain curve with a no well-defined yield point, the two-stage Ramberg-Osgood model given by Equation (B.7) and shown in Figure B.2 may be used.

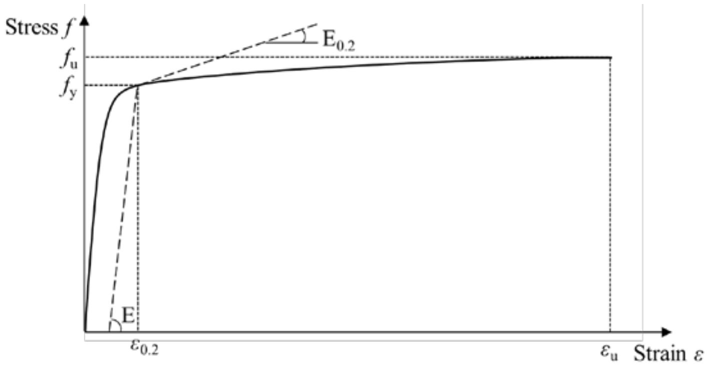


*Figure B.2*
*Two-stage Ramberg-Osgood model*

$$\varepsilon = \begin{cases} \dfrac{f}{E} + 0.002\left(\dfrac{f}{f_{\mathrm{y}}}\right)^{n}, & \text{for } f \le f_{\mathrm{y}} \\ \dfrac{f - f_{\mathrm{y}}}{E_{0.2}} + \left(\varepsilon_{\mathrm{u}} - \varepsilon_{0.2} - \dfrac{f_{\mathrm{u}} - f_{\mathrm{y}}}{E_{0.2}}\right)\left(\dfrac{f - f_{\mathrm{y}}}{f_{\mathrm{u}} - f_{\mathrm{y}}}\right)^{m}, & \text{for } f_{\mathrm{y}} < f \le f_{\mathrm{u}} \end{cases} \tag{B.7}$$

where:

$f$ is the engineering stress,

$\varepsilon$ is the engineering strain,

$E$, $f_y$ and $f_u$ are given in EN 1993-1-1 or the product standard

$n$ is a coefficient that may be taken as 14 for hot-rolled HSS and 8 for cold-formed HSS between S500 and S700 or calculated using measured properties as:

$$n = \frac{\ln(4)}{\ln(f_{\mathrm{y}}/\sigma_{0.05})} \tag{B.8}$$

in which $\sigma_{0.05}$ is the 0.05 % proof stress.

$E_{0.2}$ is the tangent modulus of the stress-strain curve at the yield strength defined as:

$$E_{0.2} = \frac{E}{1 + 0.002n\frac{E}{f_y}} \tag{B.9}$$

$\varepsilon_u$ is the ultimate strain, which is given by:

$$\varepsilon_u = 0.6\left(1 - \frac{f_y}{f_u}\right) \tag{B.10}$$

but $\varepsilon_u \leq A$, where $A$ is the elongation after fracture provided in material specifications.

$m$ is the second strain hardening exponent that may be determined as follows:

$$m = 1 + 3.3\frac{f_y}{f_u} \tag{B.11}$$

For more ductile materials, the approximate relationship:

$$\left(\varepsilon_u - \varepsilon_{0.2} - \frac{f_u - f_y}{E_{0.2}}\right) \approx \varepsilon_u\ , \tag{B.12}$$

can provide a simpler version of Equation (B.7).

As an alternative to the two-stage Ramberg-Osgood model, the above given curves may be represented by multi-linear material models. The quad-linear material model shown in Figure B.3 may be used for cold-formed structures.

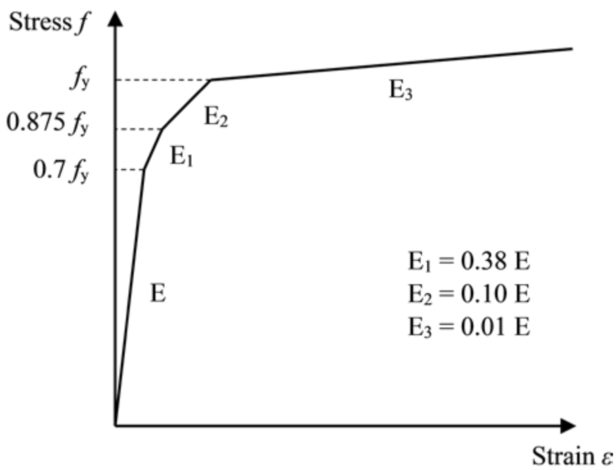


*Figure B.3 Quad-linear material model for cold-formed structures*

# APPENDIX C
# DESIGN EXAMPLES

A series of 4 design examples shows a step-by-step design procedure for HSS components. The examples are as follows:

- Example 1: Square hollow tension member (S460) welded to a connection plate (S355)
- Example 2: Top chord member in a tubular truss, subject to a high compression force and external loads (S690)
- Example 3: Internal I shape column in a multi-storey building where the beam spans and loading about each axis are identical; it is assumed that there are no out-of-balance loads and so the column is subject to axial load only (S460)
- Example 4: Hybrid girder

The examples are in accordance with Eurocode 3. The first three examples show the advantages of using a higher strength steel grade by comparing the weight of the S460 or S690 solution with that of the S355 solution.

Reference to the following documents is made in the worked examples:

**SCI publications and Non-Contradictory Complementary Information (NCCI), accessible from** https://portal.steel-sci.com/documents.html

Steel building design: Design data (P363), SCI and BCSA, 2013

NCCI: Elastic critical moment for lateral torsional buckling, SN003b-EN-EU

NCCI: Verification of columns in simple construction – a simplified interaction criterion (GB), SN048b. (This is a localized resource for the UK)

| | | | | | |
|---|---|---|---|---|---|
| Silwood Park, Ascot, Berks SL5 7QN<br>Telephone: (01344) 636525<br>Fax: (01344) 636570<br>**CALCULATION SHEET** | Job No. OSM680 | | Sheet 1 of 7 | | Rev 0 |
| | Job Title High Strength Steel Design & Execution Guide | | | | |
| | Subject Example 1 – Tension member (S460) | | | | |
| | Client Joint Industry Project | Made by CZK | | Date Nov 2014 | |
| | | Checked by FJM | | Date Oct 2020 | |

References are to EN 1993-1-1:2005 unless otherwise stated.

# 1 Tension member

## 1.1 Scope

Verify the adequacy of the internal steelwork tension member and the connection shown in Figure 1.1.

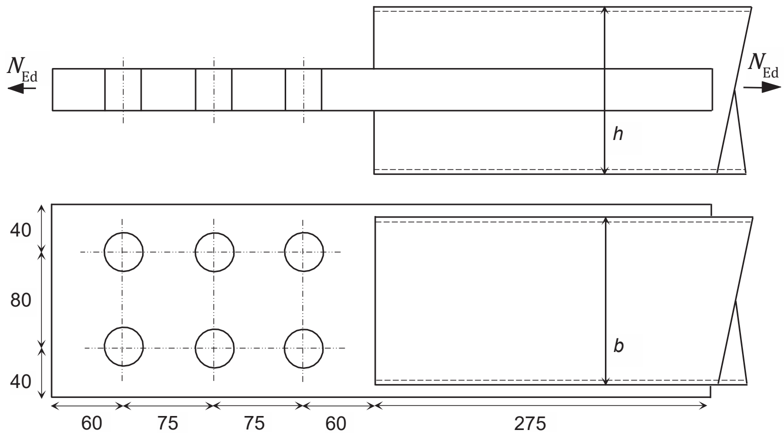


**Figure 1.1 Tension SHS member and conection**

The design aspects covered in this example are:

- Tension resistance of the SHS
- Resistance of the SHS to local shear
- Resistance of the connection plate
- Resistance of the bolts
- Resistance of the fillet welds

## 1.2 Design force for ultimate limit state

Design tension force $N_{Ed} = 1220$ kN

## 1.3 Section properties

Cold formed 120 × 120 × 6.3 SHS in S460 steel

EN 10219-2 Table B.2

Depth of section $h = 120$ mm
Width of section $b = 120$ mm
Wall thickness $t = 6.3$ mm
Cross-sectional area $A = 2730$ mm²

For S460 MH steel
Yield strength ($t \leq 16$ mm) $f_y = R_{eH} = 460\ \text{N/mm}^2$
Ultimate tensile strength ($t \leq 16$ mm) $f_u = R_m = 530\ \text{N/mm}^2$

EN 10219-1
Table B.5

Steel plate for connection 160×545×40 S355:
Thickness of plate $t_p = 40$ mm

For S355 steel
Yield strength (16 mm $\leq t \leq$ 40 mm) $f_y = R_{eH} = 345\ \text{N/mm}^2$
Ultimate tensile strength (3 mm $\leq t \leq$ 100 mm) $f_u = R_m = 470\ \text{N/mm}^2$

EN 10025-2
Table 6

## 1.4 Connection details

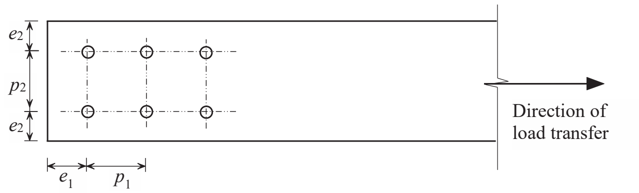


**Figure 1.2 Connection detail dimensions**

M30 Class 8.8 bolts (Class A: Bearing type connection)
Bolt diameter $d = 30$ mm
Hole diameter $d_0 = 32$ mm
Tensile stress area of the bolt $A_s = 561\ \text{mm}^2$

EN 1993-1-8
3.4.1(1)

P363, Page D-381

Yield strength of bolt $f_{yb} = 640\ \text{N/mm}^2$
Ultimate tensile strength of bolt $f_{ub} = 800\ \text{N/mm}^2$

EN 1993-1-8
Table 3.1

**Dimensions**
End distance $e_1 = 60$ mm
Edge distance $e_2 = 40$ mm
Spacing $p_1 = 75$ mm
Spacing $p_2 = 80$ mm
Dimensional limits for a connection that is not exposed to the weather or other corrosive influences

EN 1993-1-8
Table 3.3

$1.2d_0 \leq e_1$; $1.2 \times 32 = 38.4$ mm $< 60$ mm
$1.2d_0 \leq e_2$; 38.4 mm $<$ 40 mm
$2.2d_0 \leq p_1 \leq \min\ (14t_p$ or 200 mm)
$2.4d_0 \leq p_2 \leq \min\ (14t_p$ or 200 mm)
$14t_p = 14 \times 40 = 560$ mm $> 200$ mm
$2.2d_0 = 2.2 \times 32 = 70.4$ mm
$2.4d_0 = 2.4 \times 32 = 76.8$ mm

For $p_1$: 70.4 mm $<$ 75 mm $<$ 200 mm
For $p_2$: 76.8 mm $<$ 80 mm $<$ 200 mm

Therefore, the detailing of the connection is satisfactory.

| Example 1 Tension member (S460) | Sheet 3 of 7 | Rev 0 |
|---|---|---|

## 1.5 Partial factors for resistance

The values for the relevant partial factors recommended in EN 1993-1-1 and EN 1993-1-8 are: 6.1 (1)

(Note that the UK NA to EN 1993-1-1 gives $\gamma_{M2} = 1.1$)

$\gamma_{M0} = 1.0$ (EN 1993-1-1)

$\gamma_{M2} = 1.25$ (EN 1993-1-1)

$\gamma_{M2} = 1.25$ (EN 1993-1-8)

## 1.6 Resistance of the SHS

### 1.6.1 Tension resistance

Verify that:

$$\frac{N_{Ed}}{N_{t,Rd}} < 1.0$$ 6.2.3(1) Eq. 6.5

The design tension resistance of the cross-section is:

$$N_{t,Rd} = \frac{Af_y}{\gamma_{M0}}$$ Eq. 6.6

$$N_{t,Rd} = \frac{2730 \times 460}{1.0} \times 10^{-3} = 1260\ \text{kN}$$

$$\frac{N_{Ed}}{N_{t,Rd}} = \frac{1220}{1260} = 0.97 < 1.0$$

Therefore, the tension resistance of the SHS cross-section is adequate.

### 1.6.2 Resistance to local shear

The resistance to local shear in SHS member is given by:

$$V_{pl,Rd} = \frac{A_v\left(f_y/\sqrt{3}\right)}{\gamma_{M0}}$$ Eq. 6.18

$$A_v = h_p t$$

where $h_p$ is the length of the plate welded to the SHS multiplied by the number of welds.

$$h_p = 4 \times 275 = 1100\ \text{mm}$$

$$A_v = 1100 \times 6.3 = 6930\ \text{mm}^2$$

$$V_{pl,Rd} = \frac{6930 \times (460/\sqrt{3})}{1.0} \times 10^{-3} = 1840\ \text{kN}$$

$$\frac{N_{Ed}}{V_{pl,Rd}} = \frac{1220}{1840} = 0.66 < 1$$

Therefore, the shear resistance of the SHS at the steel plate weld connection is adequate.

## 1.7 Resistance of the steel plate

Four possible types of failure should be considered when determining the tension resistance of the steel plate.

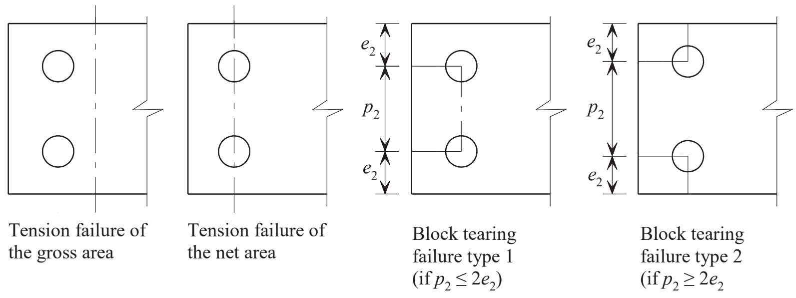


**Figure 1.3 Types of failure of the plate**

## 1.7.1 Tension failure

6.2.3(1) Eq. 6.5

Verify that:

$$\frac{N_{\mathrm{Ed}}}{N_{\mathrm{t,Rd}}} < 1.0$$

For a cross section with holes, $N_{\mathrm{t,Rd}}$ is taken as the smaller of $N_{\mathrm{pl,Rd}}$ and $N_{\mathrm{u,Rd}}$: 6.2.3(2)

Tension resistance of the gross area:

$$N_{\mathrm{pl,Rd}} = \frac{Af_{\mathrm{y}}}{\gamma_{\mathrm{M0}}}$$ Eq. 6.6

Gross area:

$$A = A_{\mathrm{p}} = (2 \times e_2 + p_2) \times t_{\mathrm{p}} = 160 \times 40 = 6400 \text{ mm}^2$$

$$N_{\mathrm{pl,Rd}} = \frac{6400 \times 345}{1.0} \times 10^{-3} = 2210 \text{ kN}$$

Tension resistance of the net area:

$$N_{\mathrm{u,Rd}} = \frac{0.9 \times A_{\mathrm{net}} \times f_{\mathrm{u}}}{\gamma_{\mathrm{M2}}}$$ Eq. 6.7

Net area: 6.2.2.2

$$A_{\mathrm{net}} = A_{\mathrm{p}} - 2d_0 t_{\mathrm{p}} = 6400 - 2 \times 32 \times 40 = 3840 \text{ mm}^2$$

$$N_{\mathrm{u,Rd}} = \frac{0.9 \times 3840 \times 470}{1.25} \times 10^{-3} = 1300 \text{ kN}$$

Since $N_{\mathrm{u,Rd}} < N_{\mathrm{pl,Rd}}$ the tension resistance of the plate is:

$$N_{\mathrm{t,Rd}} = N_{\mathrm{u,Rd}} = 1300 \text{ kN}$$

$$\frac{N_{\mathrm{Ed}}}{N_{\mathrm{t,Rd}}} = \frac{1220}{1300} = 0.94 < 1.0$$ 6.2.3(1)

Therefore, the resistance of the steel plate to tension of the gross and net area is adequate.

## 1.7.2 Block tearing failure

For block tearing, since $p_2 = 2 \times e_2$, failure types 1 and 2 are equally critical. The resistance of the plate to failure types 1 and 2 is calculated in the same way. Figure 1.4 illustrates how the plate resists block tearing failure type 1.

| Example 1 Tension member (S460) | Sheet 5 of 7 | Rev 0 |
|---|---|---|

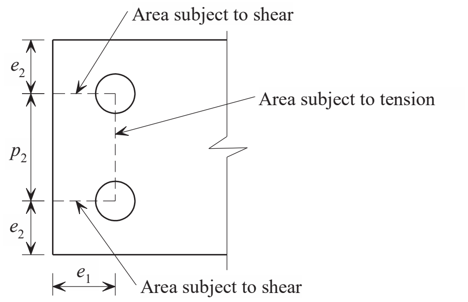


**Figure 1.4 Block tearing failure type 1**

Verify that:

$$\frac{N_{Ed}}{V_{eff,1,Rd}} \leq 1.0$$

For a symmetric bolt group subject to concentric loading, the design block tearing resistance ($V_{eff,1,Rd}$) is determined from:

$$V_{eff,1,Rd} = \frac{f_u A_{nt}}{\gamma_{M2}} + \left(\frac{1}{\sqrt{3}}\right)\frac{f_y A_{nv}}{\gamma_{M0}}$$

EN 1993-1-8 3.10.2(2)

where:

$A_{nt}$ is the net area subject to tension

$A_{nv}$ is the net area subject to shear

$$A_{nt} = (p_2 - d_0)t_p = (80 - 32) \times 40 = 1920 \text{ mm}^2$$

$$A_{nv} = 2\left(e_1 - \frac{d_0}{2}\right)t_p = 2 \times \left(60 - \frac{32}{2}\right) \times 40 = 3520 \text{ mm}^2$$

$$V_{eff,1,Rd} = \left[\frac{470 \times 1920}{1.25} + \left(\frac{1}{\sqrt{3}}\right) \times \frac{345 \times 3520}{1.0}\right] \times 10^{-3} = 1420 \text{ kN}$$

$$\frac{N_{Ed}}{V_{eff,1,Rd}} = \frac{1220}{1420} = 0.86 < 1.0$$

Therefore, the block tearing resistance of the steel plate is adequate.

## 1.8 Resistance of the bolts

References in Section 1.7 are to EN 1993-1-8

$$\frac{N_{Ed}}{F_{Rd,\,joint}} \leq 1.0$$

Where $F_{Rd,\,joint}$ is the resistance of the group of bolts.

### 1.8.1 Design bearing resistance of a single bolt

Table 3.4

$$F_{b,Rd} = \frac{k_1 \alpha_b f_u d t}{\gamma_{M2}}$$

where $\alpha_b$ is the smaller of $\alpha_d, \dfrac{f_{ub}}{f_u}$ and 1.0

For end bolts: $\alpha_d = \dfrac{e_1}{3d_0} = \dfrac{60}{3 \times 32} = 0.625$

For inner bolts: $\alpha_d = \dfrac{p_1}{3d_0} - \dfrac{1}{4} = \dfrac{75}{3 \times 32} - \dfrac{1}{4} = 0.531$

Taking the smallest $\alpha_d$, then $\alpha_d$= 0.531

$$\frac{f_{ub}}{f_u} = \frac{800}{470} = 1.70$$

Therefore, $\alpha_b = \alpha_d = 0.531$

For edge bolts, $k_1$ is the smallest of $\frac{2.8e_2}{d_0} - 1.7, \frac{1.4p_2}{d_0} - 1.7$ and 2.5

$$\frac{2.8e_2}{d_0} - 1.7 = \frac{2.8 \times 40}{32} - 1.7 = 1.80$$

$$\frac{1.4p_2}{d_0} - 1.7 = \frac{1.4 \times 80}{32} - 1.7 = 1.80$$

Therefore for edge bolts, $k_1 = 1.80$

For inner bolts, $k_1$ is the smallest of $\frac{1.4p_2}{d_0} - 1.7$ or 2.5

$$\frac{1.4p_2}{d_0} - 1.7 = \frac{1.4 \times 80}{32} - 1.7 = 1.80$$

Therefore for inner bolts, $k_1 = 1.80$

Taking the smallest $k_1$, then $k_1$= 1.80

$$F_{b,Rd} = \frac{k_1 a_b f_u d t_p}{\gamma_{M2}} = \frac{1.80 \times 0.531 \times 470 \times 30 \times 40}{1.25} \times 10^{-3} = 431 \text{ kN}$$

Therefore, the design bearing resistance of a single bolt is:

$F_{b,Rd} = 431$ kN

### 1.8.2 Design shear resistance of a single bolt

$$F_{v,Rd} = \frac{\alpha_v f_{ub} A}{\gamma_{M2}}$$ Table 3.4

For Class 8.8 bolts, assuming that the shear plane passes through the threaded portion of the bolt. $\alpha_v = 0.6$.

$A = A_s = 561$ mm$^2$ (tensile stress area of the bolt)

Therefore, the design shear resistance of one bolt in single shear is:

$$F_{v,Rd} = \frac{0.6 \times 800 \times 561}{1.25} \times 10^{-3} = 215 \text{ kN}$$

### 1.8.3 Design resistance of the bolt group

For a single bolt: $F_{b,Rd} = 431$ kN $> F_{v,Rd} = 215$ kN

Therefore the resistance of the group of 6 bolts is: 3.7(1)

$F_{Rd,joint} = 6 \times 215 = 1290$ kN

Design shear force applied to the joint is:

$F_{v,Ed} = N_{Ed} = 1220$ kN

$$\frac{N_{Ed}}{F_{Rd,joint}} = \frac{1220}{1290} = 0.95 < 1.0$$

Therefore six M30 grade 8.8 bolts are satisfactory.

| Example 1 Tension member (S460) | Sheet 7 of 7 | Rev 0 |
|---|---|---|

## 1.9 Fillet weld design

*References in Section 1.8 are to EN 1993-1-8*

The simplified method for calculating the design resistance of the fillet weld is used here. However, due to the orientation of the load relative to the longitudinal axis of the weld and the symmetry of the connection, the same result can be obtained by using the Directional method given in clause 4.5.3.2 of EN 1993-1-8.

Consider a fillet weld with a 8 mm leg length (i.e. throat $a = 0.7{\times}8 = 5.6$ mm).

Verify that: (4.5.3.3(1) Eq. 4.2)

$F_{\mathrm{w,Ed}} \le F_{\mathrm{w,Rd}}$

The design weld resistance per unit length,

$F_{\mathrm{w,Rd}} = f_{\mathrm{vw,d}} \times a$ (Eq. 4.3)

where:

$$f_{\mathrm{vw,d}} = \frac{f_{\mathrm{u}}/\sqrt{3}}{\beta_{\mathrm{w}}\gamma_{\mathrm{M2}}}$$ (Eq. 4.4)

For S355 steel,

$\beta_{\mathrm{w}} = 0.9$ (Table 4.1)

$f_{\mathrm{u}}$ relates to the weaker part joined, therefore: (4.5.3.2(6))

$f_{\mathrm{u}} = 470\ \mathrm{N/mm^2}$

Hence

$$f_{\mathrm{vw,d}} = \frac{470/\sqrt{3}}{0.9 \times 1.25} = 241\ \mathrm{N/mm^2}$$

Therefore, the design weld resistance per mm is:

$F_{\mathrm{w,Rd}} = 241 \times 5.6 \times 10^{-3} = 1.35\ \mathrm{kN/mm}$

The effective weld length $l$ is equal to the welded region between the plate and the SHS minus a leg length at either side of a continuous weld:

$l = 4 \times (275 - 2 \times 8) = 1036\ \mathrm{mm}$

The design weld force per mm is:

$$F_{\mathrm{w,Ed}} = \frac{N_{\mathrm{Ed}}}{l} = \frac{1220}{1036} = 1.18\ \mathrm{kN/mm}$$

$$\frac{F_{\mathrm{w,Ed}}}{F_{\mathrm{w,Rd}}} = \frac{1.18}{1.35} = 0.87 < 1.0$$ (4.5.3.3(1))

Therefore the design resistance of the weld with a leg length of 8 mm and throat thickness of 5.6 mm is satisfactory.

## 1.10 Weight saving

The 120 × 120 × 6.3 SHS in **S460** has a weight of 21.4 kg/m.

In addition, the lightest cold formed SHS which can resist the design tension force of 1220 kN with the same connection arrangement and material in **S355** is 110 × 110 × 8.8 SHS. This has a weight of 26.40 kg/m.

The percentage weight saving from using S460 for the SHS section is **18.9%.**

References are to EN 1993-1-1: 2005, unless otherwise stated.

# 2 Top chord in a lattice girder

## 2.1 Scope

The top chord of the lattice girder shown in Figure 2.1 is laterally restrained at locations A, B and C. Verify the adequacy of a hot finished 200 × 120 × 8 RHS in S690 steel for this chord.

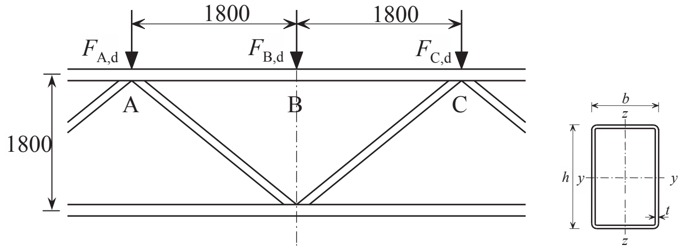


**Figure 2.1 Middle section of truss where compression is maximum for top chord**

The design aspects covered in this example are:

- Cross section classification
- Cross-sectional resistance to combined shear, bending and axial compression
- Buckling resistance for combined bending and axial compression.

The adequacy of the welded joints should be verified using EN 1993-1-8. These verifications are not shown in this example.

## 2.2 Design values of actions at ultimate limit state

Design concentrated force at A $F_{A,d} = 80$ kN

Design concentrated force at B $F_{B,d} = 80$ kN

Design concentrated force at C $F_{C,d} = 80$ kN

## 2.3 Design moments and forces at ultimate limit state

From a full analysis of the whole truss:

The compression force between A and C: $N_{Ed} = 1880$ kN

The design bending moments and shear force are shown in Figure 2.2.

The design bending moment ($M_{Ed}$) and corresponding design shear force ($V_{Ed}$) at B are:

$M_{Ed} = (F_{B,d} \times L) / 8 = (80 \times 3.6) / 8 = 36$ kNm

$V_{Ed} = F/2 = 80/2 = 40$ kN

| Example 2 - Top chord in a lattice girder (S690) | Sheet 2 of 8 | Rev 0 |
|---|---|---|

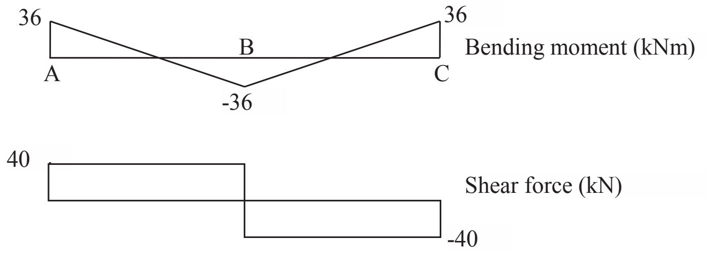


**Figure 2.2 Bending moment and Shear force diagrams**

## 2.4 Section properties

For a hot finished 200 × 120 × 8.0 RHS in S690 steel:

| | | | |
|---|---|---|---|
| Depth of section | $h$ | $= 200$ mm | P363, Page B-23 |
| Width of section | $b$ | $= 120$ mm | |
| Wall thickness | $t$ | $= 8.0$ mm | |
| Radius of gyration y-y | $i_y$ | $= 7.26$ cm | |
| Radius of gyration z-z | $i_z$ | $= 4.85$ cm | |
| Elastic modulus y-y | $W_{el,y}$ | $= 253$ cm$^3$ | |
| Elastic modulus z-z | $W_{el,z}$ | $= 188$ cm$^3$ | |
| Plastic modulus y-y | $W_{pl,y}$ | $= 313$ cm$^3$ | |
| Plastic modulus z-z | $W_{pl,z}$ | $= 218$ cm$^3$ | |
| Area | $A$ | $= 48$ cm$^2$ | |
| Modulus of elasticity | $E$ | $= 210000$ N/mm$^2$ | 3.2.6(1) |

For S690 steel and $t \leq 50$ mm

Yield strength $f_y = R_{eH} = 690\ \mathrm{N/mm^2}$

EN 1993-1-12 Table 1

*Note: Currently EN 10210-1 does not include S690 steels, but the next revision of this standard will include steel strengths up to S960.*

## 2.5 Cross section classification

Table 5.2

$$\varepsilon = \sqrt{\frac{235}{f_y}} = \sqrt{\frac{235}{690}} = 0.584$$

The elastic stress distribution in a rectangular hollow section under combined bending about one axis and compression can be sketched as shown in Figure 2.3. However, the classification of the section may be conservatively determined, according to Table 5.2 given in EN 1993-1-1, assuming the cross-section is subject to pure compression.

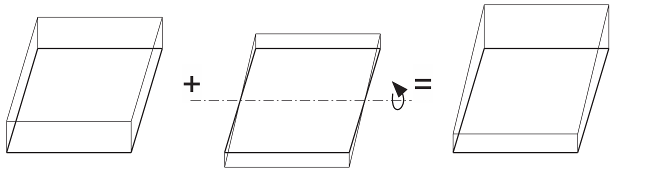


**Figure 2.3 Stress distribution for an RHS subject to combined compression and bending about one axis**

Internal compression part

$c = h - 3t \quad = 200 - 3 \times 8 \quad = 176$ mm

$\frac{c}{t} = \frac{176}{8} \quad = 22$

The limiting value for Class 1 is: Table 5.2

$\frac{c}{t} \leq 33\varepsilon \quad = 33 \times 0.584 \quad = 19.3$

The limiting value for Class 2 is:

$\frac{c}{t} \leq 38\varepsilon \quad = 38 \times 0.584 \quad = 22.2$

$19.3 < 22 < 22.2$

Therefore, the cross section is Class 2 for the case of pure compression, which means the cross-section is also Class 2 for the case of combined compression and bending about the *z-z* axis, and it is at least Class 2 for the case of combined compression and bending about the *y-y* axis.

## 2.6 Partial factors for resistance

The values for the relevant partial factors recommended in EN 1993-1-1 are:

$\gamma_{M0} \quad = 1.0$ 6.1(1)
$\gamma_{M1} \quad = 1.0$

## 2.7 Cross-sectional resistance

### 2.7.1 Bending, shear and axial force

**At cross-section B**

If the design shear force ($V_{Ed}$) is less than 50% of the design plastic shear resistance ($V_{pl,Rd}$), allowance for the shear force on the resistance moment is not required, except where shear buckling reduces the section resistance. 6.2.10(2)

$V_{pl,Rd} = \frac{A_v(f_y/\sqrt{3})}{\gamma_{M0}}$ 6.2.6(2) Eq. 6.18

$A_v$ is the shear area and for an RHS with the load parallel to the depth $h$, which is determined as follows. 6.2.6(3)(f)

$A_v = \frac{Ah}{b+h} \quad = \frac{4800 \times 200}{120 + 200} \quad = 3000 \text{ mm}^2$

| Example 2 - Top chord in a lattice girder (S690) | Sheet 4 of 8 | Rev 0 |
|---|---|---|

$V_{\mathrm{pl,Rd}} = \dfrac{3000 \times (690/\sqrt{3})}{1.0} \times 10^{-3} = 1200\ \mathrm{kN}$

Design shear force $V_{\mathrm{Ed}} = 40\ \mathrm{kN}$

$\dfrac{V_{\mathrm{pl,Rd}}}{2} = \dfrac{1200}{2} = 600\ \mathrm{kN}$

$40\ \mathrm{kN} < 600\ \mathrm{kN}$

Therefore, the criterion is satisfied, subject to verification of shear buckling.

**Verify whether shear buckling reduces the bending resistance**

The shear buckling resistance for webs should be verified according to Section 5 of EN 1993-1-5 if: — 6.2.6(6)

$\dfrac{h_w}{t_w} > \dfrac{72\varepsilon}{\eta}$ — Eq. 6.22

$h_w = h - 3 \times t = 200 - (3 \times 8) = 176\ \mathrm{mm}$

$\eta$ may be obtained from EN 1993-1-5 or conservatively taken as $\eta = 1.0$ — EN 1993-1-5 5.1(2)

$t_{\mathrm{w}} = t = 8\ \mathrm{mm}$

$\dfrac{h_w}{t_w} = \dfrac{176}{8} = 22$

$\dfrac{72\varepsilon}{\eta} = \dfrac{72 \times 0.584}{1.0} = 42$

$22 < 42$

Therefore, the shear buckling resistance of the RHS web does not need to be verified.

The effect of the shear force on the resistance to combined bending and axial force does not need to be allowed for.

**Combined bending and axial force**

For Class 1 and 2 cross sections, verify that: — 6.2.9.1(2) Eq. 6.31

$M_{\mathrm{Ed}} \leq M_{\mathrm{N,Rd}}$

where:

$M_{\mathrm{N,Rd}}$ is the design plastic moment resistance reduced due to the axial force.

For RHS where fastener holes are not to be accounted for, the design moment resistance for the major axis ($M_{\mathrm{N,y,Rd}}$) is determined from: — 6.2.9.1(5)

$M_{\mathrm{N,y,Rd}} = M_{\mathrm{pl,y,Rd}} \dfrac{1-n}{1-0.5\alpha_{\mathrm{w}}}$ but $M_{\mathrm{N,y,Rd}} \leq M_{\mathrm{pl,y,Rd}}$ — Eq. 6.39

The design plastic moment resistance of the cross section about the major axis ($M_{\mathrm{pl,y,Rd}}$) for Class 1 and 2 cross sections is determined from: — 6.2.5(2)

$M_{\mathrm{pl,y,Rd}} = \dfrac{W_{\mathrm{pl,y}} f_{\mathrm{y}}}{\gamma_{\mathrm{M0}}} = \dfrac{313 \times 10^3 \times 690}{1.0} \times 10^{-6} = 216\ \mathrm{kNm}$ — Eq. 6.13

$n = \dfrac{N_{\mathrm{Ed}}}{N_{\mathrm{pl,Rd}}}$ — 6.2.9.1(5)

$N_{\mathrm{pl,Rd}}$ is the design plastic resistance of the gross cross section:

$$N_{\mathrm{pl,Rd}} = \frac{Af_{\mathrm{y}}}{\gamma_{\mathrm{M0}}} = \frac{4800 \times 690}{1.0} \times 10^{-3} = 3310 \text{ kN}$$ Eq. 6.6

$$n = \frac{1880}{3312} = 0.568$$

$$\alpha_{\mathrm{w}} = \frac{A - 2bt}{A} \text{ but } a_{\mathrm{w}} \leq 0.5$$ 6.2.9.1(5)

$$\alpha_{\mathrm{w}} = \frac{4800 - (2 \times 120 \times 8)}{4800} = 0.6 > 0.5$$

Therefore, $\alpha_{\mathrm{w}} = 0.5$

$$M_{\mathrm{N,Rd}} = M_{\mathrm{pl,y,Rd}} \frac{1-n}{1-0.5\alpha_{\mathrm{w}}} = 216 \times \frac{1-0.568}{1-(0.5 \times 0.5)} = 124 \text{ kNm}$$ Eq. 6.39

$M_{\mathrm{Ed}} = 36 \text{ kNm} < 124 \text{ kNm}$

$$\frac{M_{\mathrm{Ed}}}{M_{\mathrm{N,Rd}}} = \frac{36}{124} = 0.29 < 1.0$$

Therefore the resistance of the cross section at B to combined bending, shear and axial force is adequate.

Since the forces at cross section A and C are the same as at B there is no further need for checking them, implying adequate cross-section resistance for the whole member.

## 2.8 Buckling resistance of member

### 2.8.1 Combined bending and axial compression

Cross-sectional resistance at the ends of the member (A and C) to combined bending, shear and axial force is satisfactory (see Section 2.7.1 above). 6.3.3(2)

For combined bending and axial compression of the member, the following criteria should be satisfied.

$$\frac{N_{\mathrm{Ed}}}{\chi_{\mathrm{y}} N_{\mathrm{Rk}}/\gamma_{\mathrm{M1}}} + k_{\mathrm{yy}} \frac{M_{\mathrm{y,Ed}} + \Delta M_{\mathrm{y,Ed}}}{\chi_{\mathrm{LT}} M_{\mathrm{y,Rk}}/\gamma_{\mathrm{M1}}} + k_{\mathrm{yz}} \frac{M_{\mathrm{z,Ed}} + \Delta M_{\mathrm{z,Ed}}}{M_{\mathrm{z,Rk}}/\gamma_{\mathrm{M1}}} \leq 1.0$$ Eq. 6.61

$$\frac{N_{\mathrm{Ed}}}{\chi_{\mathrm{z}} N_{\mathrm{Rk}}/\gamma_{\mathrm{M1}}} + k_{\mathrm{zy}} \frac{M_{\mathrm{y,Ed}} + \Delta M_{\mathrm{y,Ed}}}{\chi_{\mathrm{LT}} M_{\mathrm{y,Rk}}/\gamma_{\mathrm{M1}}} + k_{\mathrm{zz}} \frac{M_{\mathrm{z,Ed}} + \Delta M_{\mathrm{z,Ed}}}{M_{\mathrm{z,Rk}}/\gamma_{\mathrm{M1}}} \leq 1.0$$ Eq. 6.62

For Class 1, 2 and 3 cross sections $\Delta M_{\mathrm{y.Ed}}$ and $\Delta M_{\mathrm{z.Ed}}$ are zero.

$N_{\mathrm{Rk}} = Af_{\mathrm{y}}$ (for Class 1 and 2 cross sections) Table 6.7

$N_{\mathrm{Rk}} = 4800 \times 690 \times 10^{-3} = 3310 \text{ kN}$

$M_{\mathrm{y,Rk}} = W_{\mathrm{pl,y}} f_{\mathrm{y}}$ (for Class 1 and 2 cross sections) Table 6.7

$M_{\mathrm{y,Rk}} = 313 \times 10^{3} \times 690 \times 10^{-6} = 216 \text{ kNm}$

*Note: There is no need to calculate $M_{z,Rk}$ because there is no bending moment applied about the z-z axis*

| Example 2 - Top chord in a lattice girder (S690) | Sheet 6 of 8 | Rev 0 |
|---|---|---|

**Calculation of reduction factors for buckling $\chi_y$, $\chi_z$ and $\chi_{LT}$**

For flexural buckling $\chi_y = \chi$ — 6.3.1.2(1) Eq. 6.49

$$\chi = \frac{1}{\Phi + \sqrt{\Phi^2 - \bar{\lambda}^2}} \text{ but } \chi \leq 1.0$$

where: — 6.3.1.3(1) Eq. 6.50

$$\bar{\lambda} = \sqrt{\frac{Af_y}{N_{cr}}} = \left(\frac{L_{cr}}{i}\right)\left(\frac{1}{\lambda_1}\right)$$

(For Class 1, 2 and 3 cross sections).

The buckling lengths may be taken as the distance between restraints, therefore:

For buckling about *y*-*y* axis $L_{y,cr} = 3600$ mm

For buckling about the *z*-*z* axis $L_{z,cr} = 1800$ mm

$\lambda_1 = 93.9\varepsilon = 93.9 \times 0.584 = 54.8$

**Buckling about the *y*-*y* axis:**

$$\bar{\lambda}_y = \left(\frac{3600}{72.6}\right)\left(\frac{1}{54.8}\right) = 0.905$$

$$\Phi_y = 0.5\left[1 + \alpha\left(\bar{\lambda}_y - 0.2\right) + \bar{\lambda}_y^{\,2}\right]$$ — 6.3.1.2(1)

For hot finished RHS in S690 steel use buckling curve **a₀** — EN 1993-1-12, Table 6.2

For curve **a₀** the imperfection factor is $\alpha = 0.13$ — Table 6.1

$$\Phi_y = 0.5 \times [1 + 0.13 \times (0.905 - 0.2) + 0.905^2] = 0.955$$ — 6.3.1.2(1)

$$\chi_y = \frac{1}{\Phi + \sqrt{\Phi^2 - \bar{\lambda}^2}} = \frac{1}{0.955 + \sqrt{(0.955^2 - 0.905^2)}} = 0.794$$ — Eq. 6.49

$0.794 < 1.0$

Therefore, $\chi_y = 0.794$

**Buckling about the *z*-*z* axis:** — 6.3.1.3(1) Eq. 6.50

$$\bar{\lambda}_z = \left(\frac{1800}{48.5}\right)\left(\frac{1}{54.8}\right) = 0.677$$

$$\Phi_z = 0.5 \times [1 + 0.13 \times (0.677 - 0.2) + 0.677^2] = 0.760$$ — 6.3.1.2(1)

$$\chi_z = \frac{1}{\Phi_z + \sqrt{\Phi_z^{\,2} - \bar{\lambda}_z^{\,2}}} = \frac{1}{0.760 + \sqrt{(0.760^2 - 0.677^2)}} = 0.905$$ — Eq. 6.49

$0.905 < 1.0$

Therefore, $\chi_z = 0.905$

**Lateral torsional buckling**

The RHS is not susceptible to lateral torsional buckling — 6.3.2.1(2)

Therefore, $\chi_{LT} = 1.0$

**Calculation of interaction factors $k_{yy}$, $k_{yz}$, $k_{zy}$ and $k_{zz}$** — 6.3.3(5)

For sections not susceptible to torsional deformation, use Table B.1 to determine the interaction factors $k_{yy}$, $k_{yz}$ , $k_{zy}$ and $k_{zz}$. For this example, the design bending moment about the minor axis is zero, therefore values for $k_{yz}$ and $k_{zz}$ are not required.

For class 1 and 2 RHS cross sections:

$$k_{yy} = C_{my}\left[1 + \left(\bar{\lambda}_y - 0.2\right)\left(\frac{N_{Ed}}{\chi_y N_{Rk}/\gamma_{M1}}\right)\right] \leq C_{my}\left[1 + 0.8\left(\frac{N_{Ed}}{\chi_y N_{Rk}/\gamma_{M1}}\right)\right]$$ — Table B.1

$$k_{zy} = 0.6k_{yy}$$

For bending about the *y-y* axis consider points braced in the *z-z* direction. — Table B.3

Points A and C are braced in the *z-z* direction. Therefore, the following bending moment diagram between A and C needs to be considered when calculating $C_{my}$.

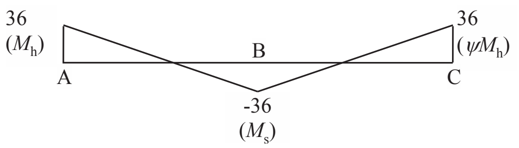


**Figure 2.4 Moment diagram for top chord**

From the above bending moment diagram:

$\psi = 1.0$ — Table B.3

$$\alpha_s = \frac{M_s}{M_h} = \frac{-36}{36} = -1.0$$ — Table B.3

Therefore,

$C_{my} = -0.8\alpha_s \geq 0.4$ — Table B.3

$C_{my} = -0.8 \times -1.0 = 0.8 > 0.4$

Hence,

$C_{my} = 0.8$

$$k_{yy} = 0.8\left[1 + (0.90 - 0.2)\left(\frac{1880}{0.80 \times 3312/1.0}\right)\right] \leq 0.8\left[1 + 0.8\left(\frac{1880}{0.80 \times 3312/1.0}\right)\right]$$ — Table B.1

$= 1.20 \leq 1.25$

Therefore: $k_{yy}$= 1.20

$k_{zy} = 0.6k_{yy} = 0.6 \times 1.20 = 0.720$ — Table B.1

Substitute:

| Example 2 - Top chord in a lattice girder (S690) | Sheet 8 of 8 | Rev 0 |
|---|---|---|

$$\frac{N_{\mathrm{Ed}}}{\chi_{\mathrm{y}} N_{\mathrm{Rk}}/\gamma_{\mathrm{M1}}} + k_{\mathrm{yy}} \frac{M_{\mathrm{y.Ed}} + \Delta M_{\mathrm{y.Ed}}}{\chi_{\mathrm{LT}} M_{\mathrm{y.Rk}}/\gamma_{\mathrm{M1}}} + k_{\mathrm{yz}} \frac{M_{\mathrm{z.Ed}} + \Delta M_{\mathrm{z.Ed}}}{M_{\mathrm{z.Rk}}/\gamma_{\mathrm{M1}}} \leq 1.0$$

Eq. 6.61

$$\left(\frac{1880}{0.794 \times 3310/1.0}\right) + 1.20 \times \left(\frac{36 + 0}{1.0 \times 216/1.0}\right) + 0 \quad = 0.915 < 1.0$$

$$\frac{N_{\mathrm{Ed}}}{\chi_{\mathrm{z}} N_{\mathrm{Rk}}/\gamma_{\mathrm{M1}}} + k_{\mathrm{zy}} \frac{M_{\mathrm{y.Ed}} + \Delta M_{\mathrm{y.Ed}}}{\chi_{\mathrm{LT}} M_{\mathrm{y.Rk}}/\gamma_{\mathrm{M1}}} + k_{\mathrm{zz}} \frac{M_{\mathrm{z.Ed}} + \Delta M_{\mathrm{z.Ed}}}{M_{\mathrm{z.Rk}}/\gamma_{\mathrm{M1}}} \leq 1.0$$

Eq. 6.62

$$\left(\frac{1880}{0.905 \times 3310/1.0}\right) + 0.720 \times \left(\frac{36 + 0}{1.0 \times 216/1.0}\right) + 0 \quad = 0.748 < 1.0$$

As both criteria are satisfied, the resistance of the member under combined compression and bending about the *y-y* axis is adequate.

## 2.9 Weight saving

The 200 × 120 × 8 RHS in S690 has a weight of 37.6 kg/m and is the lightest hollow section in S690 that can support this design combination of actions.

The lightest hollow section which can support the same design combination of actions in S420 is 250 × 150 × 8 RHS. This has a weight of 47.7 kg/m. The percentage weight saving from using S690 is 21%.

The lightest hollow section which can support the same design combination of actions in S355 is 250 × 150 × 10 RHS. This has a weight of 58.8 kg/m. The percentage weight saving from using S690 is 36%.

*References are to EN 1993-1-1: 2005, unless otherwise stated.*

# 3 Column in multi-storey building

## 3.1 Scope

This example shows the design of an internal column. It is assumed that the stiffness ($I/L$) of columns above and below are within a ratio of 1.5; applied moments are therefore distributed equally at floor levels. In all cases, the beam end reactions are assumed to be applied 100 mm from the face of the column (web or flange) in accordance with UK NCCI SN048b.

In this example, the beam spans and loading about each axis are identical; there are no out-of-balance loads – the column is subject to axial load only. The column buckling length is 4 m, and is shown, with design values of actions, in Figure 3.1 and Figure 3.2.

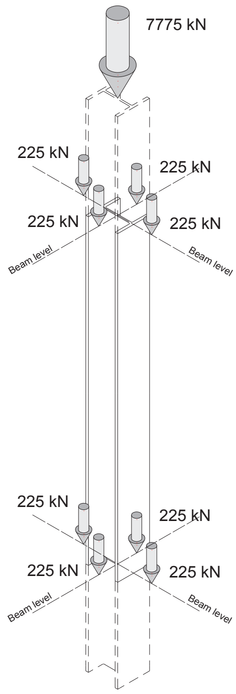


**Figure 3.1 Internal column**

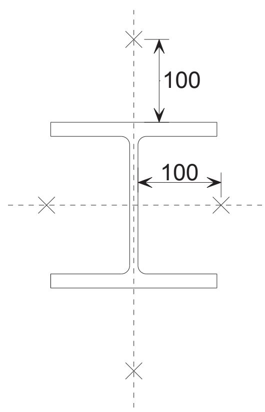


**Figure 3.2 Beam reactions**

The axial design force is equal to: $N_{Ed} = 7775 + 4 \times 225 = 8680$ kN

## 3.2 Section properties

Hot finished 356 × 368 × 177 UC in S460 steel

| Example 3 – Column in multi-storey building (S460) | Sheet 2 of 3 | Rev 0 |
|---|---|---|

| | | | | |
|---|---|---|---|---|
| Depth of section | $h$ | = | 368.2 mm | P363, Page B-8 |
| Width of section | $b$ | = | 372.6 mm | |
| Flange thickness | $t_f$ | = | 23.8 mm | |
| Web thickness | $t_w$ | = | 14.4 mm | |
| Cross-sectional area | $A$ | = | 22600 mm$^2$ | |
| Second moment of area (z-z) | $I_z$ | = | $2.05 \times 10^8$ mm$^4$ | |
| Radius of gyration (z-z) | $i_z$ | = | 95.4 mm | |
| Root radius | $r$ | = | 15.2 mm | |

## 3.3 Classification of cross-section

Outstand flange: flange under compression

For S460M steel — EN 10025-4 Table 5

Yield strength ($16 < t \leq 40$ mm) $f_y = R_{eH} = 440$ N/mm$^2$

Therefore, the coefficient $\varepsilon$ is:

$$\varepsilon = \sqrt{\frac{235}{f_y}} = \sqrt{\frac{235}{440}} = 0.731$$

$$c = \frac{(b - t_w - 2r)}{2} = \frac{(372.6 - 14.4 - 2 \times 15.2)}{2} = 164 \text{ mm}$$

$$\frac{c}{t_f} = \frac{163.9}{23.8} = 6.89$$

The limiting value for Class 2 is — 5.5.2, Table 5.2

$$\frac{c}{t_f} \leq 10\varepsilon = 10 \times 0.731 = 7.31$$

$6.89 < 7.31$

Therefore, the outstand flange in compression is Class 2

Internal compression part: web under compression

For S460 steel

Yield strength ($t \leq 16$ mm) $f_y = 460$ N/mm$^2$

However, since the cross-section and member resistances are calculated using the yield strength of the flanges (which is slightly lower than the yield strength of the web), the classification of the web should also be based on the yield strength of the flange, $f_y = 440$ N/mm$^2$. Therefore, for the web $\varepsilon = 0.731$

$$c = h - 2t_f - 2r = 368.2 - 2 \times 23.8 - 2 \times 15.2 = 290.2 \text{ mm}$$

$$\frac{c}{t_w} = \frac{290.2}{14.4} = 20.2$$

The limiting value for Class 1 is

$$\frac{c}{t_w} \leq 33\varepsilon = 33 \times 0.731 = 24.1$$

$20.2 < 24.1$

Therefore, the web subject to compression is Class 1

Overall cross-section classification is therefore **Class 2**

## 3.4 Compression resistance of cross section

For Class 1, 2 or 3 cross sections the design resistance is given by: 6.2.4 Eq. 6.10

$$N_{\mathrm{c,Rd}} = \frac{Af_{\mathrm{y}}}{\gamma_{\mathrm{M0}}} = \frac{22600 \times 440}{1.0} \times 10^{-3} = 9940\ \mathrm{kN} > N_{\mathrm{Ed}} = 8680\ \mathrm{kN}$$

## 3.5 Design member buckling resistance (minor axis)

The design buckling resistance for members with Class 1, 2 and 3 cross-sections is taken as:

$$N_{\mathrm{b,z,Rd}} = \frac{\chi_{\mathrm{z}} A f_{\mathrm{y}}}{\gamma_{\mathrm{M1}}}$$ 6.3.1.1 Eq. 6.47

Where $\gamma_{\mathrm{M1}} = 1.0$ 6.1(1)

The non-dimensional slenderness is calculated as follows:

For Class 1, 2 or 3 cross-sections: 6.3.1.3 Eq. 6.50

$$\overline{\lambda_{\mathrm{z}}} = \frac{L_{\mathrm{cr,z}}}{93.9\varepsilon i_{\mathrm{z}}} = \frac{4000}{93.9 \times 0.731 \times 95.4} = 0.611$$

For a hot rolled (hot finished) UC with *h*/*b* ≤ 1.2 and for steel grade S460, buckling curve **a** is chosen. 6.3.1.2 Table 6.2

For this curve, the imperfection factor is equal to 0.21. 6.3.1.2 Table 6.1

Therefore the reduction factor $\chi_{\mathrm{z}}$ can be calculated as follows:

$$\Phi_{\mathrm{z}} = 0.5\left[1 + \alpha\left(\overline{\lambda}_{\mathrm{z}} - 0.2\right) + \overline{\lambda_{\mathrm{z}}}^2\right] = 0.5[1 + 0.21(0.611 - 0.2) + 0.611^2]$$ 6.3.1.2 Eq. 6.49

$$= 0.730$$

$$\chi_{\mathrm{z}} = \frac{1}{\Phi_{\mathrm{z}} + \sqrt{{\Phi_{\mathrm{z}}}^2 - \overline{\lambda}_{z}^{\,2}}} = \frac{1}{0.730 + \sqrt{0.730^2 - 0.611^2}} = 0.885$$

Hence,

$$N_{\mathrm{b,z,Rd}} = \frac{0.885 \times 22600 \times 440}{1.0} \times 10^{-3} = 8810\ \mathrm{kN} > N_{\mathrm{Ed}} = 8680\ \mathrm{kN}$$

Therefore the buckling resistance of the chosen section is adequate.

## 3.6 Weight saving from using S460

The 356 × 368 × 177 UC in S460 has a weight of 177 kg/m.

In S355, the most appropriate sections would be a 356 × 406 × 235 UC which weighs 235.1 kg/m. The percentage weight saving from using S460 for the UC section is 25%.

| SCI<br>Silwood Park, Ascot, Berks SL5 7QN<br>Telephone: (01344) 636525<br>Fax: (01344) 636570<br>CALCULATION SHEET | Job No. OSM680 | | Sheet 1 of 8 | Rev 0 |
|---|---|---|---|---|
| | Job Title High Strength Steel Design & Execution Guide | | | |
| | Subject Example 4 – Hybrid girder | | | |
| | Client | Made by DGB | Date Nov 2019 | |
| | | Checked by FJM | Date Oct 2020 | |

References are to EN 1993-1-1:2005 unless otherwise stated.

# 4 Hybrid girder

## 4.1 Scope

Verify the hybrid girder shown in Figure 4.1. The flanges are S460 and the web S355. The beam span is 8 m, and is subjected to a factored uniformly distributed load (UDL) of 326 kN/m. The beam is simply supported at both ends where the web is reinforced with transverse stiffeners, which create non-rigid end posts. The web has no longitudinal or intermediate transverse stiffener.

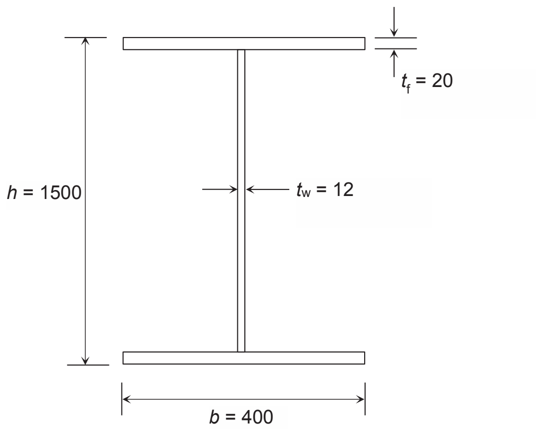


**Figure 4.1 Cross section dimensions of a hybrid girder**

The design aspects covered in this example are:

- Cross-section bending resistance
- Resistance to lateral-torsional buckling
- Shear resistance
- Flange induced buckling

## 4.2 Design moment and shear for ultimate limit state

Design moment $M_{\mathrm{Ed}} = \dfrac{wL^2}{8} = \dfrac{326 \times 8^2}{8} = 2610\ \mathrm{kNm}$

Design shear $V_{\mathrm{Ed}} = \dfrac{wL}{2} = \dfrac{326 \times 8}{2} = 1300\ \mathrm{kN}$

## 4.3 Partial factors for resistance

The values for the relevant partial factors recommended in EN 1993-1-1 are: 6.1(1)

$\gamma_{\mathrm{M0}} = 1.0$
$\gamma_{\mathrm{M1}} = 1.0$

## 4.4 Material strength and classification

The flange is greater than 16 mm, therefore $f_y = 440 \text{ N/mm}^2$ — EN 10025-2

$$\varepsilon = \sqrt{\frac{235}{440}} = 0.731$$

Length of flange outstand is:

$$\frac{400 - 12}{2} = 194 \text{ mm}$$

Therefore:

$$\frac{c}{t} = \frac{194}{20} = 9.70$$

Class 2 limit is $10\varepsilon$ = 7.3

Class 3 limit is $14\varepsilon$ = 10.2, so the flange is Class 3

For a hybrid girder, the classification of the web is based on the yield strength of the compression flange. Therefore:

$$\varepsilon = 0.731$$

and

$$\frac{c}{t} = \frac{1460}{12} = 122$$

Class 3 limit is $124\varepsilon$ = 90.6, so the web, and therefore the section, is Class 4.

## 4.5 Effective cross-sectional area due to shear lag and plate buckling

### 4.5.1 Shear lag

$$b_0 = \frac{400}{2} = 200$$

Note that $b_0$ is half the flange width — EN 1993-1-5 3.1(1)

$$\frac{L_e}{b_0} = \frac{8000}{200} = 40 < 50$$

Therefore the shear lag in the flange cannot be neglected.

From Table 3.1 of EN 1993-1-5, because there are no longitudinal stiffeners, $A_{sl}$ =0 and therefore $\alpha_0 = 1.0$

$$\kappa = \frac{\alpha_0 b_0}{L_e} = \frac{1.0 \times 200}{8000} = 0.025$$

EN 1993-1-5 Table 3.1

Since $0.02 < \kappa \leq 0.7$, and the beam is subject to sagging bending:

$$\beta = \beta_1 = \frac{1}{1 + 6.4 \times \kappa^2} = \frac{1}{1 + 6.4 \times 0.025^2} = 0.996$$

### 4.5.2 Stress distribution in the flange due to shear lag

The stress distribution in one of the flanges is shown in Figure 4.2 for $\beta > 0.2$. — EN 1993-1-5 3.2.2

| Example 4 - Hybrid girder | Sheet 3 of 8 | Rev 0 |
|---|---|---|

The ratio of stresses is needed later, so the calculation is best expressed as

EN 1993-1-5 Figure 3.3

$$\frac{\sigma_1}{\sigma_2} = 1.25(\beta - 0.2) = 0.995$$

The value of 0.995 indicates that there is hardly any influence from shear lag in this example.

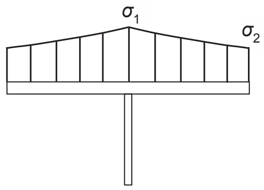


**Figure 4.2 Stress distribution across flange outstand**

### 4.5.3 Flange plate buckling

EN 1993-1-5 4.4

From Table 4.2 of EN 1993-1-5 for outstand elements,

$$\psi = \frac{\sigma_1}{\sigma_2} = 0.995$$

Therefore, the buckling factor is

$$k_\sigma = \frac{0.578}{\psi + 0.34} = \frac{0.578}{0.995 + 0.34} = 0.433$$

Then $\bar{\lambda}_\mathrm{p}$ is given by clause 4.4(2) as:

$$\bar{\lambda}_\mathrm{p} = \frac{\bar{b}/t}{28.4\varepsilon\sqrt{k_\sigma}} = \frac{194/20}{28.4 \times 0.731 \times \sqrt{0.433}} = 0.711$$

Note that $\bar{b} = c$ for outstand flanges. $c = (400 - 12)/2 = 194$ mm

Because $\bar{\lambda}_\mathrm{p} < 0.748$, $\rho = 1.0$

The effective area due to plate buckling is

$$A_\mathrm{c.eff} = \rho A_c = 1.0 \times 400 \times 200 = 8000 \text{ mm}^2$$

### 4.5.4 Combined effects of share lag and buckling in the flange

EN 1993-1-5 3.3(1), Note 3

The effective area of the compression flange considering both shear lag and plate buckling is given by:

$$A_\mathrm{eff} = A_\mathrm{c.eff}\beta^\kappa = 8000 \times 0.996^{0.025} = 7999 \text{ mm}^2$$

There is therefore virtually no reduction due to the effect of shear lag and plate buckling.

### 4.5.5 Web plate buckling

Because (in this case) there is no reduction of the area of the compression flange due to the combined effects of shear lag and plate buckling, and no reduction of the area of the tension flange due to shear lag, the gross cross section is symmetrical. The neutral axis of the gross section is at mid-height of the web.

The length of the compression part of the web $b_\mathrm{c}$ is $1460/2 = 730$ mm.

Because the gross section is symmetrical, $\psi = -1$, and then $k_\sigma = 23.9$.

EN 1993-1-5 Table 4.1

For hybrid girders, the yield strength of the flange must be used when determining the effective area of the web. Because $f_\mathrm{yf} = 400\ \mathrm{N/mm^2}$, $\varepsilon = 0.731$. Then

EN 1993-1-5 4.3(6)

$$\bar{\lambda}_\mathrm{p} = \frac{\bar{b}/t}{28.4\varepsilon\sqrt{k_\sigma}} = \frac{1460/12}{28.4 \times 0.731 \times \sqrt{23.9}} = 1.20$$

For an internal compression element:

EN 1993-1-5 4.4(2) Eq. 4.2

$$0.5 + \sqrt{0.085 - 0.055\psi} = 0.5 + \sqrt{0.085 - 0.055 \times (-1)} = 0.874$$

Because $\bar{\lambda}_\mathrm{p} = 1.20 > 0.874$, then

$$\rho = \frac{\bar{\lambda}_\mathrm{p} - 0.055(3+\psi)}{\bar{\lambda}_\mathrm{p}^2} = \frac{1.2 - 0.55(3 + (-1))}{1.20} = 0.757$$

The effective depth of the compression part of the web is therefore

$$\rho \times b_\mathrm{c} = 0.757 \times \frac{1460}{2} = 553 \text{ mm}$$

The stable length adjacent the compression flange is

EN 1993-1-5 Table 4.1

$$b_\mathrm{e1} = 0.4 b_\mathrm{eff} = 0.4 \times 553 = 221 \text{ mm}$$

The ineffective length = 730 – 553 = 177 mm

### 4.6 Stress distribution

According to clause 4.3(5) of EN 1993-1-5 the stress in the flange is considered at the mid-plane of the flange.

By postulating a position of the neutral axis, the stresses at locations through the cross section can be computed. The stress in the web is limited to $f_\mathrm{yw}$, which is 355 N/mm$^2$.

Knowing the stresses and cross sectional dimensions, the tension force and compression force can be calculated, compared, and the position of the neutral axis adjusted until equilibrium is achieved.

In this case, the final solution is shown in Figure 4.3. Summing the product of the stress and area, the following forces are obtained:

| Example 4 - Hybrid girder | Sheet 5 of 8 | Rev 0 |
|---|---|---|

| | | |
|---|---|---|
| Compression flange | 440×400×20 | = 3520000 N |
| Web ‘plateau’ | 139×355×12 | = 592140 N |
| Web above ineffective portion | 0.5×(308+355)×82×12 | = 326196 N |
| Web below ineffective portion | 0.5×207×363×12 | = 450846 N |
| | Summation | = 4890 kN |
| Tension flange | 405×440×20 | = 3240000 N |
| Web ‘plateau’ | 355×77×12 | = 328020 N |
| Web | 0.5×622×355×12 | = 1324860 N |
| | Summation | = 4890 kN |

Equilibrium of force has been achieved.

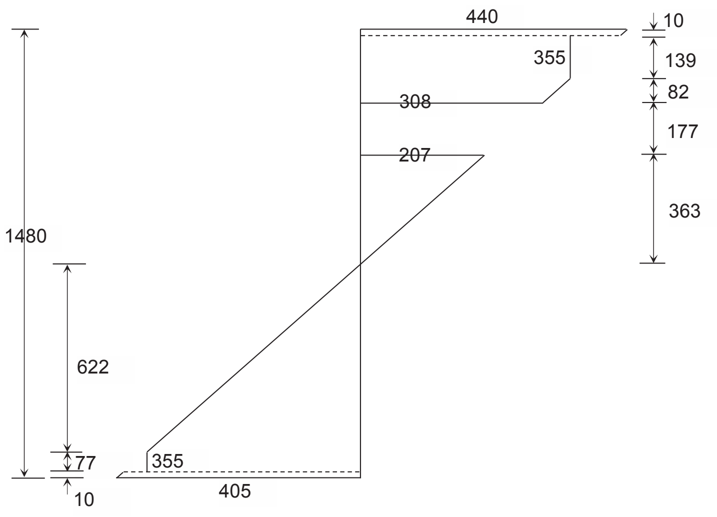


**Figure 4.3 Final stress block**

## 4.7 Moment resistance

Once equilibrium has been found, the moment resistance is simply the summation of the force in each element, multiplied by the lever arm.

$3520000 \times 771 = 2.71 \times 10^9$ Nmm

$592140 \times 692 = 409 \times 10^6$ Nmm

$326196 \times 581 = 189.5 \times 10^6$ Nmm

$450846 \times 2/3 \times 363 = 109 \times 10^6$ Nmm

$3240000 \times 709 = 2.30 \times 10^9$ Nmm

$328020 \times 661 = 217 \times 10^6$ Nmm

$1324860 \times 2/3 \times 622 = 549 \times 10^6$ Nmm

The moment resistance is:

$M_{c,Rd} = 6490$ kNm

$$\frac{M_{Ed}}{M_{c,Rd}} = \frac{2610}{6490} = 0.40 < 1.0$$ 6.2.5(1) Eq. 6.12

Therefore, the bending moment resistance of the hybrid cross-section is adequate.

## 4.8 Lateral torsional buckling

For LTB resistance, the following section properties are needed first: UK NCCI SN003

$$I_z = 2 \times \frac{20 \times 400^3}{12} + \frac{1460 \times 12^3}{12} = 214 \times 10^6 \text{mm}^4$$

$$I_w = \frac{I_z \times h_0^2}{4} = \frac{214 \times 10^6 \times (1500 - 20)^2}{4} = 1.17 \times 10^{14} \text{mm}^6$$

$$I_t = \frac{2}{3} b_t t_f^3 + \frac{1}{3} h_w t_w^3 = \frac{2}{3} \times 400 \times 20^3 + \frac{1}{3} \times 1460 \times 12^3 = 2.97 \times 10^6 \text{mm}^4$$

As the load is uniformly distributed (then $C_1 = 1.13$) and assuming the load is not destabilizing, the elastic critical moment can be computed as:

$$M_{cr} = C_1 \frac{\pi^2 E I_z}{L^2} \sqrt{\frac{I_w}{I_z} + \frac{L^2 G I_t}{\pi^2 E I_z}} = 5970 \text{ kNm}$$

The non-dimensional slenderness $\bar{\lambda}_{LT}$ is determined as:

$$\bar{\lambda}_{LT} = \sqrt{\frac{6490 \text{ kNm}}{5970 \text{ kNm}}} = 1.04$$

The reduction factor for lateral torsional buckling of the hybrid beam can be computed as: 6.3.2.2

$$\chi_{LT} = \frac{1}{\Phi_{LT} + \sqrt{\Phi_{LT}^2 - \bar{\lambda}_{LT}^2}} \text{ but } \chi_{LT} \leq 1.0$$

where

$$\Phi_{LT} = 0.5\left[1 + \alpha_{LT}\left(\bar{\lambda}_{LT} - 0.2\right) + \bar{\lambda}_{LT}^2\right]$$

$h/b = 1500 / 400 = 3.75$, so buckling curve **d** is used and $\alpha_{LT} = 0.76$ Tables 6.3 and 6.4

Working through the above expressions, the reduction factor $\chi_{LT} = 0.447$ and the buckling resistance of the beam is $M_{b,Rd} = 0.447 \times 6490 = 2900$ kNm.

$$\frac{M_{Ed}}{M_{b,Rd}} = \frac{2610}{2900} = 0.90 < 1.0$$ 6.3.2.1(1) Eq. 6.54

Therefore, the resistance of the hybrid beam to lateral-torsional buckling is adequate.

## 4.9 Shear resistance

The design plastic shear resistance of the beam is given by:

$$V_{pl,Rd} = \frac{A_v (f_y / \sqrt{3})}{\gamma_{M0}}$$ 6.2.6(2) Eq. 6.18

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The shear area for a welded I-section loaded parallel to the web is: (6.2.6(3))

$$A_{\mathrm{v}} = \eta h_{\mathrm{w}} t_{\mathrm{w}}$$

EN 1993-1-5 recommends that for steel grades up to and including S460 $\eta = 1.20$. Therefore: (EN 1993-1-5 5.1(2) (Note that the UK NA gives $\eta = 1.0$))

$$A_{\mathrm{v}} = \eta h_{\mathrm{w}} t_{\mathrm{w}} = 1.20 \times 1460 \times 12 = 21000 \text{ mm}^2$$

The plastic shear resistance is therefore:

$$V_{\mathrm{pl,Rd}} = \frac{21000 \times (355/\sqrt{3})}{1.0} = 4310 \text{ kN}$$

$$\frac{V_{\mathrm{Ed}}}{V_{\mathrm{pl,Rd}}} = \frac{1300}{4310} = 0.30 < 1.0$$

Therefore, the plastic shear resistance of the hybrid beam is adequate.

### 4.9.1 Shear Buckling

When calculating the shear buckling resistance of the web

$$\varepsilon = \sqrt{\frac{235}{355}} = 0.814$$ (6.2.6(6))

with $\eta = 1.20$, then

$$\frac{h_{\mathrm{w}}}{t_{\mathrm{w}}} = \frac{1460}{12} = 122 > 72\frac{\varepsilon}{\eta} = 72 \times \frac{0.814}{1.20} = 48.8$$

A check of shear buckling is therefore required.

The resistance to shear buckling is calculated as follows:

$$V_{\mathrm{b,Rd}} = V_{\mathrm{bw,Rd}} + V_{\mathrm{bf,Rd}} \le \frac{\eta f_{\mathrm{yw}} h_{\mathrm{w}} t}{\sqrt{3}\gamma_{\mathrm{M1}}}$$ (EN 1993-1-5 Eq. 5.1)

Where:

$V_{\mathrm{bf,Rd}}$ is the contribution from the flanges to shear buckling

$V_{\mathrm{bw,Rd}}$ is the contribution from the web to shear buckling

#### Contribution from flanges

When the flange resistance is not completely utilized in resisting bending moment the contribution from the flanges should be obtained as follows: (EN 1993-1-5 5.4(1))

$$V_{\mathrm{bf,Rd}} = \frac{b_{\mathrm{f}} t_{\mathrm{f}}^2 f_{\mathrm{yf}}}{c\gamma_{\mathrm{M1}}}\left(1 - \left(\frac{M_{\mathrm{Ed}}}{M_{\mathrm{f,Rd}}}\right)^2\right)$$ (Eq. 5.8)

$$c = a\left(0.25 + \frac{1.6 b_{\mathrm{f}} t_{\mathrm{f}}^2 f_{\mathrm{yf}}}{t h_{\mathrm{w}}^2 f_{\mathrm{yw}}}\right)$$

Where $t$ is the thickness of the web, and $a$ is the spacing between transverse stiffeners. In this case, since the beam only has transverse stiffeners at the ends $a = 8000$ mm. In addition, the width of the flange on each side of the web should be taken no larger than $15\varepsilon t_{\mathrm{f}}$. Therefore:

$b_{\mathrm{f}} = 400 \text{ mm} \le 2 \times 15\varepsilon t_{\mathrm{f}} + t = 2 \times 15 \times 0.731 \times 20 + 12 = 451$ mm, hence $b_{\mathrm{f}} = 400$ mm.

$$c = 8000\left(0.25 + \frac{1.6 \times 400 \times 20^2 \times 440}{12 \times 1460^2 \times 355}\right) = 2100 \text{ mm}$$

Considering that for a simply supported beam $M_{Ed} = 0$ at the supports, then:

$$V_{bf,Rd} = \frac{400 \times 20^2 440}{2100 \times 1.0} \times 10^{-3} = 34 \text{ kN}$$

**Contribution from web**

$$V_{bw,Rd} = \frac{\chi_w f_{yw} h_w t}{\sqrt{3}\, \gamma_{M1}}$$

EN 1993-1-5 Eq. 5.2

Since transverse stiffeners are only provided at the end supports to form non-rigid end posts, then

$$\bar{\lambda}_w = \frac{h_w}{86.4 t\varepsilon} = \frac{1460}{86.4 \times 12 \times 0.814} = 1.73$$

EN 1993-1-5 Eq. 5.5

Because $\bar{\lambda}_w > 1.08$ (From Table 5.1 of EN 1993-1-5),

$$\chi_w = \frac{0.83}{\bar{\lambda}_w} = \frac{0.83}{1.73} = 0.480$$

Thus the buckling shear resistance considering only the contribution from web is:

$$V_{bw,Rd} = \frac{0.480 \times 355 \times 1460 \times 12}{\sqrt{3} \times 1.0} \times 10^{-3} = 1720 \text{ kN}$$

EN 1993-1-5 Eq. 5.2

And the shear buckling resistance considering both the contribution from the web and flanges is:

$$V_{b,Rd} = 1720 + 34 = 1754 \le \frac{1.2 \times 355 \times 1460 \times 12}{\sqrt{3} \times 1.0} \times 10^{-3} = 4310 \text{ kN}$$

*Note that the contribution from the flange is only 34 kN, which is less than 2% of the contribution from the web – and small enough to be neglected.*

$$\frac{V_{Ed}}{V_{b,Rd}} = \frac{1300}{1754} = 0.74 < 1.0$$

Therefore, the shear buckling resistance of the hybrid beam is adequate.

## 4.10 Flange induced buckling

To prevent flange induced buckling, the following criteria must be satisfied:

EN 1993-1-5 8(1) Eq. 8.1

$$\frac{h_w}{t_w} \le k \frac{E}{f_{yf}} \sqrt{\frac{A_w}{A_{fc}}}$$

Since the cross-section is Class 4, the factor $k = 0.55$ , then

$$\frac{1460}{12} \le 0.55 \frac{210000}{440} \sqrt{\frac{1460 \times 12}{7999}} \text{ or } 122 \le 389$$

The criteria is satisfied, so there is no flange induced buckling of the web.

## HIGH STRENGTH STEEL DESIGN AND EXECUTION GUIDE

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**SCI**
Silwood Park, Ascot, Berkshire. SL5 7QN UK
T: +44 (0)1344 636525
F: +44 (0)1344 636570
E: reception@steel-sci.com
**www.steel-sci.com**