Global Research and Development
Esch-sur-Alzette

ArcelorMittal

Practical guide

Connections made of jumbo and super-jumbo steel shapes

May 2021

Table of Contents

List of Figures

List of Tables

Glossary

Backing – a material or device placed against the back side of the joint, or at both sides of a weld, to support and retain molten weld metal.

Base metal (parent metal) – the metal or alloy that is welded, brazed, soldered or cut.

Bevel angle – the angle between the bevel of a joint member and a plane perpendicular to the surface of the member.

Butt joint – a joint between two members aligned approximately in the same plane.

Butt weld – a nonstandard term for a weld in a butt joint.

CJP (complete joint penetration) – a joint root condition in which groove weld metal extends throughout the complete joint thickness.

CJP groove weld – a groove weld which has been made from both sides or from one side on a backing having CJP and fusion of weld and base metal throughout the depth of the joint.

Fatigue – damage that might result in fracture after a sufficient number of stress fluctuations within a sufficient stress range. Stress range is defined as peak-to-through magnitude of these fluctuations.

FCAW (flux cored arc welding) – an arc welding process that uses an arc between a continuous filler metal electrode and the weld pool. The process is used with shielding gas from a flux contained within the tubular electrode, with (FCAW-G) or without (FCAW-S) additional shielding from an externally supplied gas, and without the application pressure.

Filler metal – the metal or alloy to be added in making a welded, brazed or soldered joint.

Groove weld - a weld made in the groove between two pieces

GMAW (gas metal arc welding) – an arc welding process that uses an arc between a continuous filler metal electrode and the weld pool. The process is used with shielding from an externally supplied gas and without the application of pressure.

GMAW- S (gas metal arc welding – short circuit arc) – a gas metal arc welding process variation in which the consumable electrode is deposited during repeated short circuits.

HAZ (heat – affected zone) – the portion of the base metal whose mechanical properties or microstructure have been altered by the heat of welding, brazing, soldering or thermal cutting.

HISTAR® steel – innovative high strength steel for economic structures – steel structural grades developed using Quenching and Self-Tempering Process (QST) having low alloy content and combining high strength, good toughness and superior weldability.

Heat input - Heat input is typically used to directly estimate the amount of thermal energy that is introduced into the joint, using the following equation: H= 60EI/ 1000Sw; where: E= arc volts; I = amperage; Sw = travel speed and H = Heat input or arc energy per unit length.

Interpass temperature – In multiple-pass welds, the interpass temperature is the temperature of the steel before subsequent weld passes are initiated

Joint – the junction of members or the edges of members that are to be joined or have been joined.

Joint penetration – the distance the weld metal extends from the weld face into a joint, exclusive from weld reinforcement.

Joint root – that portion of a joint to be welded where the members approach closest to each other. In cross section, the joint root may be either a point, a line or an area.

Jumbo shapes - steel shapes, known as group 4 and 5 in AISC definition of W shapes, with maximum flange thickness of 115 mm, by example W 360x410x1086 (W 14 x 16 x 730).

Manual welding – welding with the torch, gun or electrode holder held and manipulated by hand, e.g., SMAW. Accessory equipment, such as part motion devices and manually controlled filler material feeders may be used.

Magnetic particle inspection (MPI or MT) utilizes the change in magnetic flux that occurs when a magnetic field is present in the vicinity of a discontinuity, can reveal cracks, incomplete fusion, slag inclusions, and porosity, but all must be surface-breaking

Nondestructive testing (NDT) - the process of determining acceptability of a material or a component in accordance with established criteria without its future usefulness.

Oxygen cutting (OC) – a group of thermal cutting processes that severs or removes metal by means of the chemical reactions between oxygen and the base metal at elevated temperature. The necessary temperature is maintained by the heat from an arc, an oxyfuel gas flame or another source.

Packing plate (filler plate) / pack – plates employed with the primary purpose of filling the gap between elements in a steel structural connection

Penetrant testing methods (PT) – nondestructive testing that involves the application of a liquid, which, through capillary action, is drawn into a surface-breaking discontinuity, such as a crack or porosity.

PJP (partial joint penetration) – a joint root condition in which groove weld metal extends throughout only a portion of the complete joint thickness.

Preheating – the application of heat to base material immediately before welding, brazing, soldering, thermal spraying or cutting.

SMAW (shielded metal arc welding) – an arc welding process with an arc between a covered electrode and the weld pool. The process is used with shielding from decomposition of the electrode covering, without the application of pressure, and with filler metal from the electrode.

Serial size – serial size represents the same family of a rolled sections, having a constant internal height "d", being suitable for column design due to their optimized shape in transferring the axial load

Strain energy - Strain energy is defined as the energy stored in a body due to deformation. The strain energy per unit volume is known as strain energy density and the area under the stress-strain curve towards the point of deformation.

Super-jumbo shapes - steel shapes (rolled or built-up) having flange thickness greater than 115 mm. The maximum thickness that can be reached is 140 mm with W360x410x1299 (W 14 x 16 x 873) steel shapes that have a flange thickness greater than or equal to 140 mm.

WPS qualification – the demonstration that welds made by a specific procedure can meet prescribed standards.

Weld pass – a single progression of welding along a joint. The result of a pass is a weld bead or layer.

Weld root – the points, as shown in cross section, at which the root surface intersects the base metal surfaces.

Weld sequence – the order of making the welds in a weldment.

Weld toe – the junction of the weld face and the base metal.

Foreword

As a global leader in the production of structural steel shapes, ArcelorMittal has developed this handbook as a consolidation of guidance and literature relevant to the connection of jumbo and super-jumbo steel shapes in the context of building frame elements that are not subject to seismic loading. Focused primarily on the existing documentation used by European designers, the handbook aims to orient these professionals to international best practices and alternative resources that can fill the gaps that may be perceived in these typical standards.

Specifications, codes and standards that are referenced by this document include the following:

- EU codes:
  - EN 1993-1-1
  - EN 1993-1-8
  - EN 1993-1-10
  - EN 10365
  - EN 1090-2: 2008
  - EN 10025-1: 2004
  - EN 10025-2: 2019
  - EN 10025-4: 2019
  - EN1011-1: 2009; EN 1011-2:2001
- US codes:
  - AWS D 1.1 2015
  - ANSI/AISC 360-16

The handbook was developed with a constant reference to ECCS book "Joints in Steel and Composite Structures" as far as it concerns structural joints. The book contains mainly aspects such as design, fabrication and erection.

Executive summary

This handbook addresses best practices to employ during the design, fabrication and erection stages of a project, especially as they relate to the connection of "jumbo shapes" and "super-jumbo shapes". With definitions in both EN 10365 « Hot rolled steel channels, I and H sections »[2] as well as AISC 360 (2016) (i.e., steel shapes that have a flange thickness higher than 50 mm), [5], Chapter A3-1c (i.e., "rolled heavy shapes" are defined as steel shapes that that have a flange thickness greater than or equal to 50 mm), it is universally recognized that special care must be taken when executing connections on these steel shapes, which may be considered to have "exceptional" flange thickness.

Steel is an anisotropic material. Therefore, for hot-rolled structural shapes, the best properties are obtained in the direction of rolling [25] and through-thickness properties must be respected. In terms of welded connections, the AISC Specification Commentary A3.1a contains this summary: "Rolled steel is anisotropic, especially insofar as ductility is concerned; therefore, weld contraction strain in the region of highly restrained welded connections may exceed the strength of the material if special attention is not given to material selection, details, workmanship and inspection." A good connection represents strong coordination between the different project stakeholders (i.e., the design engineer, architect, fabrication shop, assembler/ erector, control office and supplier).

In the normal course of events a steel structure passes through several separate stages involving design, detailing, fabrication and erection. It is important for designers to remember that a minimum of design detailing by them will assist towards economy, since the steel detailer is then left free to make the most efficient use of the particular fabricator's capabilities. Such things as a fabricator's ability to fabricate large subassemblies in the shop and subsequently transport to site and erect them will obviously have a bearing on the design of connection types and therefore on the economy of the overall project. In this regard, it must be stressed that a maximum of work done in the workshop will almost always produce better quality and more economical structures.

Awareness should also be raised on the static concept, the disposal of the connection in the structure, design requirements (EN 1993-1-8:2010-12) and material specification.

Connections need to be defined based on applied forces. They must be robust enough to fulfill requests of structural integrity, realized to avoid secondary effects, able to develop enough rotation capacity and, where needed, able to sustain cyclic solicitations. It is equally important to recognize that welded connections of jumbo and super-jumbo profiles, especially in high strength steel, require an adequate expertise in terms of welding procedure and specification of steel quality for not only the structural sections but also filler metals. Welded connections subjected to tension and flexural loads are particularly sensitive. Therefore, in addition to utilizing this document, it is advisable that designers engage ArcelorMittal's technical advisory department (sections.sales@arcelormittal.com), especially when managing the design of welded connections in cases where the structural sections have flange thicknesses exceeding 50 mm for ML sub-grade and 80 mm for M sub-grade.

Organization of ArcelorMittal's Handbook of connection solutions for jumbo and super-jumbo steel shapes is as follows:

- Chapter 1 explains the principle of choosing the right steel grade and its importance in the lifespan of a structure and Arcelormittal's guidance in the designing process from scratch and until the is used as a structural element. Ductility, through thickness and material properties are explained into detail to help the reader understand into detail fracture toughness introduction paragraph. Furthermore, the methodology of material selection based on fracture toughness from EN 1993-110:2005 is detailed along with a procedure that derivate from Table 1.1 for the above mention design code, subjected to quasi-static compression loading.
- Chapter 2 details the welded connections from the design code principles to the metallurgical point of view. The preparation of jumbo beams before welding affects the quality and fabrication cost. The ductile nature of the steel is not translated into a ductile structure, and the placement of weld access holes needs to be placed strategically. Lamellar tearing being a welding- related type of cracking in the base material, is being explained from its development, how to avoid it and method of detection. Welding parameters, like the suitable welding process, the electrode, heat input and not the last post weld heat treatment, play a major role into execution of a proper weld and consequently a reliable connection.

- **Chapter 3** classifies the connections from their structural role and the external efforts to which they are submitted. Several categories are considered, connections submitted to tension respectively compression loads, beam and column splices, trusses and beam -column connection. Examples of actual structures using jumbo connections are presented.
- **Chapter 4** contains the main influencing factors for jumbo connections, from the architectural aspects, specifying the proper steel shape and grade, manufacturing, safety and transportation.
- **Annex A:** Example of butt welding with H5 of HD 400 x 1299 (tf = 140 mm) of HISTAR® 460 contains a butt weld assembly performed by Wallerich workshop situated in Sanem, Luxembourg. The welding properties are accordingly EN ISO 15614.
- **Annex** B: The correct specification of steel – column under axial compression and **Annex C:** The correct specification of steel - column under eccentric compression load presents two examples for the correct specification of steel to avoid brittle fracture are presented: one example with a column under axial compression and a second example with a column under eccentric loading (i.e. parts of the column section are under tension).
- **Annex D** Design example - column splice - bearing type (No net tension) presents a splice column design of two columns coming from the same serial size - HD 400 x 744 and respectively HD 400 x 990.

1. Steel shape and material specification [4]

The safety and serviceability of civil engineering works in steel require the correct specification of steel according to EN 1993-1-1[6]. The various parts of EN 1993-1-1 provide requirements for resistance, serviceability, durability and fire resistance design. These requirements are based on limit state design. Simplified design models and a safety concept to verify the compliance of a steel with requirements have been implemented in the various parts of EN 1993-1-1. The application condition of the simple design models and the safety concept are first: that the resistance of the cross-sections and members are determined in tests in which the steel properties comply with product standards (e.g. EN 10025-4: 2005 [7]) and the execution quality complies with execution standards (e.g. EN 1090-2: 2008 [8]) and secondly: that test results are evaluated according to EN 1990: 2002 [9]. This procedure provides consistency between the properties in product norms and the design rules in EN 1993 and is only valid for the upper shelf region of the temperature toughness curve behavior [10].

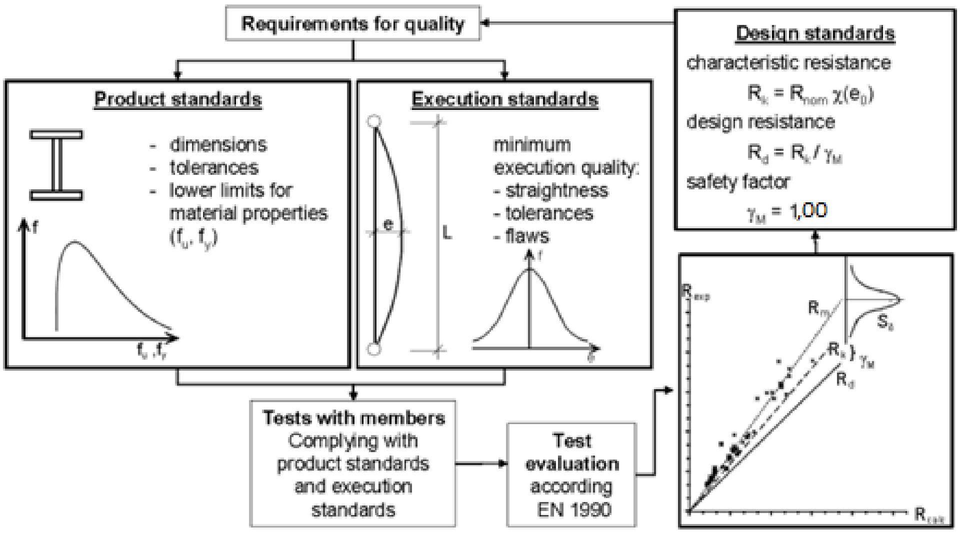


Figure 1-1. Consistency of product, execution and design standards.

In fact, the tests are conducted at room temperature corresponding to the upper shelf region of the temperature-toughness curve for which ductile failure of the steel occurs. In consequence, brittle failure of steel is not covered by the simplified design models, nor the safety concept, and, therefore, must be excluded by an appropriate specification of steel with sufficient toughness.

Therefore, two conditions must be met to secure ductile behavior for the design situations in EN 1993:

- Material properties must be selected within the upper-shelf region of the temperature-toughness diagram as described in Section 3 of EN 1993-1-1[6].
- The selected steel must have toughness properties considered sufficient to avoid brittle fracture fraction when performing additional verification in the transition range of the temperature-toughness diagram in EN 1993-1-10 [11].

1.1 Designation of steel grade

Designation of the steel grade according to EN 10025: 20-19 includes information on steel group; minimum yield strength at ambient temperature; minimum guaranteed toughness at a specific temperature (Charpy V-notch test); additional requirements, such as through-thickness properties; and delivery conditions (see Figure 1-2). With respect to the requirements listed above, designation of the steel grade is defined in the product standard for hot-rolled products of structural steels EN 10025: 2004.

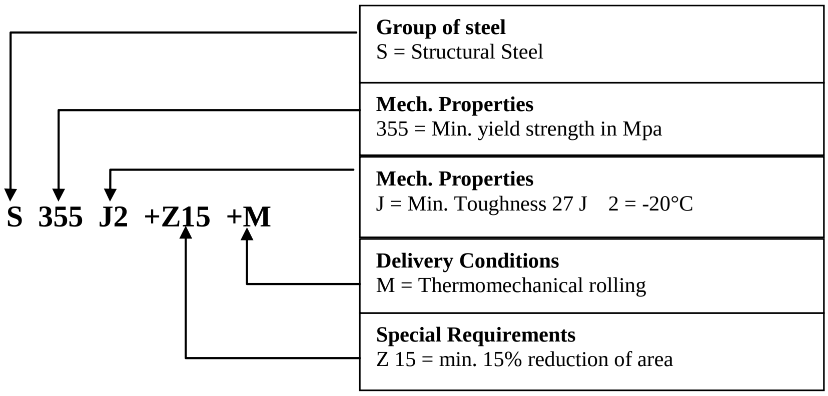


Figure 1-2. Designation of steel grades.

1.2 Delivery conditions

The product standard EN 10025-2:2019 [12] differentiates delivery conditions based on rolling process, with primary rolling processes of note defined as follows:

- Hot rolling, in which steel is rolled (i.e. formed or deformed) only under elevated temperatures (i.e. above the material's recrystallization temperature).
- Controlled rolling, in which steel is rolled (i.e. formed or deformed) under a series of decreasing temperatures to, therefore, impose stricter control on the final formation of the steel's internal microstructure.

Per EN 10025, options on delivery conditions are defined as follows:

- Delivery Condition "+AR" (i.e. "as-rolled"): Based on the classic hot rolling process, the delivery condition "+AR" implies that the steel will be delivered (i.e. produced) without the application of controlled rolling processes nor the application of controlled cooling processes.
- Delivery Condition "+M" (i.e. "thermo-mechanically rolled"): Based on a controlled rolling process, the delivery condition "+M" implies that the steel will be delivered (i.e. produced) by deforming the steel within controlled temperature ranges to therefore form a specific final internal microstructure and specific material properties, which cannot be obtained by a thermal treatment alone and which cannot be repeated. With the thermo-mechanical rolling process, it is possible to produce fine-grain steels with high strengths, good toughness properties and improved processing characteristics. A further refinement of the steel microstructure, especially for greater material thicknesses, can be achieved with accelerated cooling and controlled reheating process – using water (to cool) and internal energy or heat (to self-temper). This process, referred to as quenching and self-tempering (or QST) results in fine-grain steels with very high guaranteed yield strength and excellent toughness properties, even for low temperature applications.

For greater thicknesses, and in specific applications, enhanced toughness is required by European design standards. As an example, EN1993-2: 2007 requires fine-grain steels in accordance with EN 10025-3:2004 or EN 10025-4 [7], notably S355M, S460M for thicknesses over 30mm and S355ML, S460ML for thicknesses over 80mm.

1.3 Material properties

Material properties, including yield strength f_y and ultimate strength f_u, are specified in relevant ENs or ETAs. Yield strength is defined as the stress at which a material begins to plastically deform, while ultimate strength represents the maximum stress that a material can withstand before breaking when subjected to stretching or pulling.

The specification of the mechanical material properties for structural steel concerns selection of the appropriate f_y and f_u for a particular application. According to EN 1993-1-1 the nominal values for f_y and f_u should be obtained by adopting the values $f_y = R_{eh}$ and $f_u = R_m$ direct from the product standard or from Table 3.1 in EN 1993-1-1 and Figure 1-3.

Verification of the steel's mechanical properties is carried out by the producer, who uses standard tensile tests according to EN 10002-1:2001[13], which indicates the tensile testing of steel materials at ambient temperature, and EN 10025-1:2004 [7], which indicates the general technical delivery conditions for hot-rolled products. Results of the tensile tests are observed as a stress-strain curve from which yield (f_y) and ultimate (fu) strengths are determined and subsequently indicated on the mill test certificate, as in Figure 1-4 a).

Especially in the context of jumbo and super-jumbo shapes, it is important to be aware that according to EN 10025-4:2019, f_y of a steel typically decreases with increasing material thickness. In fact, to maintain a constant value of f_y over the full range of thicknesses for a steel requires either the addition of more alloying elements – which can increase carbon equivalent value and negatively impact weldability of the steel – or modification to the production process – which can increase material cost. Capable of mitigating the loss in efficiency compared to other steels, **HISTAR® steels are permitted under ETA-10/0156 to be used with just limited, or no (HISTAR 355), reduction in strength as thickness of a shape grows**. Comparison of the behavior between HISTAR® steels and other European specifications is shown in Figure 1-3.

1.4 ArcelorMittal HISTAR® steel

Development of HISTAR® steels demonstrated success in creating structural steels that combine high yield strength, excellent toughness at low temperature, and outstanding weldability. Available for hot-rolled HD and HL sections, HISTAR® steel enables the development of innovative and competitive structural solutions for buildings. Engineers can take full advantage of HISTAR® steel's properties when an element's strength drives the design. Common applications include gravity columns of high-rise buildings, long-span trusses and components in offshore structures. Furthermore, the new steels are recommended in case of stress governed as well as seismic design [15]. HISTAR® steel is available in grades ranging from minimum yield strengths of 355 MPa to 460 MPa. A particularly unique characteristic of HISTAR® steel is the constant yield value that it features along its complete range of thicknesses for HISTAR® 355, and the minimal drop in yield strength for only sections with thicknesses exceeding 100mm for HISTAR® 460 – see Figure 1-3. A complete list of high rise buildings with HISTAR® or ASTM 913 steel grades can be found in the ArcelorMittal high rise brochure [38] and [39].

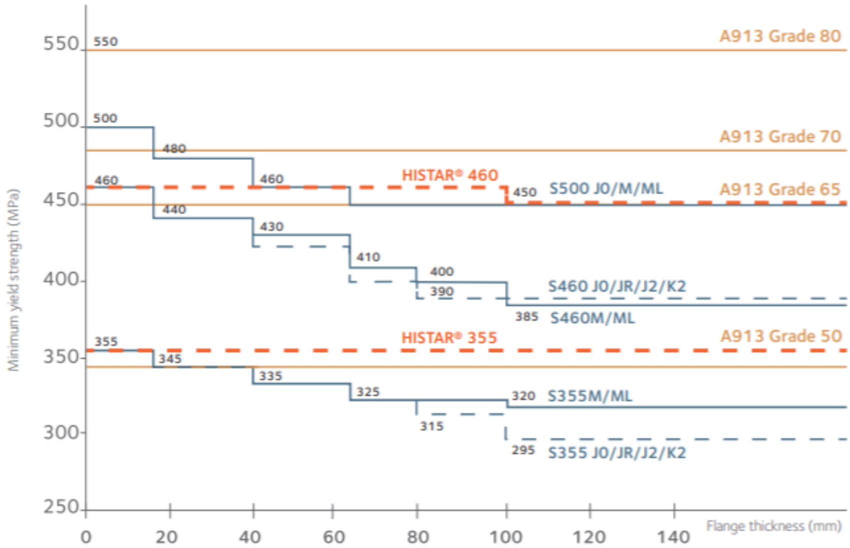


Figure 1-3. Minimum yield strength variation according to the material thickness for HISTAR® steels in comparison with EN 10025-4:2019, ASTM A913 steels.[45]

1.5 Ductility

Ductility represents a measure of a material's ability to undergo significant plastic deformation before rupture or breaking and can be expressed as a percent elongation or percent area reduction from a tensile test. In EN 1993-1-1, various requirements on ductility are specified to avoid brittle failure of structural steel elements. In particular, it states that a minimum ductility should be expressed with limits as follows:

1. The ratio f_u/f_y of the specified minimum ultimate tensile strength f_u to the specified minimum yield strength f_y, where EN 1993-1-1 recommends a value of $f_u/f_y \geq 1.10$.
2. The elongation at failure on a gauge length of $5.65\sqrt{A0}$ (with A0 defined as the initial cross-sectional area of the test sample). EN 1993 -1-1 recommends an elongation at failure not less than 15 %.
3. The ultimate strain ε_u where ε_u corresponds to the ultimate tensile strength f_u. EN 1993-1-1 recommends $\varepsilon_u \geq 15\varepsilon_y$ where ε_y is the yield strain ($\varepsilon_y = f_y / E$).

In the context of high-strength steels (e.g. S500), the second and third criteria are of particular interest as it is typically expected that high-strength steels will present a lower elongation at failure when compared to lower strength steels (e.g. S235). This expectation is also reflected in the product standard (EN 10025-4) [7], which requires high-strength steels to meet more severe elongation limits than EN 1993-1-1. To combat this common belief, Figure 1-4 a) features a comparison of typical stress-elongation curves among steels of varying strengths. This comparison demonstrates that modern, thermo-mechanically produced, high-strength steels meet the minimum required elongation values of both EN 1993-1-1 and EN 10025-4 with high safety margins, and several tension tests have been conducted to verify that the criterion of the ratio f_u/f_y – which is often more critical than the elongation criteria – is also met. As demonstrated in the figure, for steels over 460 MPa, it is only modern, thermo-mechanically produced steels – with their refined microstructure and reduced microalloying content – that can respect the three limits mentioned above.

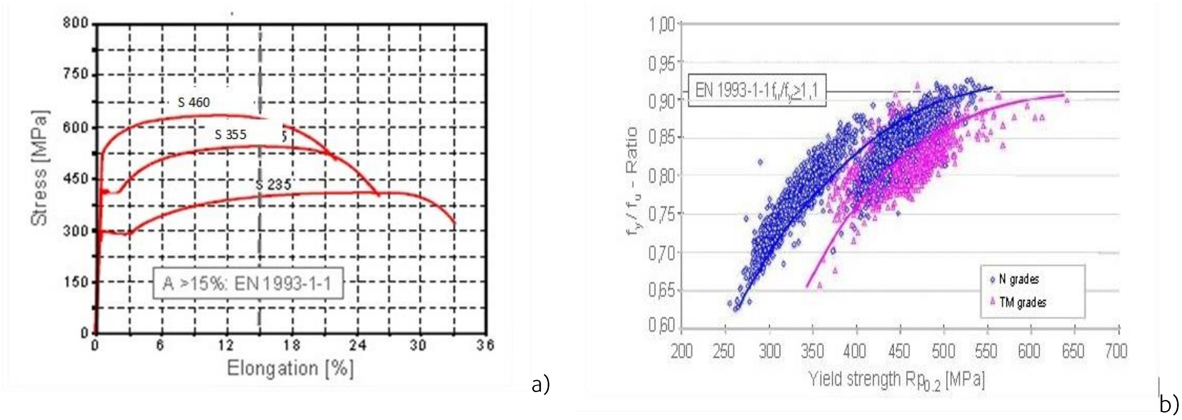


Figure 1-4. a) Typical stress-elongation curves of structural steels; b) Tensile to yield strength ratio according to yield strength.

1.6 Through-thickness properties

Requirements on through-thickness properties of structural steel should be respected to avoid, where susceptible, lamellar tearing as presented in Paragraph 2.2.

For the specification of through-thickness properties, EN 1993-1 refers to EN 1993-1-10 [11]. Per EN 1993-1-10, the proper specification of through-thickness properties depends on two classes reflecting the risk of occurrence of lamellar tearing: (1) for a *considerable* risk of lamellar tearing, the specification of the through-thickness properties corresponds to the Z-grade which identifies the quality class for through-thickness ductility with respect to EN 10164:2004 [29]; and (2) for an *insignificant* risk of lamellar tearing, post fabrication inspection should be carried out to identify whether lamellar tearing has occurred.

Lamellar tearing may be considered insignificant if the following condition is fulfilled: $Z_{Ed} \leq Z_{Rd}$

where: Z_{Ed} is the design Z-value resulting from the magnitude of strains from restrained metal shrinkage under the weld beads and Z_{Rd} is the design Z-value to avoid lamellar tearing.

$$Z_{Ed} = Z_a + Z_b + Z_c + Z_d + Z_e$$

for which Z_a, Z_b, Z_c, Z_d and Z_e are given in Table 3.1 in EN 1993-1-10. These factors represent the influence of the local straining on the through-thickness ductility of the material, where:

- Z_a accounts for the weld depth;
- Z_b accounts for the position and the shape of the weld;

- Z_c considers the effect of the material thickness;
- Z_d quantifies the remote restraint of shrinkage after welding; and
- Z_e accounts for the influence of preheating.

1.7 Fracture toughness – Introduction

The ability of a material to resist brittle fracture when stressed is termed toughness, and the numeric value used to represent the amount of energy per volume that a material can absorb before rupture [10] is considered its fracture toughness. The brittle fracture of steel is not covered by the design models and the safety concept presented in EN 1993-1, and it may be excluded when a material of sufficient toughness is employed in design. The specification of a material with appropriate toughness properties for a given design is addressed in EN 1993-1-10. The basis of the specification is an Ultimate Limit State verification based on fracture mechanics for an accidental design situation, with the verification accounting for the following influences:

- Detailing of the steel member
- Geometry and position of crack-like flaws observed during inspection
- Residual stresses that develop from welding, as well as stresses from remote constraints
- Dependency of the toughness on the temperature
- Strain-rate dependency of the toughness
- Dependency of the toughness on the degree of cold forming

The principle of fracture mechanics consists of quantifying the stress concentration in the vicinity of a crack tip by a single parameter, with three parameters typically being used in evaluations:

- Stress intensity factor K (in MPa√m). This factor was derived by Westergaard [16] and is a measure for the linear elastic stress field near the crack. The K-factor is only valid for linear elastic stress distributions and plastic deformations at the crack tip may be limited for its use. Three different stress intensity factors exist for three different load and crack opening modes (Mode I - tensile; Mode II – in-plane shear; and Mode III – out-of-plane shear). In practice, Mode I is most common and the other two load types are not further considered. Therefore, the stress intensity factor is noted with KI (with "I" signifying Mode I). When the KI-factor reaches a critical value, KIC, unstable crack growth occurs.
- J-Integral (in J/mm2): This concept was introduced by Rice [17] and is a line integral of the strain energy including the crack. According to the law of conservation of energy, the integral along a closed line is zero; however, this is not the case if a singularity in the form of a crack is present in the line integral. By the free choice of the line integral, plasticity in the crack area can be considered. J-Integral can be determined numerically or experimentally.
- Crack Tip Opening Displacement, or CTOD (in mm). This is a parameter that measures the opening δ at the crack tip. If the opening reaches a critical value δ_c at the fatigue-blunted crack tip, critical crack growth occurs. CTOD permits consideration of plastic deformation near the crack. It is measured optically or through the potential method [19].

The K-factor (stress-intensity) concept is limited to linear elastic stresses, while J-Integral and CTOD also account for plastic strains. Although it is only based on linear elastic material behavior, the K-factor concept is adopted in EN 10365 [2]. In fact, the stress intensity factor can easily be determined for most common cases with formulas available from handbooks.

1.8 Fracture toughness – Methodology in EN 1993-1-10:2005

The Ultimate Limit State verification consists of a comparison of the action effect $E_d = K^*_{appl,d}$ to the design values of the toughness resistance $R_d = K_{mat,d}$, thus: $K^*_{appl,d} \leq K_{mat,d}$

In general, stress level at the crack tip exceeds the yield strength of the steel material and local plasticity develops in the crack area. Properly speaking, the K-factor concept would not be valid in this case. The solution is the use of the Failure Assessment Diagram (FAD), which corrects the stress intensity factor $K_{appl,d}$ on the action side for plastic strains $K^*_{appl,d}$. The FAD accounts for the interaction between brittle failure and plastic collapse, and a description of the FAD is given in [19]

$$K^*_{appl,d} = \frac{K_{appl,d}}{k_{R6}-\rho} = \frac{\sigma_{Ed}\cdot\sqrt{\pi\cdot a_d}\cdot Y\cdot M_K}{k_{R6}-\rho}$$

where σ_{Ed} is the design value of the applied tension stresses from external loads [MPa],

a_d is the design size of the crack [m],

Y is a correction function for the position and the shape of the crack [-], see [10]

M_k is a stress concentration factor for non-structural welded attachments and semi-elliptical crack shapes [-], see [10]

k_{R6} is a plasticity correction factor from the FAD [-],

ρ is a correction factor for residual stresses [-]

Fracture toughness of a steel is not indicated in the form of K_{mat}-values in the product standard but rather as minimum guaranteed values for the notch impact energy K_V for a given testing temperature T in the form of T_{KV}.

Therefore, the verification in EN 1993-1-10:2005 is based on temperatures:

$$T_{Ed} \geq T_{Rd}$$

Where: T_{Ed} is the temperature requirement on toughness in the form of a reference temperature [°C]

T_{Rd} is the temperature corresponding to the toughness resistance of the material [°C]

In detail:

$$T_{Ed} = T_{md} + \Delta T_r + \Delta T_\sigma + \Delta T_R + \Delta T_{\dot{\varepsilon}} + \Delta T_{\varepsilon_{cf}} \geq T_{Rd} = T_{K100} + \Delta T_{27J}$$

where T_{md} is the lowest air temperature [°C],

ΔT_r is an adjustment for radiation losses [°C],

ΔT_σ is an adjustment for stress and yield strength, the crack imperfection and member shape and dimensions [°C],

ΔT_R is a safety allowance, in general +7°C for nominal values of T_{27J} and -38°C for measured values of T_{27J} [°C],

ΔT_ε is an adjustment for the strain rate [°C],

$\Delta T_{\varepsilon cf}$ is an adjustment for the degree of cold forming [°C],

T_{K100} is the test temperature for which the toughness is KV = 100 MPa√m [°C],

ΔT_{27J} is an adjustment for the influence of the material thickness [°C] – for more information please see the background document related to material toughness and trough thickness properties based on EN 1993-1-10 recommendations [49]

The correlation between test temperature T_{K100} and toughness value KV = 100 MPa√m is given by the standardized Master-Curve approach of Wallin [21] The temperature T_{K100} is correlated via the Modified Sanz-Correlation [13] to the Charpy V-notch test temperature T_{27J}, which is indicated in the product standard EN 10025-4.

The design size a_d of the crack due to fatigue is carried out by calculating the crack growth according to the Paris equation: $\frac{\Delta a}{\Delta N} = C \cdot \Delta K^m$

where Δa is the crack growth increment [m],

ΔN is the number of cycles, depending on the length of the "safe service periods" [-],

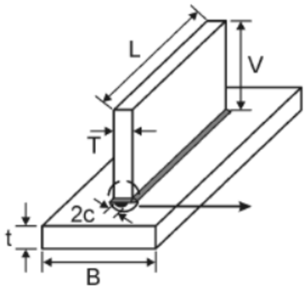
C is a constant depending on the material, for structural steels $C = 1.8\cdot 10^{-13}$,

m is a constant depending on the material, for structural steels m = 3,

ΔK is the stress intensity factor corresponding to the stress amplitude Δσ [MPa√m]

To simplify application of the above described approach for the user, a procedure has been developed for a standardized welded bridge detail and a semi-elliptical surface crack with standardized dimensions (see Figure 1-5).

The welded detail and the corresponding stress intensity factor safely cover all detail categories in EN 1993-1-9 [20]. This simplification led to the development of Table 2.1 of EN 1993-1-10, which delivers the maximum permissible value of element thickness for a particular material grade and subgrade based on reference temperature T_{Ed} and stress level σ_{Ed} for the accidental load case. The table represents a simple design aid for the practicing engineer to establish the correct specification of the material against brittle failure. It applies mainly to tension elements, as well as welded and fatigue-stressed elements in which some portion of the stress cycle is tension. For elements not subjected to tension stresses, welding or fatigue, application of the Table 2.1 can be conservative.



L/t = 8.20
T/t = 0.15
B/b = 7.50
a/c = 0.40
Φ = 45°

Figure 1-5. Standardized welded bridge detail.

Table 1-1. Maximum permissible values of element thickness t in mm (EN 1993-1-10:2005, Table 2.1)

Steel grade	Sub-grade	Charpy energy CVN at T [°C]	Charpy energy CVN J_{min}	Reference temperature T_{Ed} [°C] $\sigma_{Ed} = 0{,}75\ f_y(t)$: 10	0	-10	-20	-30	-40	-50	$\sigma_{Ed} = 0{,}50\ f_y(t)$: 10	0	-10	-20	-30	-40	-50	$\sigma_{Ed} = 0{,}25\ f_y(t)$: 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
S690	Q	0	40	40	30	25	20	15	10	10	65	55	45	35	30	20	20	120	100	85	75	60	50	45
	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	-20	40	60	50	40	30	25	20	15	95	80	65	55	45	35	30	165	140	120	100	85	75	60
	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	-40	40	90	75	60	50	40	30	25	135	115	95	80	65	55	45	200	190	165	140	120	100	85
	QL1	-60	30	110	90	75	60	50	40	30	160	135	115	95	80	65	55	200	200	190	165	140	120	100

In addition to Table 2.1, section 2.4 in EN 1993-1-10:2005 enables the use of the fracture mechanics method as well as numerical evaluation to cover details and crack geometries not covered by Table 2.1.

In US, the code AASHTO LRFD Bridge Design Specifications [46] recommends toughness requirements for structural steel in bridges. The code differentiates between fracture-critical and non-fracture critical details subject to tension loading.

For buildings, it is up to the project team to determine toughness requirements for a particular element or connection. In the particular cases where rolled heavy shapes and/or built-up heavy shapes, are employed for specific situations (as an example, as members subject to primary tensile forces and spliced using CJP welds), specific guidance is provided in AISC 360, Sections A3.1c and A3.1d for toughness requirements to be invoked in the base materials. For rolled heavy shapes, the guidance – identified as supplementary clause S30 per ASTM A6 [47] requires that the impact test taken at the alternate core location meet a minimum average value of 27 J absorbed energy at a maximum temperature of +21°C.

1.9 Specifying the correct steel for elements subjected to quasi-static compression loads

Within this handbook, quasi-static loading refers to loading with stress amplitudes that are noncritical concerning fatigue effects (i.e. no crack initiation or crack growing occurs under the loading).

Currently, EN 1993-1-10:2005 describes two different methods for the selection of materials for fracture toughness: (1) the simplified method (Table 1-1) as described in EN 1993-1-10:2005, section 2.3; and (2) utilization of a fracture mechanics method that accounts for a specific detail, flaw shape and position as described in EN 1993-1-10:2005, section 2.4.

The simplified method can be conservative for elements where fatigue plays a minor role. The same applies to elements not subject to tension stresses and welding. This results from the fact that the values in Table 1-1 have been derived based on a standardized welded bridge element subject to fatigue.

An alternative Table 1-1 the application of fracture mechanics theory for the selection of the right material against brittle failure is only generally described in EN1993-1-10:2005, section 2.4. Most practicing engineers do not have enough experience in fracture mechanics and they conservatively apply Table 1-1 to elements subject to quasi-static loads. As a result, the specification of steel that meets elevated toughness requirements may not be economical in general structural applications. Therefore, in this section, a table for elements subject to quasi-static compression loading is derived by applying fracture mechanics. The procedure is explained as follows:

- The derivation of a table like Table 1-1 that allows the specification of the steel grade for elements subject to quasi-static loading requires a well-defined procedure. The procedure is comprised of two steps:
- Step 1: Analyze how much the boundary conditions for elements under quasi-static loads differ from the assumptions made for the derivation of Table 1-1. The influencing parameters are identified and adapted to the boundary conditions.
- Step 2: Derive, based on the adapted parameters of "Step 1", a table that delivers the maximum permissible thickness values for elements under quasi-static compression loads. Table 1-2 delivers the maximum permissible thickness values for structural elements subject to predominant quasi-static compression loads. Under the influence of compression loads, the cracks are closed and subsequently considered less severe than they would be under tension load, where they are opened. Limiting this table's applicability to elements under predominant compression loads thereby allows application of the same fracture mechanic approaches, as well as the standard detail, size, and type of initial crack as defined in section 1.3.7 [3],[4]. The only difference results from adaption of the crack growing to quasi-static loading.

Table 1-2. Maximum permissible thicknesses for quasi-static compression loads.

Steel grade		KV		Reference Temperature						
				T_{Ed} [°C]						
Steel grade	Subgrade	T [°C]	J_{min}	10	0	-10	-20	-30	-40	-50
				$\sigma_{Ed} = 0.25 \cdot f_y(t)$						
S355	M	-20	40	150	150	150	150	150	150	150
	ML	-50	27	150	150	150	150	150	150	150
	J2	-20	27	150	150	150	150	150	150	150
S460	M	-20	40	150	150	150	150	150	150	150
	ML	-50	27	150	150	150	150	150	150	150
	J2	-20	27	150	150	150	150	150	150	150

According to section 3.2.3 in EN 1993-1-1, it is proposed to apply Table 1-1 with a tensile stress level of $\sigma_{Ed} = 0.25 \cdot f_y(t)$ for elements subject to compression loads. The application of a given portion of tensile stresses to sections subject to pure compression covers the presence of resulting residual tension stresses from production and/or fabrication. The maximum permissible thicknesses can be indicated with 150 mm for each reference temperature and steel grade. Because fatigue crack growth is not considered for quasi-static loading, much higher maximum permissible values could be justified with the applied method; however, values above 150 mm are not covered by the product standard, EN 10025-4.

When applying Table 1-2, $\Delta T_\sigma = 0$ °C and $\Delta T_R = 0$°C may be assumed, since the values are already included in the derivation of the table. Table 1-2 exemplary shows the advantage of offering in addition to Table 1-1 a table for elements subject to quasi-static loading: a more economic specification of steel for toughness assessment becomes possible. It is important to know, that Table 1-2 however is only applicable to bolted or welded connections submitted to compression details covered by the detail categories as described in [20].

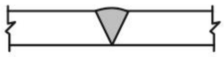
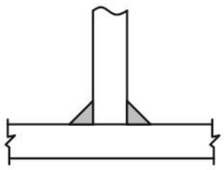
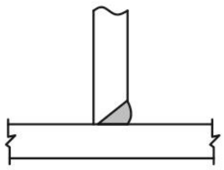
In the American specifications for structural steel buildings (ANSI/AISC 360-16), when members are subjected exclusively to compression loading, there are no specific toughness requirements to invoke.

2 Welded connections

Most common welding processes used in the fabrication and erection of structural steel involves melting and mixing of base metals and/or filler metals, followed by solidification and fusion of the various metals.

Three categories of welds that are especially common in major structural connections (i.e., the connection of jumbo and super-jumbo shapes) are the following: complete joint penetration (CJP) groove welds, partial joint penetration (PJP) groove welds and fillet welds, illustrative examples are shown in Figure 2-1.

Figure 2-1. Weld type definition.

		Weld type	
		Fillet	Groove
Joint type	Butt	N.A.	
	T		

Before welding, an appropriate Welding Procedure Specifications (WPS) should be established. The WPS can be based on a procedure that is prequalified in codes (i.e. ISO 15607:2019, ISO 15609-1:2019, ISO 15614-1:2017, or AWS D1.1), or a project team can develop a custom WPS that is qualified through testing as outlined in applicable design and construction standards.

Best practices

The AISC design guide for welded connections mentions several principles to conduct a correct and proper welded splice connection [24]

- The smallest weld size should be specified and consistent with the design requirement.
- The groove weld details that requires the least amount of weld metal should be selected.
- The joint preparation angles should be controlled.
- For a given weld size, the fewest number of weld passes should be designed.
- Overwelding should be avoided.
- In general, the highest levels of preheat should be used and a greater volume of base metal should be heated.
- A high attention should be payed to excessive preheating, that may result in excessive interpass temperatures. It can reduce the strength and the toughness. Our recommendations for HISTAR® material are not to exceed 250 °C interpass temperature.
- The weld is strong enough to transfer all the applied loads through the connections but doesn't have to be more than necessary.
- The connection is easy, economical and safe to fabricate and erect.
- The connection has a clear and direct load path. It should provide a path, so a transverse force can enter that part of the member or section that lies parallel to the force - Figure 2-2.

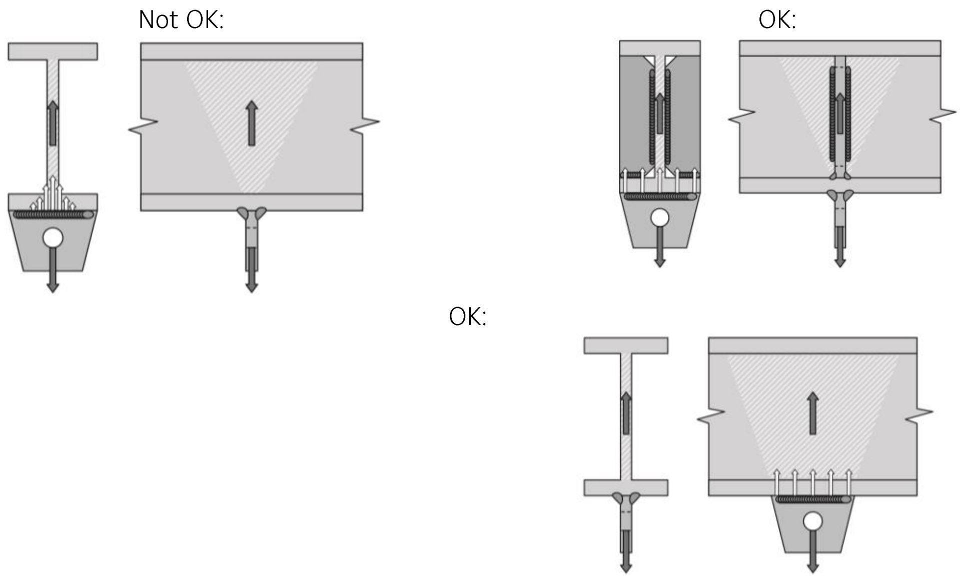


Figure 2-2. Weld load paths.

- Stress raisers should be avoided. Stress raisers are active if there is a tensile stress component perpendicular to the stress raiser. A planar discontinuity might be considered as a potential stress raiser such as a crack in the weld, an incomplete fusion plane or an unfused interface between steel backing and the surrounding steel.
- The joint is not constrained, does not subject the weld to bending, and has a clearly defined throat. A weld should respect the design dimensions -Figure 2-3.

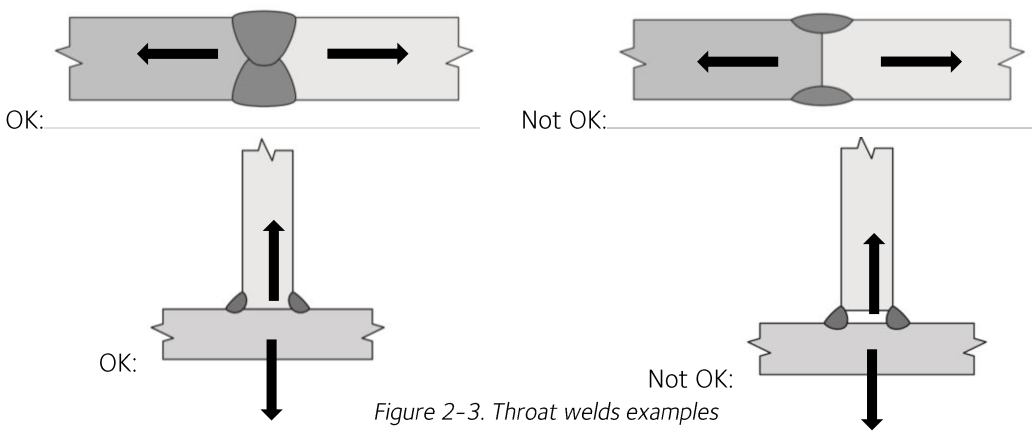


Figure 2-3. Throat welds examples

- The connection recognizes commercial realities -Figure 2-4.

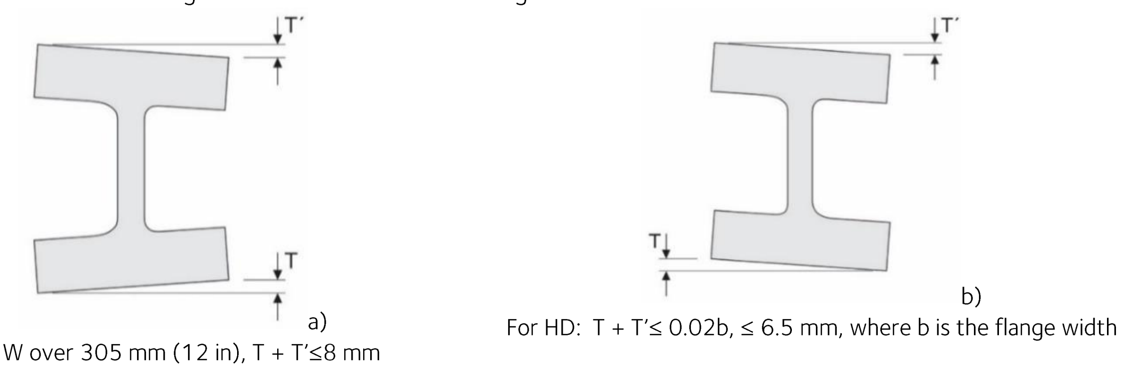


Figure 2-4. Tolerances for H shapes: a) ASTM A6; b) EN 10034 tolerances.

In addition to guidance from other norms, to overcome the myriad of challenges that arise when welding, it is important for designers to consider how the material that they are specifying can serve to facilitate easier processing. A product such as HISTAR® steel, whose chemical analysis demonstrates a low carbon equivalent value (CEV) – as shown in Table 2-1 – has a proven better weldability compared to conventional steel grades.

An evaluation of contributions from carbon and additional steel alloying elements, carbon equivalent formulae were originally developed to translate a steel's chemical composition to an indication of the equivalent level of hardenability. These formulae were later extended to reflect the susceptibility of the steel to hydrogen cracking, and they are also used as compositional characterizing parameters for other properties that may be linked to hardness, steel grade weldability and evaluation of preheating temperature as shown in Table 2-1.

Table 2-1. Chemical composition of HISTAR® steel grades for general applications.

Grades	Chemical composition														
	Ladle analysis [%]														
	C	Mn	Si (3)	P	S	Al (2)	Cr	Ni	Mo	Nb	Ti	V	CEV(1) max. Nominal thickness [mm]		
	max.	max.	max.	max.	max.	min.	≤	≤	≤	max.	max.	max.	t ≤ 63	63 < t ≤ 125	125 <t≤ 140
HISTAR® 355	0.12	1.6	0.5	0.03	0.03	0.02	0.3	0.3	0.2	0.05	0.05	0.1	0.39	0.39	0.39
HISTAR® 355 L	0.12	1.6	0.5	0.03	0.025	0.02	0.3	0.3	0.2	0.05	0.05	0.1	0.39	0.39	-
HISTAR® 460	0.12	1.7	0.6	0.03	0.03	0.02	0.3	0.7	0.2	0.05	0.05	0.12	0.41	0.43	0.43
HISTAR® 460 L	0.12	1.7	0.6	0.03	0.025	0.02	0.3	0.7	0.2	0.05	0.05	0.12	0.41	0.43	-

(1) CEV = C + Mn/6 + (Cr + Mo + V) /5 + (Cu + Ni) /15

(2) If enough nitrogen binding elements are present, the minimum aluminum requirement does not apply.

(3) Upon agreement: Si = 0.14 - 0.25 % and P ≤ 0.035% max. for capability of forming a zinc layer during hot-dip galvanization.

In the case of CJP groove welds for hot-rolled jumbo and super-jumbo sections, guidance from European norms is limited. An alternative source of suggestion is AISC 360 (2016), Section A3.1.c and J1.5, which is invoked when tensile forces are transmitted in heavy, hot-rolled jumbo and super-jumbo) sections by complete joint penetration groove welds. Under AISC 360, the following provisions should apply:

- Charpy V-Notch impact tests for structural hot-rolled shapes – alternate core location, requirements in accordance with ASTM A6/A6M (S30). The impact tests shall meet a minimum average value of 27J absorbed energy at maximum temperature of +21 °C.
- Weld access hole details as given in Section J1.6 and Paragraph 2.1.
- Filler metal requirements as given in Section J2.6 and Paragraph 2.3.
- Thermal cut surface preparation as given in [21].
- When the above-mentioned requirements are respected for CJP groove welds of jumbo and super-jumbo sections, profound effects due to weld shrinkage should be limited.

Challenges

Cracking is an issue that can arise in welded connections especially when material thickness exceeds 50 mm and strength is superior to 500 MPa; however, it is rare when fabrication is properly performed. The term weld cracking is used to describe the cracking that occurs in the heat-affected zone (HAZ) and/or weld metal during or after the time the weld is performed. The weld cracks can be defined into two categories: hot and cold cracks [24]

Hot cracking occurs only when the weld is hot, and it is related to solidification. Cold cracking, on the other hand, occurs when the weld is cool, and it is hydrogen related. As both types of cracking are completely different mechanisms, different approaches to mitigation are necessary.

Cold cracking can occur in the heat affected zone (HAZ) or sometimes in the weld itself. Hydrogen contamination can result from a variety of sources: (a) the parent metal; (b) the consumable; and (c) the atmosphere. It is therefore important to minimize the introduction of hydrogen in the weld pool, which can be controlled through electrode selection (see Section 2.3.2). Furthermore, the risk of cold cracking is increased if the weld cools quickly. For these reasons, a WPS should be prepared that describes how the weld is to be made: the equipment settings, type of consumable, preheat temperature, cooling rate, etc.

Hydrogen can be introduced in the arc atmosphere primarily from the decomposition of moisture released during the melting of fluxes present in electrode coatings or cores, and from the moisture surrounding the arc environment and be absorbed by the molten metal. At high temperatures the solubility of the hydrogen is high, and as temperature decreases, the capacity of the metal to hold the hydrogen in solution is greatly decreased. However, hydrogen can be trapped into the microstructure and the accumulation and saturation of residual hydrogen in the solidified structure of the weld joint can cause the cracking [26] To decrease the potential for cold cracking by hydrogen, the appropriate WPS needs to be applied, which controls the temperature and cooling rate in the range of low temperature of thermal cycle, typically from 300°C to 100°C. Considering the risk of delayed cracking, according to the EN 1011-2 [27] for high yield strength thick (> 50 mm) profiles, the final weld control must be performed after at minimum 16 hours after the welding. The AWS D1.1 clause 6.11, requires a delay of 48 hours after completing the weld.

2.1 Joint preparation and fit-up

Joint detailing and preparation are two critical factors that affect the quality of a weld and cost of its fabrication. The time spent preparing the assembly is more than offset by the welding speed and the improved quality of the weld. Careful edge preparation is essential to mitigate the development of additional complications in the weld.

Joint preparation, which often requires introducing a bevel to the steel edge, can be executed by several processes, including thermal cutting, machining, and water-jet cutting [21]. Such processing can be executed by a steel fabrication centre like ArcelorMittal's Steligence® Fabrication Centre (SFC), which provides value-added services like cold sawing, drilling, cambering, bending, and oxyacetylene, flame and robotic plasma cutting, that supplement the technical capabilities of key project stakeholders (steel fabricators and general contractors). Potential benefits include the assurance that a project partner will be capable of processing jumbo and super-jumbo sections, the availability of parallel workflows between the primary fabricator and the mill, and the potential for significant time and cost savings.

The ductile nature of the steel does not translate into a ductile structure, because the inherent material ductility alone is not sufficient to overcome compromising scenarios. Under conditions of low temperature, rapid loading, and/or high constraint (e.g. when the principle stresses σ1, σ2 and σ3 have similar values, i.e., in conditions susceptible to the inducement of triaxial stress, see Figure 2-5) even ductile materials can exhibit brittle fracture, or the absence of any deformation before fracture.

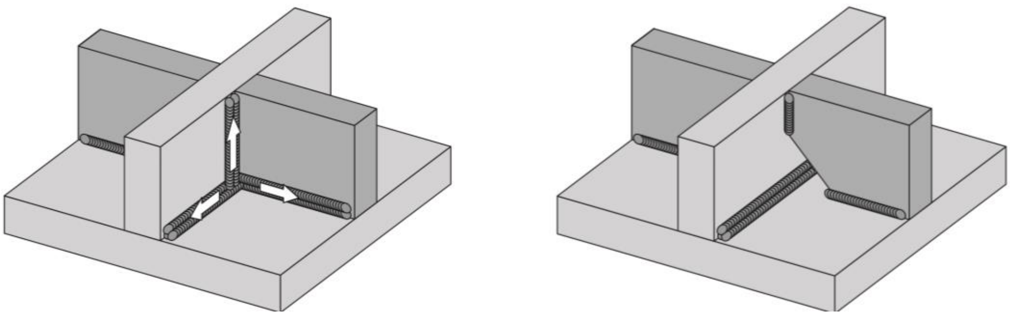


Figure 2-5. Weld access holes – principle stresses.

Residual stresses develop as a result of the tendency of material to expand when hot and contract when cool. In the case of welded connections, the hot weld material heats localized portions of the base metal, and when all of this material (both from the weld and the base metal) contracts while cooling, it causes a phenomenon called shrinkage. The shrinkage is resisted by the surrounding rigid base material, and this interaction leads to a restraint in the connection and straining of the base material. To reduce the strains and associated stresses, the volume and localized nature of the heated metal should be minimized. Reduction can be accomplished by detailing the connection in such way that mitigates the potential for highly restrained conditions, and/or it can be managed through preheating of the base material.

Weld access holes are strategically placed openings that allows access for welding, backgouging and for the insertion of backing material. They should be large enough to permit the welder to insert the weld electrodes or welding gun into position and to permit the welding to be cleaned and visually inspected between weld passes. The presence of weld access holes also minimizes the introduction of stress concentrations and limits a connection's susceptibility to the development of triaxial stresses.

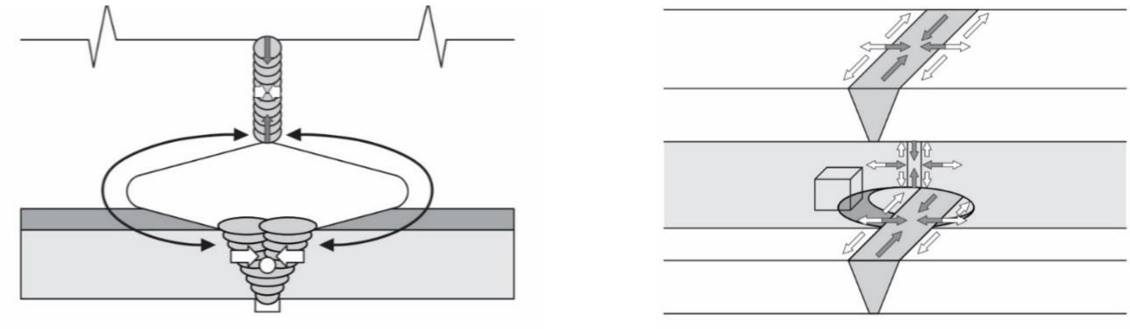


Figure 2-6. Weld access holes – tridimensional effects.

In the absence of guidance from European norms, access holes can be designed according to AWS D1.1/D1.1M:2015, Clause 5.16.1 [1] recommendations (see Figure 2-7), where:

- length L (1) > max (1.5 x web thickness; 38 mm)
- height H (2) > max (1.5 x web thickness; 20 mm) < 50 mm

In addition, AISC 360 Clause J1.6 has the following requirements for access hole:

- All access holes shall be detailed to provide room for weld backing as needed
- For sections that are rolled or welded prior to cutting, the edge of the web shall be sloped or curved from the surface of the flange to the reentrant surface of the access hole
- The access hole is permitted to terminate perpendicular to the flange, provided the weld is terminated at least a distance equal to the weld size away from the access hole.
- For heavy shapes, as defined in Sections A3.1c and A3.1d, the thermally cut surfaces of weld access holes shall be ground to bright metal.
- If the curved transition portion of weld access holes is formed by predrilled or sawed holes, then that portion of the access hole need not be ground

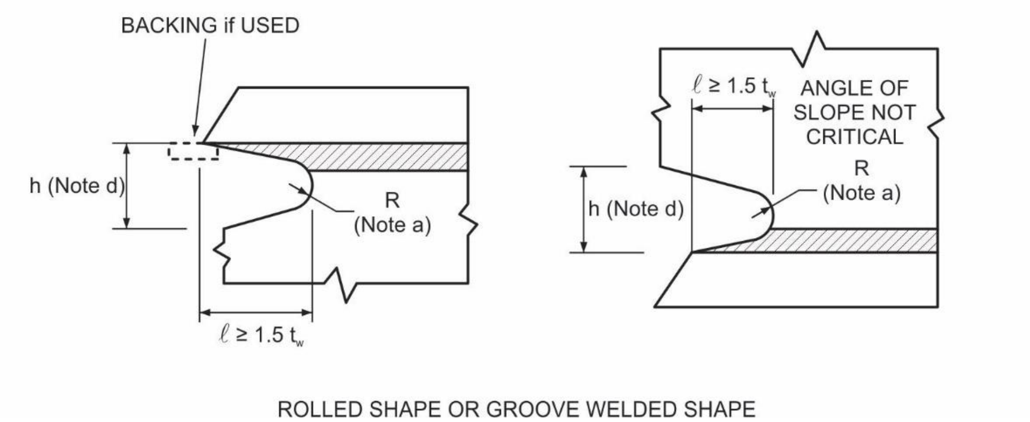


Figure 2-7. Weld access holes geometry – AWS D1.1/D1.1M:2015 [26].

AWS D1.1:2015 Table 3.3 specifies that A913 steel products (Grades 50 and 65 comparable to HISTAR® 355 and 460) can be welded with minimal to no preheat when the product temperature is above 0°C and when using a low-hydrogen electrode (< H8). Clause 3.5.1 of AWS D1.1:2015 mentions that the minimum preheat and interpass temperature applied to a join composed with different minimum preheats shall be the highest of the minimum preheats. For all types of steel, complex and highly restrained conditions may require preheating. Oftentimes, this can be avoided by careful coordination of the weld sequencing; however, if the special situation persists, a minimum preheat of 65°C is recommended.

AWS D1.1/D1.1M:2015, Clause 5.16.3 refers to weld access holes for heavy (jumbo and super-jumbo) shapes in which the curved surface of the access hole is thermally cut. Good practice is to apply a minimum preheat of 65°C to the base material prior to thermal cutting. The preheated area should extend 75 mm from the location where the curve is to be cut. For jumbo and super-jumbo sections, the thermally cut surfaces of weld access holes – and copes – shall be ground to bright metal and inspected by either magnetic particle inspection (MT) or penetrant testing (PT) methods prior to deposition of welds.

The surface quality should be made according to EN 1090-2 / ISO 9013. All surfaces subjected to heat-assisted processing are ground and inspected either by magnetic method or by penetration method, if specified.

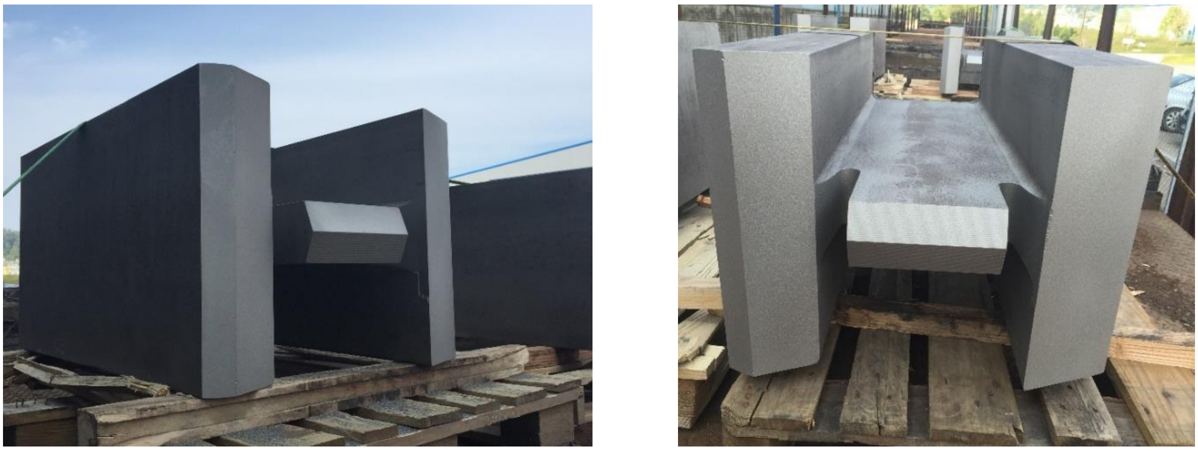


Figure 2-8. End preparation (bevel) of super-jumbo shape for CJP weld.

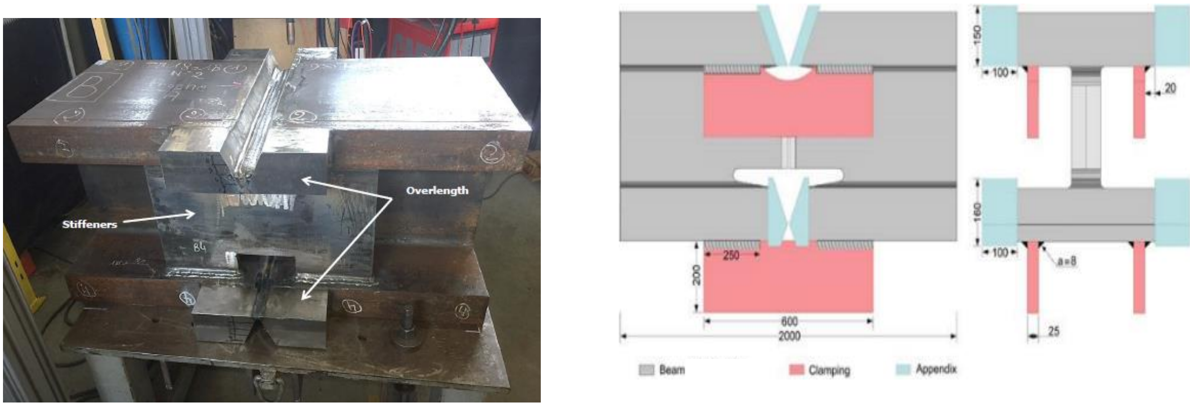


Figure 2-9. Beam splice, including fabrication aides.

Due to the amount of heat input of the arc process, expansion and the contraction of the weld metal and the adjacent base can lead to shrinkage of the weld and surrounding parent (base) metal. To minimize distortions created by shrinkage, three basic concepts can be employed: (1) reduce the localized nature of the heated base metal; (2) reduce stresses associated with the strains; and (3) reduce the volume of heated metal [22]. Some solutions for overcoming distortion issues include the use of temporary ties and stiffeners (to hold base material in place and prevent movement), as well as adjusting the weld sequence and/or number of weld passes. It is worth noting that stiffeners and temporary ties that are welded to the assembly are typically recommended to be of the same specification as the parent (base) metal, Figure 2-9.

The main steps of welding jumbo and super-jumbo shapes are summarized in Figure 2-10: when possible, in the first phase the flanges shall be welded alternatively and in the second phase the web shall be welded. Acc. to AWS D1.1, the welds shall be made in a sequence that will balance the applied heat of welding while the welding progresses, and all welding parameters and sequence must be in accordance with the qualified WPS.

For T-joints the following recommendation should be considered:

- Preparation of upper and lower flange at 2/3 & 1/3 of the thickness – keeping the 1/3 thickness on the outside for ease of gouging,
- Weld the web either at 2/3 & 1/3 or 1/2 & 1/2 of the thickness.

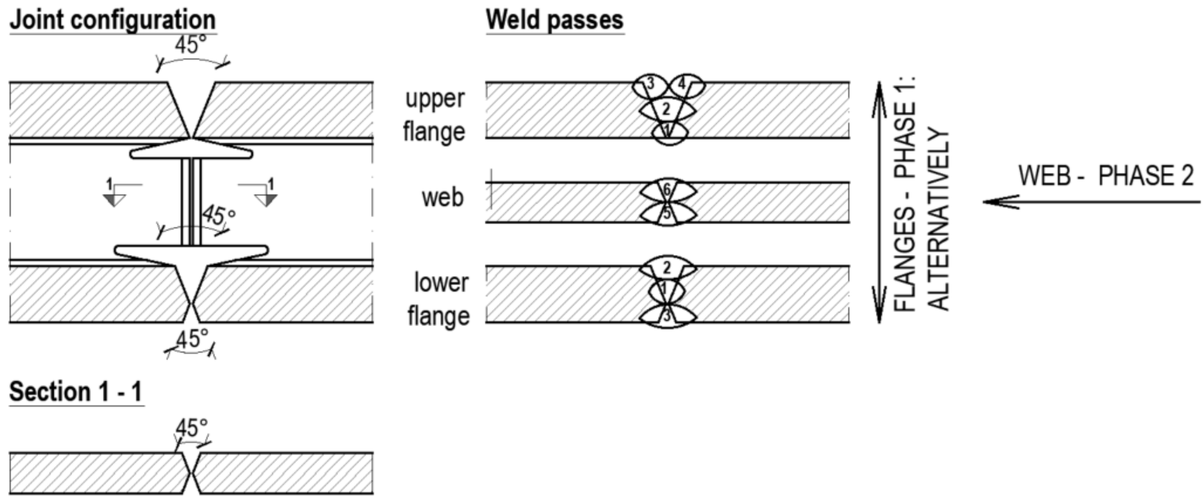


Figure 2-10. Recommended welding sequences heavy sections.

The main steps of welding jumbo shapes in North America per AWS D1.1 [1] are summarized in Figure 2-10. Phase 1: weld both flanges alternatively, respectively phase 2: weld the web. In Phase 1, the flanges are prepped on both sides for balance of weld around the neutral axis. Based on web thickness, the web is either prepped on one or both sides. The welds shall be made in a sequence that will balance the applied heat as welding progresses, and all welding parameters must be in accordance with the qualified WPS.

Material Preparation:

- Pre-heat to 65∘C prior to cutting and prepping flanges and web.
- Bevel flanges welded at 2/3 & 1/3 at 45∘C with root-face 3mm. Smaller bevel should be located at the outside of the flange for ease of back gouging.
- Grind each bevel smooth prior to fitting to remove notches, gouges, and sharp corners.
- Perform visual inspection to eliminate surface defects.

In general, when welding of Jumbo and Super-Jumbo Sections:

- Keep fit-up gaps less than 2mm max to reduce weld shrinkage.
- Use run-off tabs at each end for complete joint penetration weld (CJP).
- Make sure that the preheat temperature is above the specified minimum.
- Welding root passes in the flat or horizontal position is recommended.
- Clean each root pass and visually inspect it prior to depositing subsequent passes.
- Weld half of the large bevel first then back gouge (if applied) and weld the back side completely.
- Go back and finish the remaining weld at the large bevel.
- Weld the web after completely welding both flanges.
- Perform UT inspection prior to removal of run-off tabs and MT after removal of run-off tabs.

Figure 2-11. Welding progress of a butt weld jumbo beam per EN 1090.

2.2 Lamellar tearing

When shrinkage of welds is restrained in the through-thickness direction, a connection can become susceptible to lamellar tearing, which is a phenomenon defined as a separation initiating in the parent material, where the cause is identified as through-thickness strains induced by weld metal shrinkage.

Lamellar tearing typically results from a confluence of factors [10]:

- base metal properties though-thickness ductility
- design and detailing of connections
    - cruciform, T- and corner welds in combination with full penetration joints (i.e., highly restrained connections) represent connection types particularly vulnerable to this problem.
    - material thickness; thick material is more susceptible
    - large-groove CJP weld
- fabrication and welding:
    - full penetration T-joints with single or double-sided full penetration welds [30]
    - welding electrode, welding procedures and welding sequences

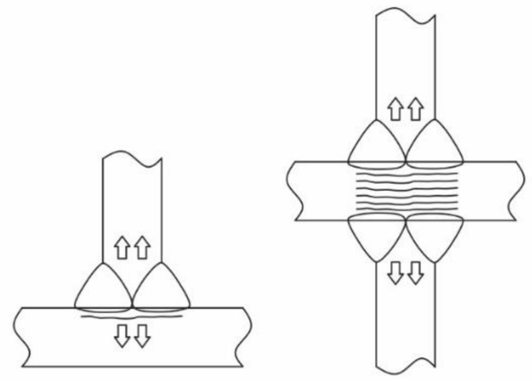


Figure 2-12. Lamellar tearing development.

T-joints rather than simple fillets rather than full penetration welds do not give such severe problems and less replies reported tearing with this type of joint than full penetration. In addition, balanced double – sided welds appear to present less risk than large single sided welds.

The modification which can be made to reduce the risk of tearing in a cruciform joint are:

- Replacement of a single-sided joint with a double-sided balanced joint. The volume of the weld is reduced, and the strains are balanced, but double-sided joints remain a risk situation Figure 1-2 a).
- Replacement of full penetration welds with simple fillet welds if all other design requirement allows the change
- Welds should not be larger than necessary; they might increase the risk of lamellar tearing, Figure 1-2 b).

To detect the lamellar tearing after welding, several methods are presented in the following *Table 2-2* with their potential problems above.

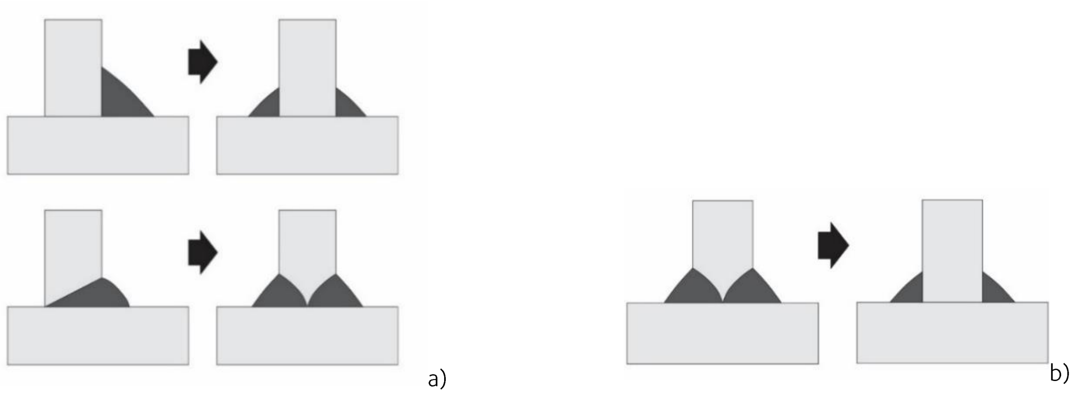


Figure 2-13. Lamellar tearing avoidance.

Table 2-2. Methods to detect lamellar tearing.

Method		Problems
Surface technics	Visual	Satisfactory for surface defects but not for subsurface tears.
	Dye penetrant	
	Magnetic particle	
Sub-surface technics	Radiography	Orientation of tearing usually makes this impossible
	Ultra-sonic testing	Access is sometimes difficult. Problems of interpretation, but it is the only practical &economical method for subsurface detection.

The most effective method of combating lamellar tearing is avoiding during the design and implementation of highly restrained connections; however, as that is not always possible, specifying a base material with sufficient properties in the through-thickness direction (e.g. ductility in Z direction), among other considerations, is often employed to mitigate the potential for lamellar tearing. Project stakeholders should be aware, however, that good materials and workmanship cannot overcome the drawbacks of a poorly designed or specified connection. Therefore, the best way to reduce the potential of this issue is by minimizing the concentration of localized strains, which can be achieved through the adoption of appropriate joint and welding details and by adopting procedures with proper weld metal for through-thickness connections.

Best practices for minimizing and identifying lamellar tearing issues:

- In steel assemblies where a plate is loaded in tension perpendicular to its plane, the material should have adequate through thickness proprieties to prevent lamellar tearing – EN 10164 (CEN, 2004e) [29].
- Modify the weld joint design so that lamellar tearing tendencies are minimized.
- Select low hydrogen diffusible electrodes where appropriate, as lamellar tearing tendencies might be aggravated by the presence of hydrogen, although not caused by it [24].
- Ensure that welding procedures are correct (by using joint simulations if necessary)

- In critical areas, carry out a detailed survey of the base material prior to welding. Retest after welding, using the results before welding to serve as a map of pre-existing defects (large inclusions, etc.). This enables pre-existing defects to be distinguished from lamellar tearing.
- Pay particular attention to the position of cracking in relation to the plate product and weld fusion, to avoid confusion with lack of penetration defects.

2.3 Establishing welding parameters.

2.3.1 Specifying a suitable welding process

The innovative manufacturing process of thermomechanical rolling in combination with Quenching and Self-Tempering (QST) enables HISTAR® steels to feature an optimized chemical composition and subsequently maintain low carbon equivalent values (CEV) compared to other structural steels of comparable yield strengths. A benefit of this is the potential to perform major welding processes with minimal to no preheat (see Figure 2-14 for comparison of HISTAR® preheat temperatures to those of other steel grades).

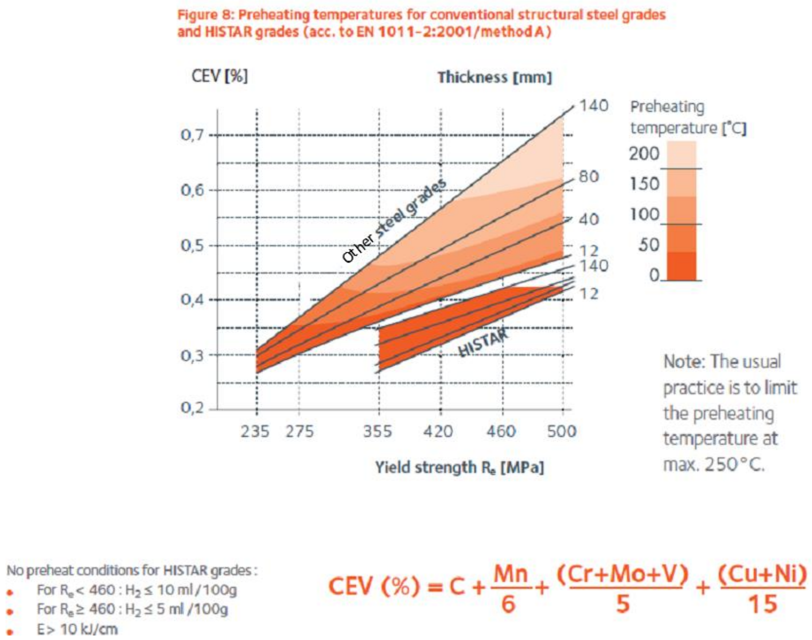


Figure 2-14. Comparison between preheating temperature for HISTAR® steels and other structural steel grades as a function of CEV and yield strength for varying thicknesses [15].

HISTAR® steel can be successfully welded by the most widely used arc welding processes Flux-Cored Arc Welding (FCAW) Shielded-Metal Arc (SMAW), Submerged Arc Welding (SAW) and Gas-Metal Arc Welding (GMAW), provided the general rules for welding are respected. However, the suitability of a specific weld process and welding procedure intended to be used should be established by the user in the WPS.

2.3.2 Specifying a suitable electrode.

The consumables for welding shall be selected to ensure that required mechanical properties of the welded joint can be achieved. The criteria to choose an electrode include the following: mechanical properties, chemical composition, and impact energy of the weld metal. Decisions on acceptability of the electrode should also take into account the base metal(s) on which they will be used. Though not addressed in EN standards, the AISC specification never requires overmatching properties for the filler metal; however, it is permitted to use filler metal with a strength of up to 70 MPa higher than that which would match the base metal [24].

In high-strength, low-alloy carbon steels, keeping diffusible hydrogen content in the welding consumable at a low level is crucial in preventing cold cracking which can undermine the integrity of the steel structure by effectively reducing its toughness and load capability compared to a sound weld. For shapes with flange thickness higher than 100 mm, the use of welding consumables containing diffusible hydrogen content less than 5 mL / 100 g of deposited metal (DM) is recommended.

Hydrogen cold cracking occurs primary in fast cooled zones, either in the weld metal and in the heat affected zone (HAZ) following several directions. The phenomenon is due to complex combined interactions between three factors: hydrogen atoms (diffused hydrogen H), brittle microstructure and the local tensile stress conditions on the weld. The elimination or the reduction of the effect of one of these factors will therefore reduce the noticeable crack propensity. The use of low hydrogen electrodes can be suitable solution to avoid or decrease the sensibility to cold cracking.

Controlled-hydrogen (< 5 mL / 100 g DM acc. to AWS D1.1 annex H6.2.2 and SEW 088 – 2017 [41]) flux cored-electrodes for non-alloy and fine grain steels used for welding HISTAR® JUMBO shapes are produced to meet the requirements of the standards. In the EN NF 17632-A [25] classification, the first letter "T" is the designation for flux cored electrode, the next two digits give indication on the minimum yield strength of the deposited metal. The next digit indicates the temperature in which the impact test gives minimum 47 J. The next three characters indicates the chemical composition of the weld metal, followed by one character indicating the type of electrode core, the shielding gas and the welding position. The last suffix "H" related to the diffusible hydrogen is followed by one digit designating the hydrogen content (5, 10 or 15 in mL / 100 g DM).

FCAW filler metals are always tubular, with a metallic tube that surrounds the internal flux. The diameter of such electrodes ranges from 0.8 mm to 3.2 mm, with 1.2 to 2.4 mm being typical for structural steel work. For HISTAR® JUMBO sections when no preheat is used to weld, the diffusible hydrogen content of welding consumables shall be lower than 5 mL / 100 g DM. T46 4 P M 1 H5, T46 2 P M 2 H5, are examples of electrodes that can be used to weld HISTAR 460® FCAW-G welding process.

Table 2-3: Choice of the welding consumables metals acc. to EN and AWS standards.

Welding process (EN ISO 4063:2010)			
SMAW (111)	MAG (135) GMAW (13)	FCAW (136)	SAW (121)
Standard (Designation)	Standard (Designation)	Standard (Designation)	Standard (Designation)
EN ISO 2560-A (E 42 3 *** H5)	EN ISO 14341-A (G 42 3 ***)	EN ISO 17632-A (T 42 3 **H10)	EN ISO 14174 EN ISO 14171
EN ISO 2560-A (E 42 5 *** H5)	EN ISO 14341-A (G 42 5 ***)	EN ISO 17632-A (T 42 5 **H5)	EN ISO 14174 EN ISO 14171
EN ISO 2560-A (E 46 3 *** H5)	EN ISO 14341-A (G 46 3 ***)	EN ISO 17632-A (T 46 3 **H5)	EN ISO 14174 EN ISO 14171
EN ISO 2560-A (E 46 5 *** H5)	EN ISO 14341-A (G 46 5 ***)	EN ISO 17632-A (T 46 5 **H5)	EN ISO 14174 EN ISO 14171

The choice of an electrode shall be in accordance with the choice of the welding process. Table 2-3 resumes the standards allowing a suitable choice of the welding consumables for different welding processes. Obviously, other choices may also be adequate, advices on commercial designations should be provided by the welding consumable producers.

Low-hydrogen electrodes and fluxes must be dry to ensure good performance. Manufactures typically supply electrodes and fluxes in moisture-resistant packaging. Once fluxes or electrodes are removed from the hermetically sealed container, they should be kept in a dry atmosphere (e.g. in a drying oven) to minimize or prevent the absorption of moisture from the atmosphere. The storage and use conditions of welding consumables provided by the manufacturer shall be applied. Extremely low hydrogen is present in the weld consumables for the GMAW welding processes.

Prior to any welding operation, groove faces must be free from impurities and defects that could affect the weld quality. Cleanness of the weld joint, in each phase, must be carefully made. Components must be clean and dry. Any rust, cutting slag, grease, paint (except for approved over weldable shop primers according to EN ISO 17652), moisture and any dust must be removed before welding. Concerning moisture, a preheating at a low temperature (50 to 80 °C) is recommended in order to dry the pieces to be welded, especially when the pieces are cold (condensation).

2.3.3 Setting the heat input.

The heat input during welding can be viewed as a main influencing factor affecting the properties of classical structural steels, including HISTAR®. Depending on the type of material, thickness of material and heat input, preheating and the maintenance of an upper- or lower-interpass temperature may be required. In multi-pass welds, the interpass temperature is the temperature in the weld area immediately before the second and subsequent pass of a multi-pass weld. Weldments on shapes of large cross-sectional area tend to dissipate interpass heat; therefore, the temperature tends to decrease with each weld pass, unless an external heating source is provided. For a HISTAR® jumbo or super-jumbo section, the interpass temperature shall be lower than 250°C [15]. Heat input is frequently used to directly estimate the amount of thermal energy that is introduced into a joint when the weld is made. The level of heat input serves as an indicator for the solidification and cooling rates that will be experienced by the weld metal and the heat affected zone (HAZ).

According to the EN1011-1 [43], heat input can be calculated with the following equation:

$$Q = \mathrm{k}\frac{UI}{v}10^{-3} \qquad \text{(eq. 1)}$$

$Q = \mathrm{k}\frac{UI}{v}10^{-3}$where:

Q= heat input (KJ/mm)
k= thermal efficiency (acc. to Table 1 EN1011-1)
U = arc voltage (V)
I= welding current (A)
v = travel speed (mm/s)

Table 2-4 indicates preheating requirements applicable for HISTAR® steel as a function of thickness, heat input and hydrogen content of the weld consumables. Otherwise, for consumables having higher diffusible hydrogen content (5 to 10 mL / 100 g DM) some preheating may be necessary, unless the heat input is in the range of 15 kJ/cm to 60 kJ/cm.

Recommendations for preheating temperatures of fine grain steels are given in NF EN 1011-2, in AWS D1.1, CSA W59. The preheating temperature is a function of the carbon equivalent, the thickness of the product, the hydrogen content of the welding consumables and the heat input. These recommendations apply to normal fabrication restraint conditions and welding at temperatures higher than 0 °C.

Table 2-4: Preheating requirements for HISTAR® (acc. to EN 1011-2)

Combined thickness [mm]	Hydrogen content of consumables [ml/100g]			
	5 -10		≤5	
	Heat input [kJ/cm]		Heat input [kJ/cm]	
	10 - 15	15 - 60	10 - 15	15 - 60
≤ 50	No preheat	No preheat	No preheat	No preheat
> 50	100°C	No preheat	No preheat	No preheat

From these recommendations and specific trials on HISTAR® 355 and HISTAR® 460 grades, the following preheating temperatures have been deduced:

- HISTAR® 355: no preheating required over the entire thickness range with diffusible hydrogen content of deposited metal ≤ 5 ml/100g and heat input values ≥ 10kJ/cm.
- HISTAR® 460: no preheating required over the entire thickness range with diffusible hydrogen content of deposited metal ≤ 5 ml/100g and heat input values ≥ 10kJ/cm.

2.3.4 Post weld heat treatment.

A post weld heat treatment (PWHT) is defined as any heat treatment after welding that can affect the properties of the weld and the base material [1]. All details related to the heat treatment shall be indicated in the WPS to take into account its effects on the mechanical properties of the base metal, the HAZ, and the weld after PWHT.

According to the design AISC guide [24], mostly welds on structural steel applications are kept in the as-welded condition and are rarely subjected to a post-weld heat treatment. Consequently, electrodes classified in the post-weld heat treatment condition should not be used for as-welded applications except if test data does exist to support their use in this condition.

The properties of HISTAR® as other structural steels, can be adversely affected by improper PWHT if the temperature exceeds those prescribed by the standards. As per EN10025-4: stress relieving at more than 580 °C or for over 1 h may lead to a deterioration of the mechanical properties of the steel grades. If the purchaser intends to stress relief the products at higher temperatures or for longer times than mentioned above, the minimum values of the mechanical properties after such a treatment should be agreed upon at the time of the order.

PWHT may be applied for stress relief when the layout of the structure and/or the expected stress condition after welding requires a reduction of the residual stress. Thermal stress relief is more frequently used to achieve dimensional stability in post-welding machining operations. Few structures are machined after welding, but components on moving bridges like gear racks may need to be stress relieved to avoid movement of the parts during machining [24]. The guidelines for stress relief procedure are given in SEW 088-2017, §5 and AWS D1.1., § 5.8. The procedures described in the following paragraphs have slight differences and nevertheless are applicable for HISTAR®.

Stress relieving or PWHT of HISTAR® steel grades is performed at temperatures between 530°C and 580° C. The holding time should be 2 minutes per mm of product thickness, but not less than 30 minutes and not more than 90 minutes. To assure the toughness of the welded joint, the cooling rate after stress-relieving must not be too slow. In the temperature range above 300 °C cooling rates from 50 °C /h to 100 °C /h are frequently used.

In case of stress relief according to AWS D1.1 clause 5.8, the stress-relief heat treatment when required, shall be performed by heat treating. Final machining after stress relieving shall be considered when needed to maintain dimensional tolerances.

Table 2-5. Minimum holding PWHT time as function of thickness acc. to AWS D1.1:2015.

Thickness (t_f) [mm]	$6 < t_f < 50$mm	$t_f > 50$mm
Holding time [min]	15 min for each 6 mm or fraction thereof	2 h plus 15 min. for each additional 25 mm or fraction thereof over 50 mm

The procedure for stress relief heat treatment acc. to AWS D1.1 shall be conformed to the following requirements:

1. The furnace temperature shall not be higher than 315°C at the time the welded assembly is placed in it;
2. Above 315°C the rate of heating shall not exceed 560°C per hour divided by the maximum metal thickness of the thicker part, but in no case more than 220°C per hour;
3. After a maximum temperature of 600°C is reached, the temperature of assembly shall be held within the specified limits in Table 2-5, based on the weld thickness. When the specified stress relief is for dimensional stability, the holding time shall be not less than the specified;
4. Above 315°C, cooling shall be done in a closed furnace or cooling chamber at a rate of no greater than 260°C per hour divided by the maximum metal thickness of the thicker part, but in no case more than 260°C per hour.

The minimum holding time to apply a PWHT for stress relief as function of the material thickness according to the AWS D1.1 is resumed in Table 2-5.

2.4 Welding suggestions for rolled Jumbo shapes.

An example of a designed CJP joint that could be chosen to perform a butt welding of a jumbo or super-jumbo shape, as well as a schematic detail of the assembly after machining, is shown in Figure 2-7. A double V-groove joint type and a X-groove joint or K-type the flanges; and a X-groove joint type in the web, with access holes at the transition between flange and web. Any other pre-qualified joint designs for jumbo shapes can be chosen if they are in accordance with the applicable standard and must be validated by the Welding Procedure Qualification (WPQ).

When a thermal cutting method is used for jumbo or super-jumbo shapes, special attention is required. According to AWS D1.1, Clause 5.16.3, it is recommended to preheat the area of beam copes and weld access holes to a temperature of no less than 65°C before cutting, extending the heated area 75 mm from the area where the curve is to be cut. For jumbo and super-jumbo sections, the thermally cut surfaces of beam copes and access holes shall be ground to bright metal and inspected by either MP or PT methods prior to deposition of welds.

According to AISC 360-16 clause M 2.2. recommends that thermally cut edges shall meet the requirements of AWS D1.1 clauses 5.14.5.2 and 5.14.8.3, with the exception that thermally cut free edges that will not be subject to fatigue shall be free of round-bottom gouges greater than 5 mm deep and sharp V-shaped notches. Gouges deeper than 5 mm and notches shall be removed by grinding or repaired by welding. Weld access holes shall meet the geometrical requirements of Paragraph 2.1. Beam copes and welds access holes in shapes that are to be galvanized shall be ground to bright metal. For shapes with a flange thickness not exceeding 50 mm, the roughness of thermally cut surfaces of copes shall be no greater than a surface roughness value of 50 µm as defined in Surface Texture, Surface Roughness, Waviness, and Lay (ASME B46.1), hereafter referred to as ASTM B46.1. For beam copes and weld access holes in which the curved part of the access hole is thermally cut in ASTM A6/A6M hot-rolled shapes with a flange thickness exceeding 50 mm, a preheat temperature of not less than 65°C shall be applied prior to thermal cutting.

The choice of the design joints and access holes shall be planned at the design stage that the weld joint is able to physically accommodate the welding process and the welding equipment (nozzle, torches, etc.).

Also, per AWS D1.1, Clause 5.16.1, weld access holes shall have a length (l) from the toe of the weld preparation not less than 1.5 times the thickness of the material in which the hole is made. The height (h) of the hole shall not be less than the thickness of the material or 20 mm. The height need not exceed 50 mm. It is required that the access hole be detailed to provide the deposition of sound weld metal and allow weld tabs to be placed as [1].

According to AWS D1.1, the welds shall be made in a sequence that will balance the applied heat of welding while the welding progresses, considering the qualified WPS. Some examples of welding sequence chosen to successfully perform welding of Jumbos shapes on are showed in Annex A: Example of butt welding with H5 of HD 400 x 1299 (tf = 140 mm) of HISTAR® 460

3 Connection classifications

3.1 Connection types and recommendations

Major connection types that are common in structural steel building systems are depicted in Figure 3-1:

- Beam splices (3);
- Beam – column connections (1; 2)
- Column splices (4)
- Lattice nodes (5)
- Truss-chord splices: compression elements (6); tension (i.e., traction) elements (7)

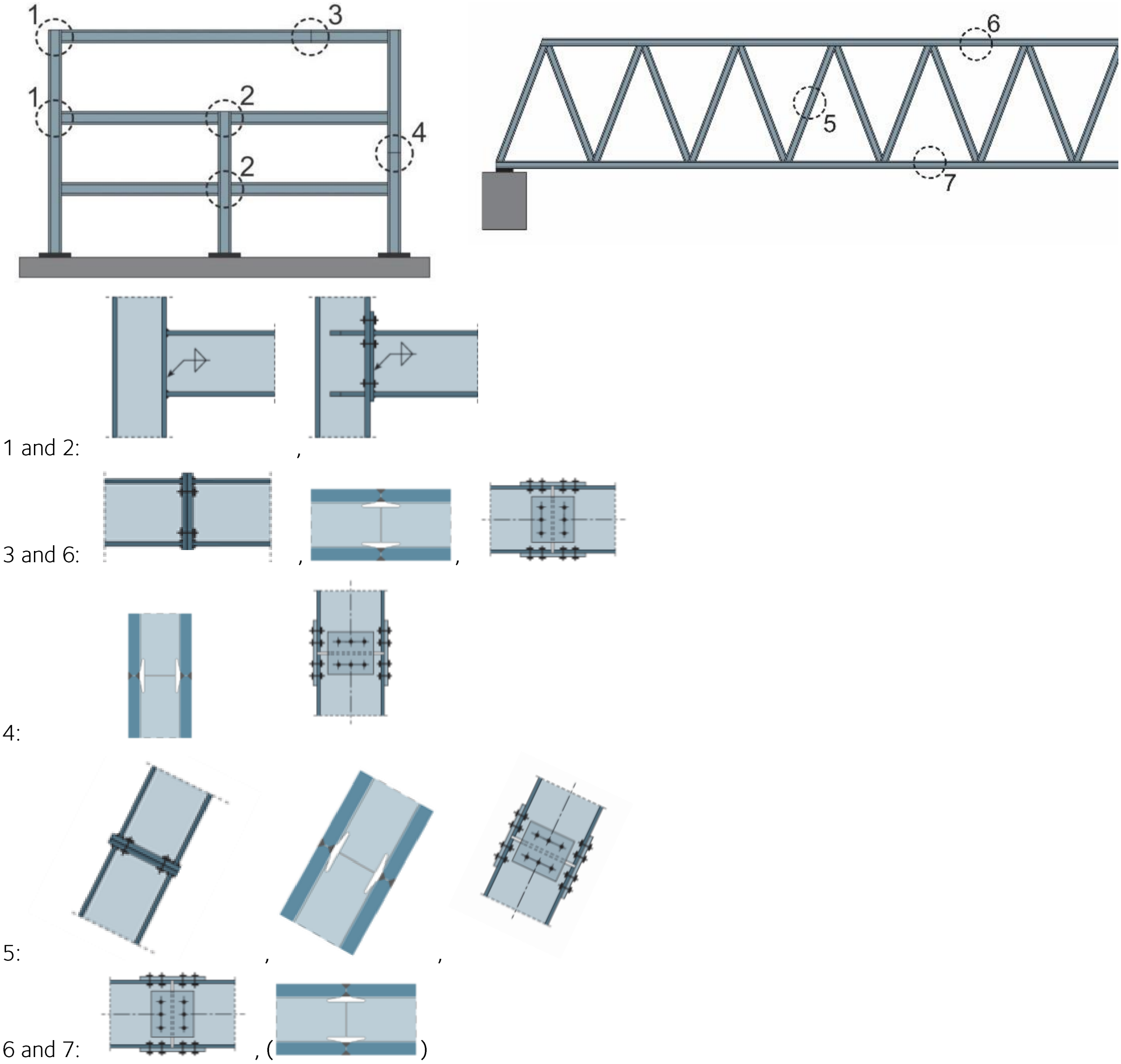


Figure 3-1. Types of connections in structures and recommended connections types in structures.

Simple connections transmit only a negligible moment and are considered to have sufficient capacity to accommodate the rotation that would be determined through analysis of the structure. Therefore, do not transfer significant moments at the ultimate limit state [34].

Moment connections are considered fully restrained (FR) or partially restrained (PR). FR connections transfer moment with negligible rotation between the connected members. The connections have sufficient strength and stiffness to maintain the initial angle between the connected members even at the strength limit state. PR connections transfer moment, but the rotation between the connected members is not negligible. The behavior of a PR connection is established by analytical or experimental means. The component elements of a PR connection must have sufficient strength, stiffness and deformation capacity at the strength limit [5].
The optimal load path is the one that maximizes a connection's capacity and minimizes its costs.
Best practices [33]:

- Design for the required forces: when designing splices that transfer a moment, the minimum axial load in the column may be used to offset the tension force induced by the moment.
- Avoid overusing complete joint penetration (CJP) welds, especially at field-welded column tension splices;
- partial joint penetration (PJP) weld connections are typically suitable to transfer applied forces due to no gap and allowing some direct bearing at the flanges.
- For gravity splices select steel shapes as W14 for a clean column line and to easily facilitate framing connections. The advantage of the W14 is that the inside flange to inside flange dimension is equal of all W14 columns and provides full-contact bearing at the thinner column flange.
- Consider end plate connections for compression members other than columns. AISC 360, Section J1.4 allows the splices to be proportioned for the lesser of either 1) a tensile force equal to 50% of the required compressive force; or 2) the moment and shear resulting from a transverse load equal to 2% of the required compressive strength of the member. The transverse load is applied to the splice location exclusive of other loads that act on the member.
- Weld multi-member joints in the shop and introduce a simple splice away from the joint. If three or more members intersect at one location, it is often more feasible to weld the joint or node in the shop for a better control of the geometry and assembly.
- For splices with complex geometries, perform trial fittings in or near the shop.
- Select the splice location so as to optimize the handling of components. Overhead crane capacity in the shop, size and weight limitations during transport, and crane access on-site can all influence the determination of an ideal location for a splice.
- When evaluating a column splice for erection purposes, an essential consideration is safety. OSHA 1926 Subpart R – "Safety Standard for Steel Erection" has some specific requirements to consider for column splices. 1926.756 "Beams and columns, Paragraph (d) Column Splices" states: "Each column splice shall be designed to resist a minimum eccentric gravity load of 300 lb (136.2 kg) located 18 in (46 cm) from the extreme outer face of the column in each direction at the top of the column shaft."
- To avoid conflict with the installation of safety cables, a column splice should be conservatively located 1.5m above the beam.
- The ideal welded splice is direct (i.e., shape-to-shape or tube-to-tube). A directly welded splice is made with groove welds (complete or partial, depending on the loading conditions) and eliminates the needs for gussets, knife edges, flanges and lap plates.
- Details includes backing, access holes and tabs should be optimized.
- Collaboration between the architect, engineer, fabricator, steel producer and erector is required to develop innovative "splicing systems" to make seemingly simple splices in a cost-effective and reliable manner.
- Though it served to improve the safety and behavior of steel connections, the post-Northridge earthquake splice led to welds that are more expensive to construct, as they require more weld metal to be used in execution of the full-penetration connection, they demand a greater number of weld passes, and back gouging is necessary to assure complete fusion of the weld root. Therefore, it is especially important for design and construction teams to explore cost reduction measures, such as the use of higher strength steels and/or those materials with better welding qualities, such as HISTAR®, to ensure that the economic impact on the project can be minimized.

3.2 Splice connections

"When the splice is right, the price is right"

3.2.1 Beam-to-beam splices.

In members predominantly subject to bending, splices are normally placed close to points of contraflexure, i.e. at points where the internal force from bending is minimized. The main reason for implementing a beam splice is as an aid to precambering or to achieve economy by reducing member sizes in regions of low moment.

Beam-to-beam splices can be welded or bolted, and though common perception is that welded connections are most suitable for tensile splices, bolted can be just as effective in that scenario. The most common types of splices when beams have the same serial size are shown in Figure 3-2.

Fully welded splices using full-penetration groove welds are typically executed in the workshop. On site, an alternative splice featuring cover plate(s) (single-sided or double-sided) may be used. The alternative connection may be preferred on site over fully welded butt welds, as it provides more flexibility to accommodate dimensional issues related to tolerances.

When properly executed in terms of electrode selection and implementation of the correct welding procedures, a connection comprised of a full-penetration groove weld should be at least as strong as the base metal.

When splices use bolted connections, high strength bolts are commonly employed as they can help reduce the splice length. Preloaded (or pretensioned) bolts, which induce a compressive (or clamping) force in a bolted joint, can improve joint rigidity and help to limit deflections due to their ability to prevent slip. Such a consideration can be of major importance where serviceability conditions are a major factor in the beam design.

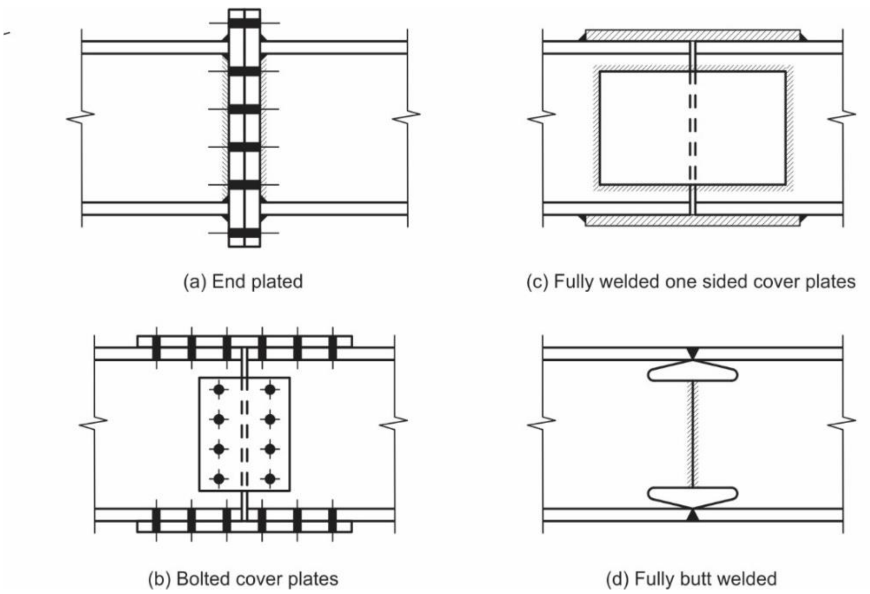


Figure 3-2. Types of splice arrangements.

In splice connections that are not executed via full-penetration groove welds, it is common to use plate components to transfer forces between the connected elements. Existent as cover plates on one or both flanges, as well as web plates on both sides of the webs, the design of these elements is typically based on the static theorem of plastic design. Reference is commonly made to a statically admissible force distribution, where the bending moment is resolved into a pair of equal but opposite flange forces while the web provides resistance to shear forces only.

In some situations, architectural considerations – such as tight local headroom requirements – may demand that a splice take place between two beams of different serial size. In these situations, it is still possible to utilize the aforementioned detailing options, provided due allowance is made for appropriate packs and that web stiffeners are employed, as required, to diffuse flange forces.

As a best practice, it is recommended to locate beam splices at a region of little to no internal moment in the span. This enables the connection to be designed without special consideration of rotational demand. However, in rare occasions, it may be necessary to accommodate a splice a location of significant internal moment. In such situations, it is important to design and detail the splice connection such that it has a rotation capacity that is consistent with the global analysis of the overall structure.

3.2.2 Column-to-column splices.

Typical erection practices dictate column-to-column splices at every second level of a building (i.e., two-story column lifts are common practice.) This usually corresponds to a change in size of the section and results in a field splice being executed on site. Usually this change in section is made at every second level, where a shop or field splice is located.

With respect to moment resistance, a conventional column splice typically develops 20% of the moment capacity of the column. Common column-to-column splices are presented in Figure 3-11 and Figure 3-12.

Research completed after the Northridge earthquake primarily addressed welded beam-to-column connections, as a clear majority of fractures were observed in these connections. However, the research led to broader findings regarding the fracture susceptibility resulting from the combination of details with sharp flaws and brittle materials. These findings resulted in updated design requirement for other types of connections, including column splices. The main difference between pre- and post-Northridge connections details is that the post-Northridge connection detail employs CJP welds in the flanges and webs, in combination with a weld access hole, to mitigate risks from unfused weld roots and stress concentrations at a critical point in the connection. Figure 3-3 shows such use of weld access holes in the practice of column splice details.

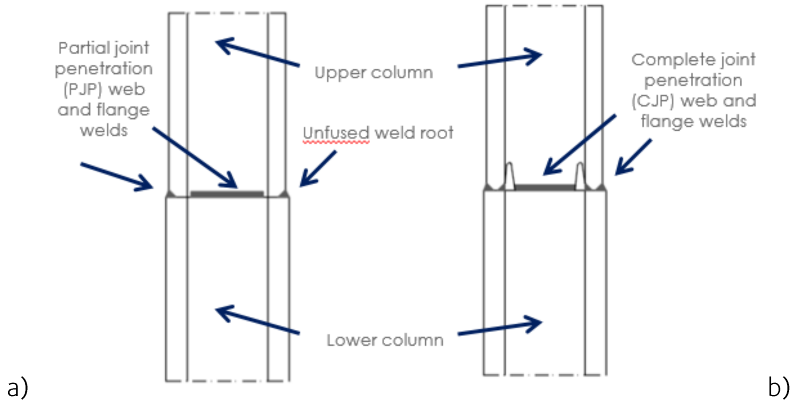


Figure 3-3. Typical CJP column splice construction details.: a) Pre-Northridge earthquake b) Post-Northridge earthquake.

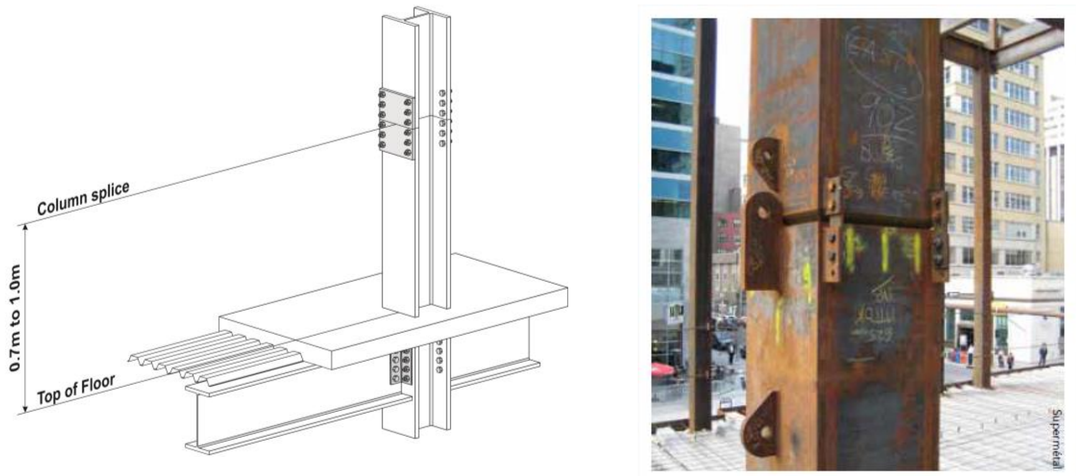


Figure 3-4. Splice connections – interrupted zone [33].

Generally, splice locations should optimize the handling and erection of components. As a result, splices are typically located between 0.7 m to 1 m above the floor above floor-beam connections - see Figure 3-4. Because column stresses are transferred from column to column by bearing, splice plates are generally of nominal size, commensurate with the need for safe erection and bending moments to which the joint may be subjected during erection. A clean column line splice along with the splice alignment and temporary support should be provided. To control costs, complex nodes are usually welded in the shop, while bolting of connections are preferable on site.

Figure 3-5. Splice connections – complex node [33].

3.2.3 Case studies by load type

3.2.3.1 Connections submitted to tension loads.

Tension members are generally more slender and feature a smaller cross-sectional area when compared to compression members. Tension members that are likely to be spliced include the following: truss elements (e.g., tension chords and web members); lattice girders; tension braces; and other bracing members. Most splices in tension members use splice plates and overlapped connections; however, butt-welded splices are executed in special circumstances (see also Paragraph 2.1). End-plated splices are infrequently used, but when they are it is typically for hollow sections.

Any type of structural shape is appropriate for tension members. For large tensile forces, H and I sections can be used [31]. Tensile forces are distributed proportionally across a member based on the cross-sectional areas of the tension member's component parts, or planar elements (i.e. planar elements of an "I" section would be the flanges and the web). The splice plate(s) associated with each planar element should be designed to resist the relevant tensile force component.

Bolted tension splices utilize either: (1) bearing bolts, where no special consideration is given to slip in the splices; and/or (2) slip-critical (i.e., slip-resistant or friction-type) bolts, which prevent movement in the splice under service conditions and, in some cases, at the ultimate limit state of the connection. Friction-type bolts provide the joint and the tension member with a larger extensional rigidity.

In bolted connections, steel is removed from the member to accommodate placement of the bolts. This characteristic of the connection results in a reduction in the member's ultimate strength. In design, the impact of the presence of bolt holes is assessed by evaluating the net section.

Though it is best practice to detail a splice such that eccentric loading is avoided, in some cases this is not possible and, consequently, the tensile load must be transmitted eccentrically. When the load eccentricity is small and the resulting bending effects are considered negligible, the phenomenon of plastic redistribution is relied upon to resist secondary bending and, as a result, the eccentricity need not be explicitly accounted for in the ultimate strength calculations of both the member and the splice, meaning that they will instead be designed to resist only pure tensile loading. When the eccentricity of the force is not negligible, then either a conservative approximation should be made for bending effects or an explicit account should be taken of the bending effects when designing the splice.

It is often the preference of steel fabricators to perform welded splices in the shop, whereas erectors tend to prefer using bolted splices [32]. Following are several examples of details for bolted connections that were required to transfer large truss loads.

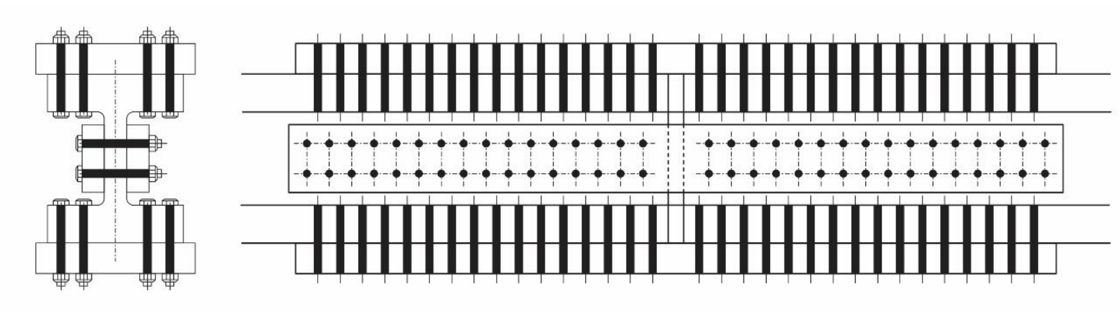


Figure 3-6. Bolted connection required to transfer large truss loads.[32]

Example 1: Epic, Deep Space Auditorium in Verona, Wisconsin; Vakaris Renetskis/Thornton Tomasetti; Cuningham Group Architecture, P.A. (Architect) & J.P. Cullen & Sons, Inc. (Contractor), LeJeune (Fabricator).

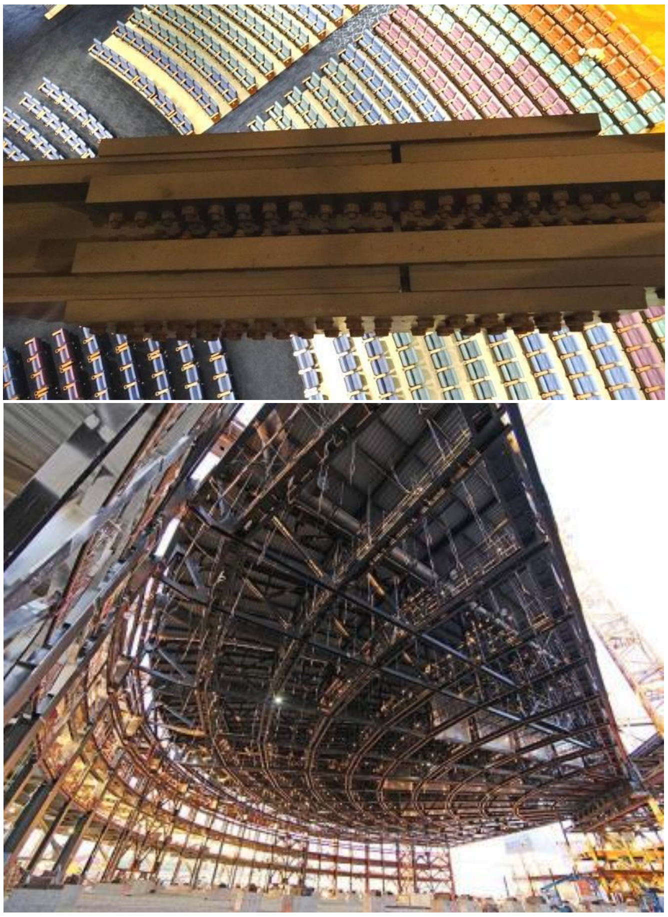


Figure 3-7. Bolted connection - Epic, Deep Space Auditorium in Verona, Wisconsin (credits Robert Caroti/ArcelorMittal International and Vakaris Renetskis).

Example 2: MARINER STADIUM, Baseball Park, Seattle; Total tonnage: ~ 25 000 t, Roof tonnage – span length 200 m: ~ 12 000 t; Fabricator: Supremek. Lower chord section in tension: HD 400x1086 in HISTAR® 460.

Figure 3-8. MARINER STADIUM, Baseball Park, Seattle – retractable roof.

Example 3: BANK ONE BALL PARK Phoenix – Arizona, Fabricator: Schuff.

Figure 3-9. Bolted connection - BANK ONE BALL PARK Phoenix – Arizona.

3.2.3.2 Connections submitted to compression loads.

Columns, struts, bracing elements, and select web members in trusses and lattice girders are amongst the most common compression members. Due to their tendency to buckle, compression members are normally stockier than tension members.

Splices in compression members may use arrangements quite like those for tension members. Cover plates and fasteners should be designed based on similar guidelines and design rules, with special considerations for localized effects such as buckling in the plate due to the accommodation of compression forces.

In contrast to tension members – where fasteners serve as an integral part of the load path - compressive loads can be transferred through direct bearing of adjacent ends of the compression members that are being spliced. Provided they are flat within appropriate tolerances, the faces of compression members in these types of connections do not need to be machined. Such tolerances can usually be satisfied by cutting with a milling saw to tolerances as mentioned in EN1090 – 2 Table 6 clauses 1,5 and 6, respectively AISC 360, Chapter M (Sections M2 and M4.

Figure 3-10 presents edge preparation for connections by contact while Figure 3-11 presents various situations in which a connection could be made by contact (bearing connection).

Figure 3-10. Edge preparations for connections by contact.

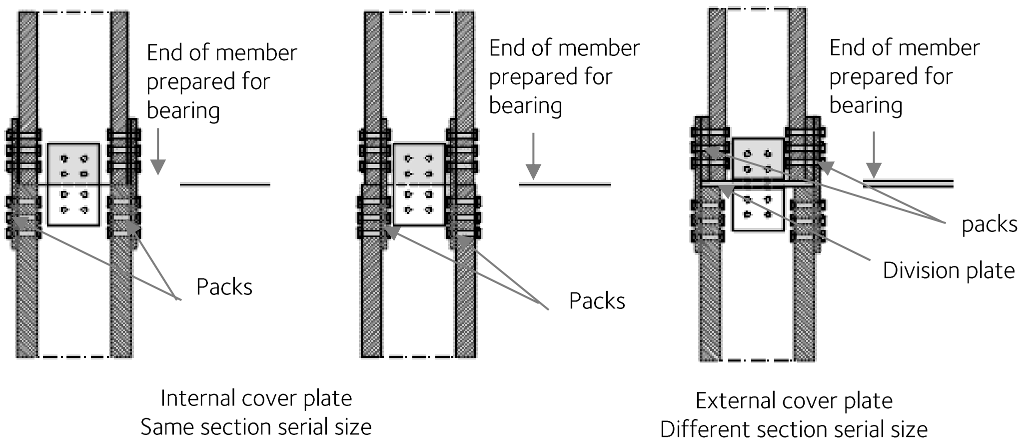


Figure 3-11. Examples of connections by contact – bearing splices for rolled sections.

Example 1: STADSKANTOOR – UTRECHT; Design: Kraaivanger architecten and Zonneveld Ingenieurs; General contractor: Boele & van Eesteen and G & S Bouw; Workshop: ASK Romein - Oostingh Staalbouw

Structural characteristics: Height 92m; 22 floors - +/- 65000m2; +/- 8000 t of sections; +/- 5000 t of HD columns in HISTAR® 460

Figure 3-12. Connections submitted to compression loads - STADSKANTOOR – UTRECHT.

Splices are joints made within the length of any structural member. They should be designed to allow for the transfer of stress resultants existing in the joint, with due allowance for second order effects, imperfections, and load eccentricities. Usually this transfer is made with transitional plate elements and fasteners Figure 3-13.

Load paths through splice components and fasteners should be correctly identified and load components proportioned to satisfy equilibrium. Packs may be necessary to compensate for fabrication tolerances and changes in serial size.Welded splices are usually executed in the workshop, while bolted splices are most commonly executed on site. Splices executed on site must allow for limited dimensional control.

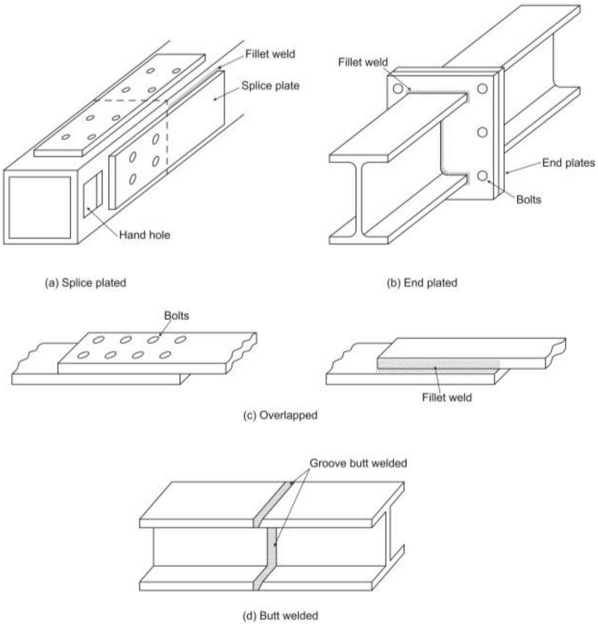


Figure 3-13. Types of splice arrangements

The principal action on a bolt in a splice joint of the type shown in Figure 3-14 is shearing on its cross-sectional plane, which is caused by bearing between opposing plates in the joint. The elastic distribution of these bearing stresses and the stresses produced in the bolt are complex. However, for fully developed plastic conditions, the distribution of shear stress is effectively uniform so that the shear strength is the product of the cross-sectional area of the bolt in the shear plane multiplied by the shear strength of the material.

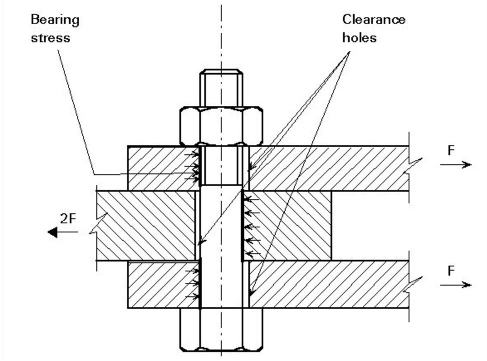


Figure 3-14. Load transmission in a bolted splice joint.

AWS D 1.1/D1.1M: 2015 Clause 5.19 states that all welded subassemblies splices in each component part of a cover-plated beam or built-up member shall be made before the component part is welded to other component parts of the member.

AISC 2016 Chapter J6 states that groove-welded splices in plate girders and beams shall develop the nominal strength of the smaller spliced section. Other types of splices in cross sections of plate girders and beams shall develop the strength required by the forces at the point of the splice.

3.3 Simple connections

Three types of simple connections are recommended to connect a beam ("supported member") to another beam or a column ("supporting member") [34]:

1. Header plate [aka end plate] connections - Figure 3-15

A typical header plate connection is composed of a steel plate, two lines of fillet weld (i.e., along both sides of the web of the supported beam), and two vertical lines of bolts (single or double) (see Figure X). The plate is of a height that does not exceed the clear depth of the supported beam web and is typically welded to the supported member in the shop then bolted to the supporting member in the field. The end of the supported steel beam may be un-notched or double notched.

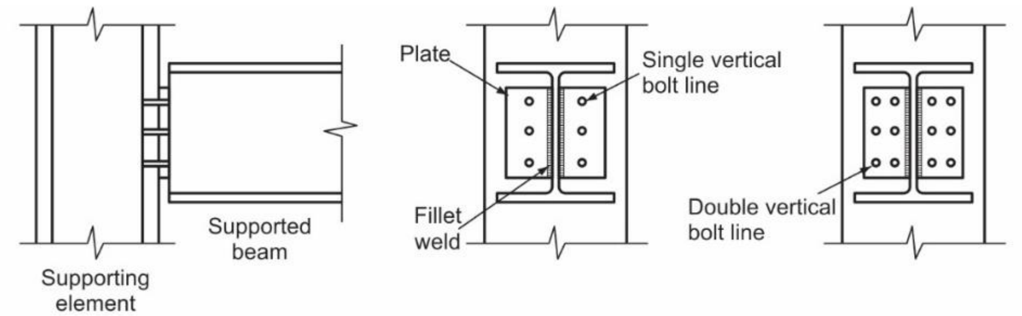


Figure 3-15. Header plate connection.

2. Fin plate [aka shear tab] connections - Figure 3-17

A typical fin plate connection is composed of the fin plate, two lines of fillet weld (i.e., along both sides of the fin plate at the supporting member), and a vertical line of bolts (single or double). The plate is welded to the supporting member (i.e., the column). The end of the supported steel beam may be un-notched, single notched or double notched, Figure 3-16.

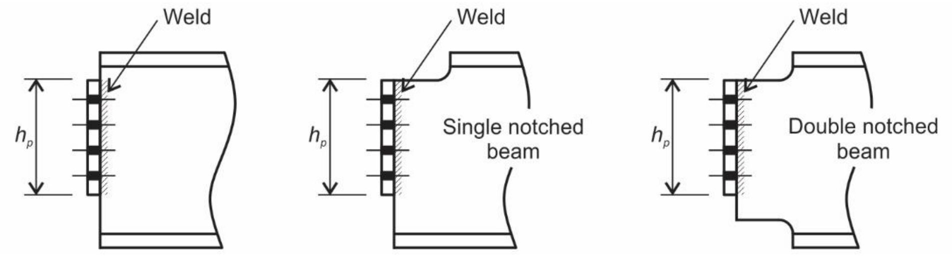


Figure 3-16. Fin plate connection – end of supported beam.

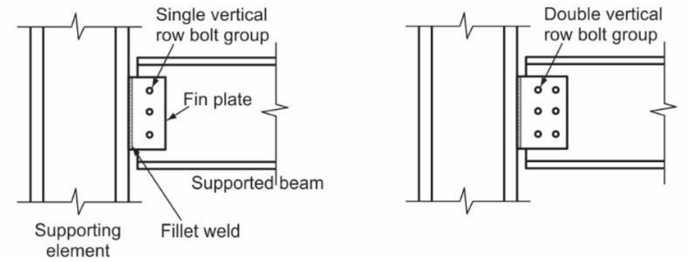


Figure 3-17. Fin plate connections.

3. Web cleat [aka double-angle] connections- Figure 3-18

A web cleat connection is characterized by two web cleats and their single double vertical bolt line (two on the supporting element and one on the supported member). The cleats are bolted to the supporting and supported members. Un-notched, single notched or double notched supported beams may be considered.

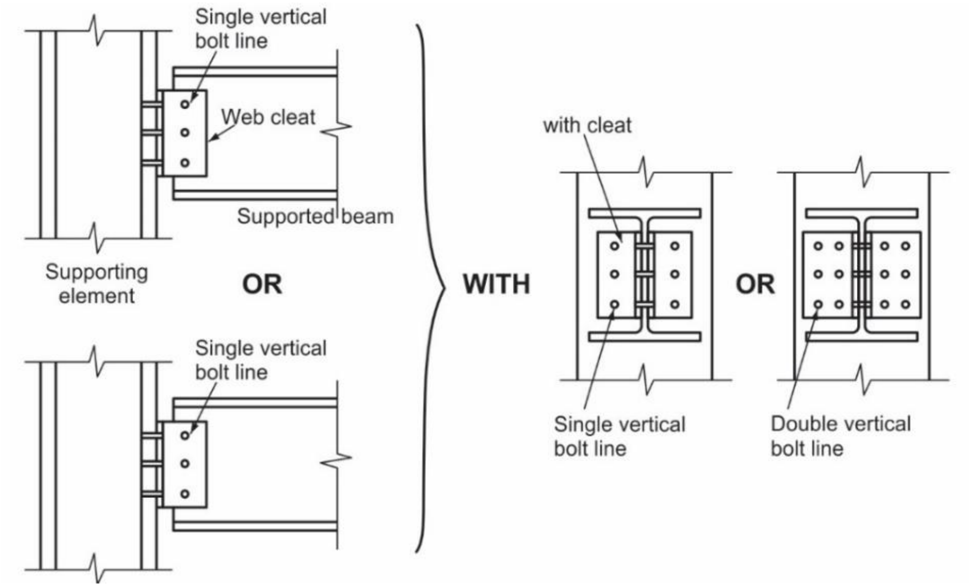


Figure 3-18. Web cleat (a.k.a double angle) connections.

Traditionally, other types of beam to column connections are considered, in the Figure 3-19 you can find examples of semi-rigid joints.

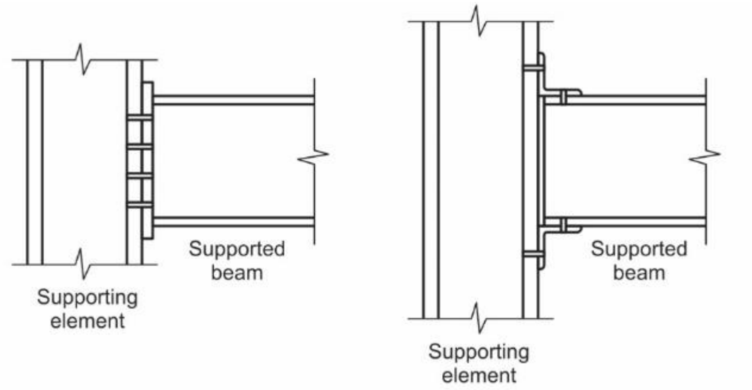


Figure 3-19. Other simple beam to column connections.

3.4 Joints in lattice structures

While dealing with large trusses, collaboration between architects, engineers, contractors, fabricators and erectors is key to optimizing the location and types of splices, the method of cambering, the member types and the assembly of the trusses from start to finish.

Example 1 New Pacific NW Pacific Ballpark: design office: Skilling Ward Magnusson & Barkshire, Fabricator;

- Steel HISTAR® grade 65, steel shapes: W 14x730; Mega trusses of 212 m;

Figure 3-20. Example of jumbo sections used in trusses connections: a) New Pacific NW Pacific Ballpark; b) Chase Field (Formerly Bank One Ballpark) in Phoenix, AZ. Fabricator: Schuff.

Figure 3-21. Example of jumbo sections used in trusses connections: Chase Field (Formerly Bank One Ballpark) in Phoenix, AZ. Fabricator: Schuff.

4 Connections – best practices.

The configuration and design of connections are driven by a variety of considerations including architectural requirements, contractor (i.e., fabricator and erector) preferences, and delivery logistics. Herein are identified several influencing factors for which advisable best practices for addressing – often rooted in code recommendations – are supplied for reference.

1. Weight and length limits

Connection solutions are primarily driven by the hot-rolled sections selected to meet the loading demands and configuration of a structural system. An optimized and practical design, however, will also consider fabrication and erection preferences and limitations, which are influenced by supply chain standards (e.g., the general length for a steel beam is 12 m), handling considerations (e.g., transport restrictions from truck loading capacities, or lifting limits on cranes), and construction efficiencies (e.g., columns are usually specified with the same cross section for two or three floors to limit the number of required splices). Properly accounting for both design demands and fabrication and erection preferences and limitations will result in an appropriate optimization of element lengths and weights and a simplification of connection solutions. Common limits in dimensions and weights of selected steel shapes are summarized in Table 4-1.

Table 4-1. Dimensions and weights for several types of steel shapes.

Section	Weight [kg/m]	Weight [lb/ft]	L_{max} [m]	Tonnage
HD 400 W 360x410	818	550	32.8	26.83
	900	605	29.5	26.55
	990	665	26.4	26.14
	1086	730	23.7	25.74

	1202	808	21.1	25.36
	1299	873	19.2	24.94
HL 920 W 920x420	970	652	27.1	26.29
	1077	723	24.0	25.85
	1194	802	21.2	25.31
	1269	853	19.7	25.00
	1377	925	17.9	24.65

2. Selecting steel shapes for columns

In the specific case of columns, it is common practice to select elements that simplify execution of splice connections. All HD profiles have the same web height, which enables full-contact support between members; as a result, specifying HD sections for columns not only enables a direct descending line for gravity loads (i.e., minimized load eccentricities), but it also provides a facility for simplifying erection, especially in the case of welding. In a parallel to the benefits of HD sections, HL sections carry an advantage of having constant depth between fillets "d" – ANSI/AISC 358-16 manual provides a high range of predefined connections that can be used in this situation.

3. Material specification

Figure 4-1 presents the advantage of using HISTAR® 460 material over S355 grade. By employing a steel of higher yield strength, the section size can decrease, which is accompanied by a decrease in thickness of component elements. For example, in a case where an HD 400*677 (S355) can be substituted with an HD 400*463 (HISTAR®460), the thickness of the flange is adjusted from (5h) to (3h). If the design demands a complete joint penetration weld, the change in thickness translates to significant reductions in the volume of the weld, which translates to a large decrease in the number of weld passes, consumables, welding time, namely a 31% reduction. The reduction in passes translates to a savings in weld material and in fabrication and/or erection time.

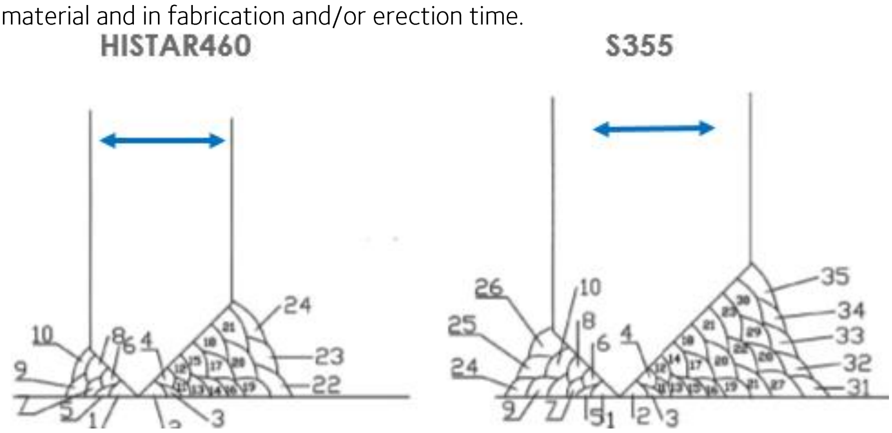


Figure 4-1. Weld passes through thickness

4. Assembly type and its application

Complex nodes require specialized sequencing (i.e., order of placement) for their structural members. The most complex components of the connection should, where possible, be performed in workshops. This typical amounts to complex welded connections occurring in the workshop and more simple bolted connections occurring on site – see Figure 4-2, which features an example of such a complex system. In this assembly the contact bearing was machine finished, CJP welds were executed in the shop, and bolting was used for connections in the field.

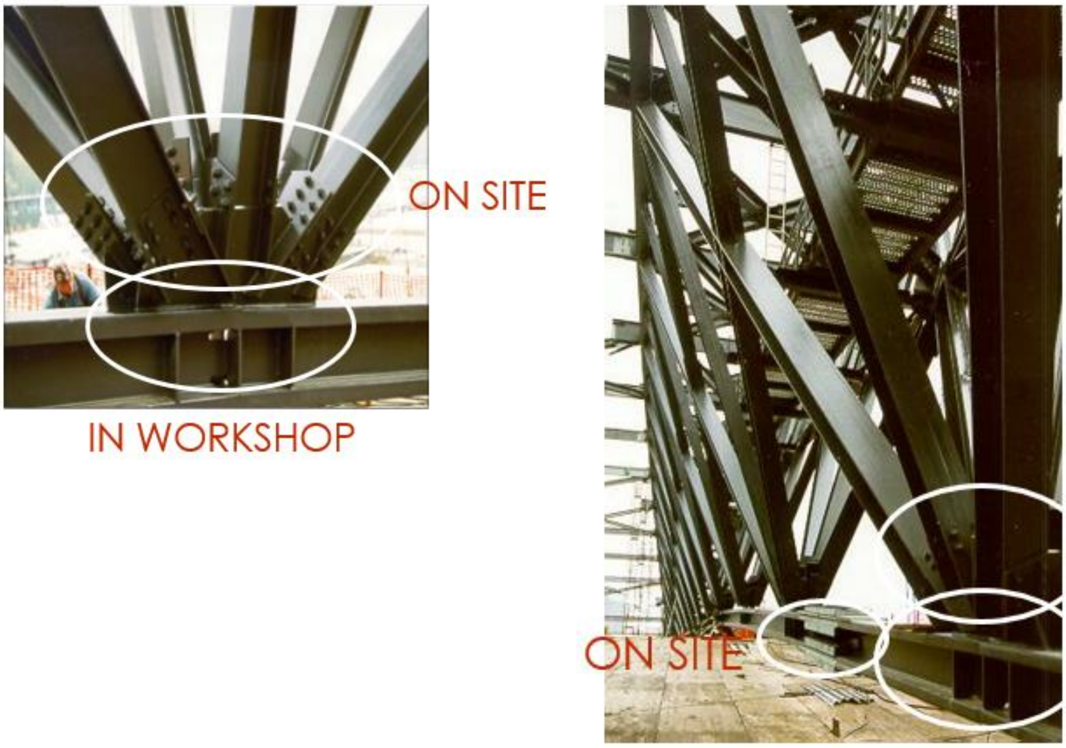


Figure 4-2. Complex connections- specifying the proper place to realize the connection.

5. Limit the specification of full penetration welds

Full-penetration (or complete-joint penetration) welded connections can add complexity to fabrication and erection processes, thereby resulting in higher construction costs and elevated risk on the project. Specific contributing factors include special detailing requirements (e.g., the chamfer of a CJP weld needs supplementary restraints during erection and before welding); ductility concerns (CJP connections are generally less ductile than PJP connections dependent on loading directions); and the high number of weld passes – see Figure 4-3(one weld pass is comprised of depositing the weld, Figure 4-3a; slag removal, or cleaning of the weld, through use of a chisel, Figure 4-3b, or grinding, Figure 4-3c; and performing any necessary quality control procedures, Figure 4-3d, (e.g.., visual inspection, temperature checks, temperature control, etc.)). To reduce costs, use less material, and limit testing requirements, it is advisable to limit the specification of CJP connections and, wherever possible, use PJP connections to transfer the applied forces between connected elements. The overspecification of CJP connections is particularly common in connections submitted to tensile loading. While they may sometimes be more cost effective when the connection requires development of the full tensile capacity of the connected elements, in other situations, PJP connections may suffice.

Figure 4-3. Weld passes through thickness.

6. Visual aspects

For welded connections, two levels of visual acceptance can be considered: performance and aesthetics. Performance-focused visual acceptance is based on visual evaluation of the weld for imperfections that may affect the weld's effectiveness and complemented by a desire to demonstrate good workmanship. Properly welded connections will have a visually pleasing appearance. Total grinding of the weld may reinforce the aesthetics of the welded joint, but they are expensive. For small imperfections, local grinding is recommended. AWS D 1.1 /D1.1M: 2015 Clause 5.23 [1] presents visual acceptance criteria as it relates to performance-focused issues, as does AISC 360 Chapter N. Highlights of the AWS requirements follow:

- All welds shall be free of cracks and overlaps.
- Where welds are required to be flush, they shall be finished so as not to reduce the thickness of the thinner base metal or weld metal by more than 1mm. Remaining reinforcement shall not exceed 1 mm in height and shall blend smoothly into the base metal surface with transition areas free from undercut. All reinforcement shall be removed where the weld forms part of a faying or contact surface.
- Where surface finishing is required, surface roughness values shall not exceed 6.3 micrometers. Chipping and gouging may be used provided these are followed by grinding or aching.

Visual acceptance as it relates to weld aesthetics is oft based on architectural requirements for a project. A major consideration that is related to aesthetic-focused visual acceptance is the distance from which people will view connections. Any "reworking" of visible joints at more than 5-6 m is generally unjustified and only increases costs on the project [44].

7. Safety

For a column joint, safety must be considered, especially during erection. To ensure ease of access, the column splice (assembly point) should be located at +/- 1,5 m the beam. Such a detail also allows for the installation of safety equipment (e.g., safety cables (lifeline) and perimeter cables) before the next level is installed, and easy access for on-site quality management.

8. Pre-trial test assembly for complex geometrical connections in workshop

Trial assembly is advisable for "complex" connections and may even be requested in tender documents. It is advisable as a best practice, as this practice erection of elements in the workshop can reduce erection time on site; identify imperfections, thereby enabling them to be resolved; and mitigate other risks.

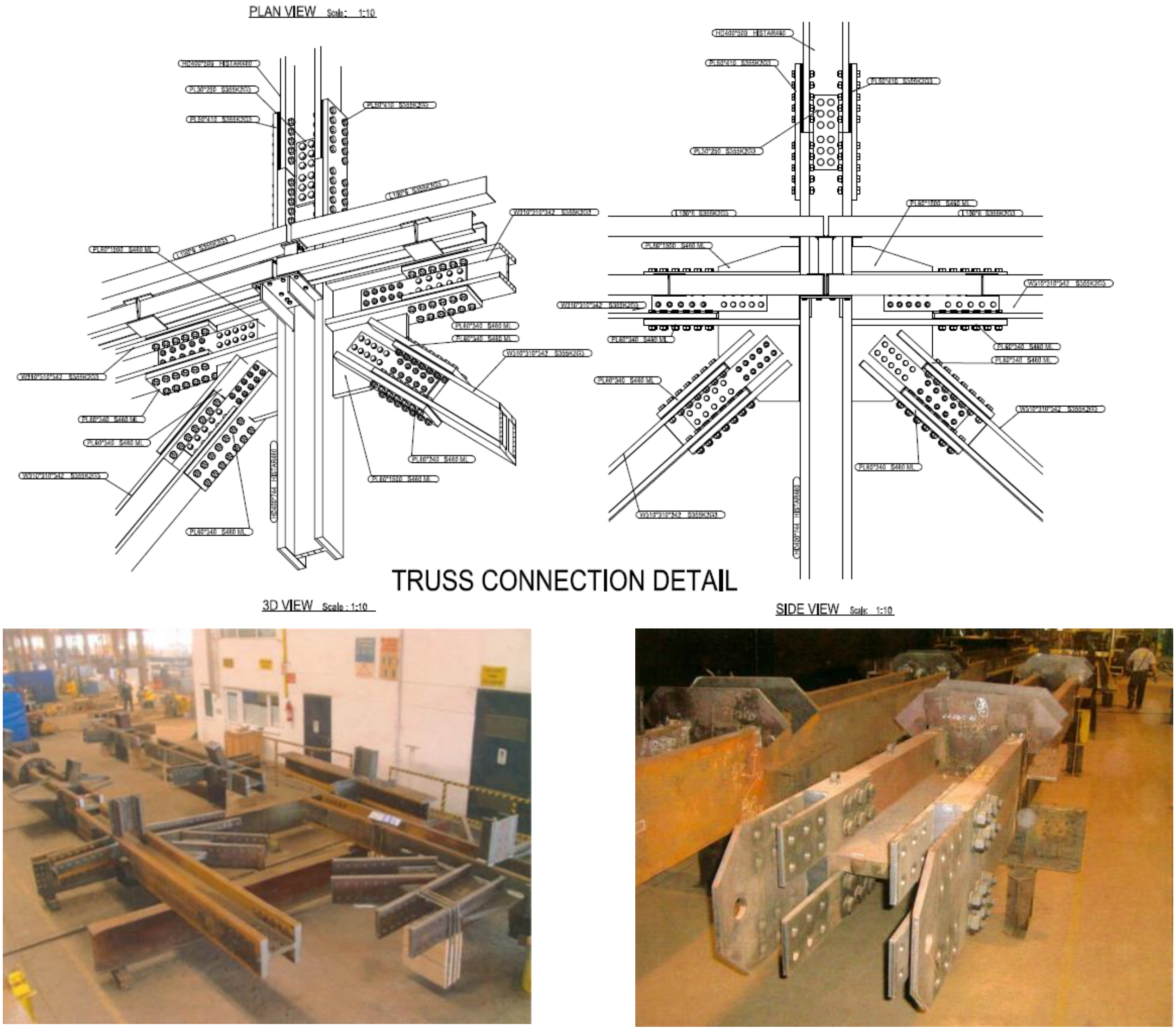


Figure 4-4. Truss connection detail.

9. Transport

Particular attention should be paid to the transport conditions and accessibility of the workshop and site. Methods of transport are diverse and subject to special considerations, as follows:

- Truck - typically restricted to length standards and/or restrictions, depending on country and truck type.
- Train - particularly attractive for exceptional transport - Figure 4-5 and *Figure 4-6*.
- Ship – when utilizing containers, lengths are limited to approximately 12m.
-

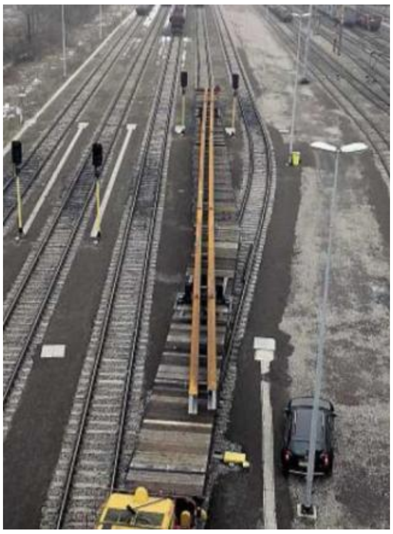
 

Figure 4-5. Transport of 60 m steel shape – Dresden site.

Figure 4-6. Steel connection – Two Manhattan West tower [48].

10. Fabrication and erection

In the selection of connections, the designer should observe the following principles [36]:

- Select members and connections to maximize repetition throughout a structure. This enables the fabricator to develop jigs and other fixtures that will speed up the fabrication process.
- Where possible, select connections whose assembly of fitments on the member can be carried out in one position. This will reduce the amount of handling or rotating operations needed during fabrication.

- Minimize the number of components in the connection.
- Configure connections such that assembly of components occurs on the fewest number of members.
- Where possible, use standardized connections; be aware that common practices may vary among industries.
-
- Before committing a design to the detailing phase, it is advisable for designers to communicate with technical representatives of material suppliers and fabricators, as they can help to identify solutions that may optimize the use of material and labor.
- In general, welding should be done in the workshop and bolting on site. Specifically, on the subject of welding, the following should be considered:

  When welding in the workshop:
  - Simplified and controlled welding processes and access to more certified welders
  - Ability to impose more automated processes
  - Lifting capacities are typically higher than on site
  - Ease of implementing repair processes (e.g., grinding)

  When welding on site:
  - Provide weld access holes, temporary / intermediate fixation points for manipulation, alignment and stabilization; and if necessary temporary support, etc.
  - Lifting capacities are limited Pre-erection possibilities can occur on the ground
  - Staging areas, electric equipment and access to the site may be limited.

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Annexes

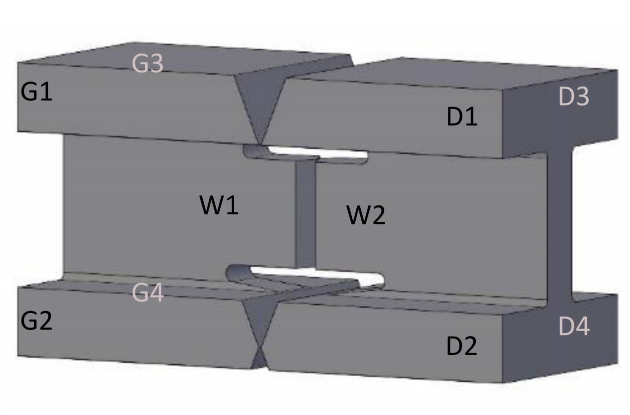
Annex A: Example of butt welding with H5 of HD 400 x 1299 (tf = 140 mm) of HISTAR® 460

1. Welding procedure specification (WPS)

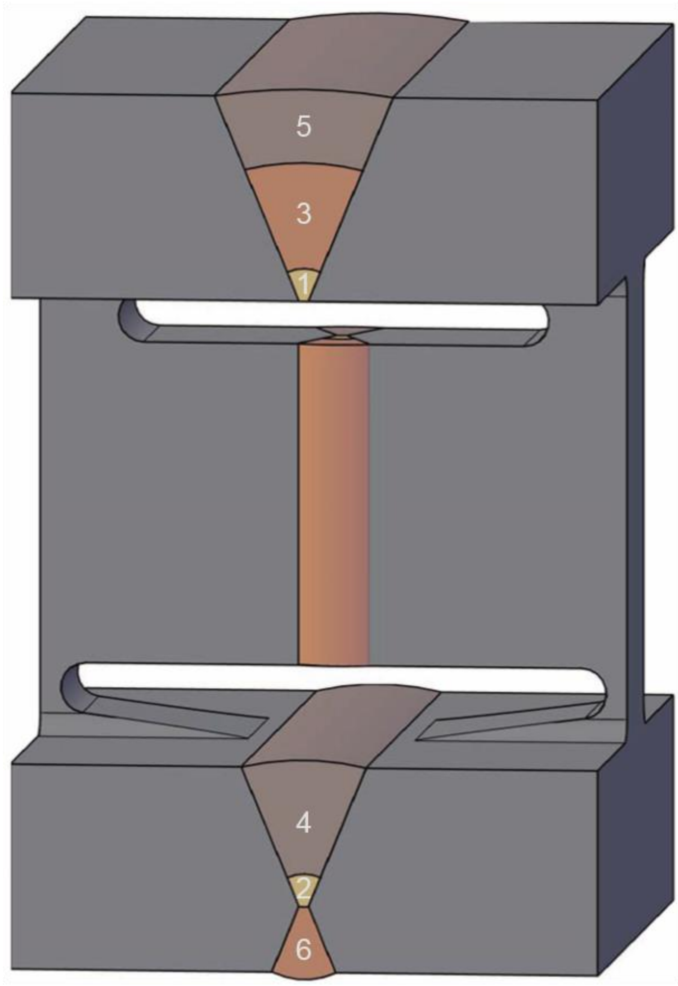
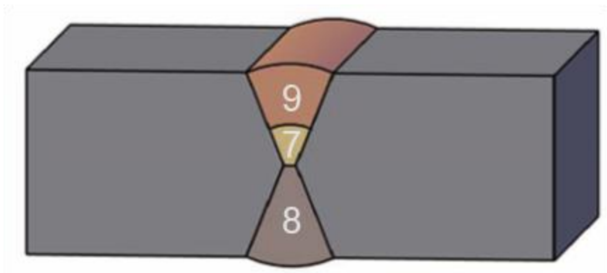
Butt welding assembly performed by Wallerich-Lux Sanem

To characterize properties after welding acc. EN ISO 15614

Welding process ISO 4063	Heat Input* (kJ/cm)	Thermal treatment	Bevel machined	Filler metal		
				Ø Core	Designation ISO 17632-A	H content (ml/100g)
FCAW / 138	10 - 12	No preheating Interpass max T°< 250°C	Upper flange V 45° Lower flange X 45° Web X 45°	1.2 mm	T46 4 M M2 H5	≤ 5



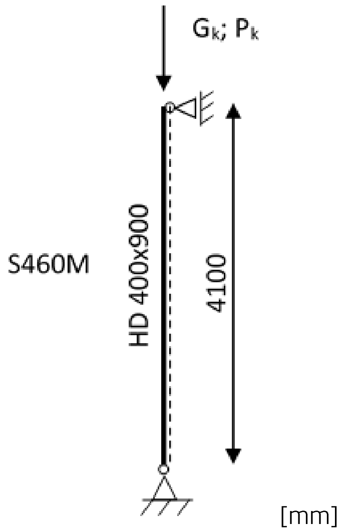
2. Welding procedure sequence

N°	Welding sequences	Position	Joint design
1	Weld the root of the upper flange between 20 to 30 mm	PA / 1G	
2	Weld the root of the lower flange between 20 to 30 mm	PA / 1G	
3	Weld up to the 2/3 upper flange thickness	PA / 1G	
4	Weld externally up to the access hole	PA / 1G	
5	Fill the remaining upper flange	PA / 1G	
6	After cleaning the root, fill the remaining lower flange	PE / 4G	
7	Weld the root of the web between 20 to 30 mm	PA / 1G	
8	After cleaning the root, fill the remaining half web	PA / 1G	
9	Finish the weld, fil the remaining web	PA / 1G	

Annex B: The correct specification of steel – column under axial compression

The column is located in the basement of a 30 storey building and it is subject to axial compression. The static system of the column is indicated in the picture below:

Static system



- Height: 4.1 m
- Distance between columns: 6.0 m
- Slab thickness: 0.20 m
- Floor beams: IPE 500
- Floor beam span: 8.0 m
- Installations: $g_k = 0.1$ kN/m^2
- Partition walls: $q_{k,1} = 0.5$ kN/m^2
- Live loads: $q_{k,2} = 5$ kN/m^2
- Concrete density: 25 kN/m^3
- Steel grade: S460M
- Column: HD 400x900

Loading

- Permanent loads: $G_k = 8616$ kN
- Live loads: $Q_k = 4320$ kN

Verification against brittle fracture

1. The column is subject to quasi-static compression loads. Therefore Table *1-2* in chapter 1 of this document can be applied to check the correct specification of steel to avoid brittle fracture.
2. According to section 3.2.3 in EN 1993-1-1, the tension stress level due to residual stresses is assumed with $\sigma_{Ed} = 0.25 \cdot f_y(t)$.
3. The reference temperature is calculated as follows:
   $T_{Ed} = T_{md} + \Delta T_r + \Delta T_\sigma + \Delta T_R + \Delta T_\varepsilon + \Delta T_{\varepsilon lf}$

With: $T_{md} = 5$ °C (good practice for internal structural elements)
$\Delta T_r = -5$ °C (Background-document EUR 23510 EN – 2008)
$\Delta T_\sigma = 0$ °C (a value $\Delta T_\sigma = 0$ may be assumed when using the tabulated values in table 1.4)
$\Delta T_R = 0$ °C (a value $\Delta T_R = 0$ may be assumed when using the tabulated values in table 1.4)

$\Delta T_{\dot{\varepsilon}} = 0\,°C$ (strain rate $\dot{\varepsilon} < 4 \cdot 10^{-4}$ 1/s, see EN 1993-1-10)

$\Delta T_{\varepsilon lf} = 0\,°C$ (non-cold formed material)

$T_{Ed} = 5\,°C - 5\,°C + 0\,°C + 0\,°C + 0\,°C + 0\,°C = 0\,°C$

4. The maximum allowable thickness can be read out of Table *1-2*:

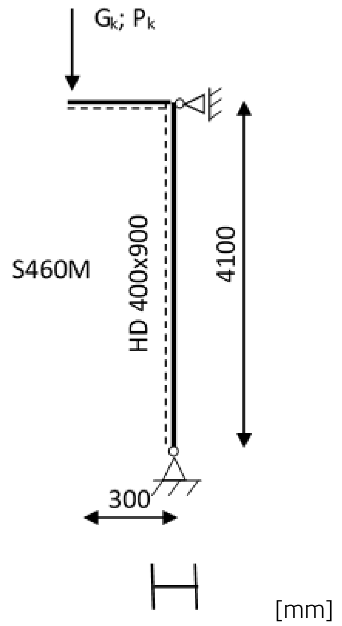
Steel grade		KV		Reference Temperature						
				T_{Ed} [°C]						
Steel grade	Subgrade	T [°C]	J_{min}	10	0	-10	-20	-30	-40	-50
				$\sigma_{Ed} = 0.25 \cdot f_y(t)$						
S355	M	-20	40	150	150	150	150	150	150	150
	ML	-50	27	150	150	150	150	150	150	150
	J2	-20	27	150	150	150	150	150	150	150
S460	M	-20	40	150	150	150	150	150	150	150
	ML	-50	27	150	150	150	150	150	150	150
	J2	-20	27	150	150	150	150	150	150	150

5. The maximum permissible thickness t_{max} = 150 mm > t_f = 106 mm (HD 400x900). As a result, the profile HD 400x900 can be applied in the building without any risk of brittle fracture.

Annex C: The correct specification of steel – column under eccentric compression load

The column is located in the basement of a 30 storey building and it is subject to eccentric compression loads. The eccentricity covers the eccentricities in the columns connections and the misalignment of the column.
The static system of the column is indicated in the picture below:

Static system



- Height: 4.1 m
- Distance between columns: 6.0 m
- Eccentricity: 0.3 m
- Slab thickness: 0.20 m
- Floor beams: IPE 500
- Floor beam span: 8.0 m
- Installations: $g_k = 0.1$ kN/m^2
- Partition walls: $q_{k,1} = 0.5$ kN/m^2
- Live loads: $q_{k,2} = 5$ kN/m^2
- Concrete density: 25 kN/m^3
- Steel grade: S460M
- Column: HD 400x900

Loading

- Permanent loads: $G_k = 8616$ kN
- Live loads: $Q_k = 4320$ kN

The tension part in the section is calculated according to the accidental load case as indicated in EN 1993-1-10.

$$E_d = E\left\{A[T_{Ed}]\ \text{" + "} \sum G_k \text{"+"}\ \psi_1 Q_{k1} \text{" + "} \sum \psi_{2,i} Q_{ki}\right\}$$

With: $\psi_1 = 0.5$

$$\sigma_{Ed} = \frac{N_{Ed}}{A} + \frac{M_{Ed}}{I_y} z = -\frac{(8616 + 0.5 \cdot 4320) \cdot 10^3}{114920} + \frac{(8616 \cdot 0.3 + 0.5 \cdot 4320 \cdot 0.3) \cdot 10^6 \cdot 265.5}{4502000000} = 96.88\ N/mm^2$$

This leads to: $\sigma_{Ed} = 0.22 \cdot f_y(t)$ with $f_y(t) = 450\ N/mm^2$

Verification against brittle fracture

1. The column is subject to quasi-static eccentric compression loads. Parts of the section are subject to tension loads and therefore table 1-3 in chapter 1 of this document shall be applied to check the correct specification of steel against brittle fracture.
2. According to section 2.3.2, Note 1 in EN 1993-1-10, an extrapolation beyond $\sigma_{Ed} = 0.25 \cdot f_y(t)$ is not valid. Therefore, the stress level $\sigma_{Ed} = 0.25 \cdot f_y(t)$ is applied for the verification. The stress level is only slightly higher than $\sigma_{Ed} = 0.22 \cdot f_y(t)$ and leads to safe results.
3. The reference temperature is calculated as follows:
   $T_{Ed} = T_{md} + \Delta T_r + \Delta T_\sigma + \Delta T_R + \Delta T_\varepsilon + \Delta T_{\varepsilon lf}$

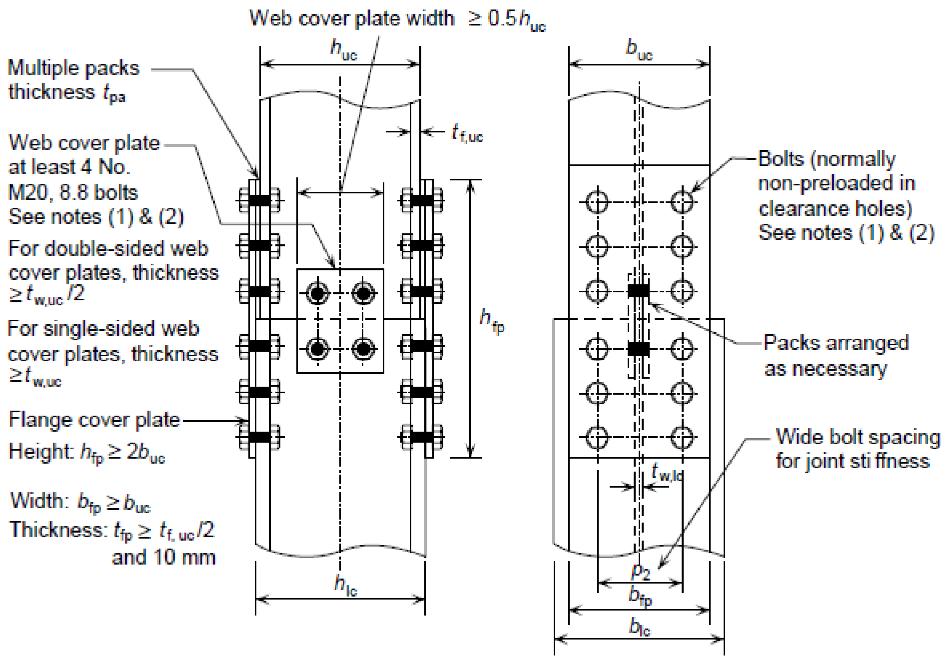
   With: $T_{md} = 5\ °C$ (good practice for internal structural elements)
   $\Delta T_r = -5\,°C$ (Background-document EUR 23510 EN – 2008)
   $\Delta T_\sigma = 0\,°C$ (a value $\Delta T_\sigma = 0$ may be assumed when using the tabulated values in table 1.4)
   $\Delta T_R = 0\,°C$ (a value $\Delta T_R = 0$ may be assumed when using the tabulated values in table 1.4)
   $\Delta T_\varepsilon = 0\,°C$ (strain rate $\varepsilon < 4 \cdot 10^{-4}$ 1/s, see EN 1993-1-10)
   $\Delta T_{\varepsilon lf} = 0\,°C$ (non-cold formed material)

   $T_{Ed} = 5\,°C - 5\ °C + 0\,°C + 0\,°C + 0\,°C + 0\,°C = 0\,°C$
4. The maximum allowable thickness can be read out of table 1-3:

Steel grade	Sub-grade	Charpy energy CVN at T [°C]	J_{min}	Reference temperature T_{Ed} [°C]																				
				10	0	-10	-20	-30	-40	-50	10	0	-10	-20	-30	-40	-50	10	0	-10	-20	-30	-40	-50
				$\sigma_{Ed} = 0{,}75\ f_y(t)$							$\sigma_{Ed} = 0{,}50\ f_y(t)$							$\sigma_{Ed} = 0{,}25\ f_y(t)$						
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	165	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	$\boxed{200}$	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
S690	Q	0	40	40	30	25	20	15	10	10	65	55	45	35	30	20	20	120	100	85	75	60	50	45
	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	-20	40	60	50	40	30	25	20	15	95	80	65	55	45	35	30	165	140	120	100	85	75	60
	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	-40	40	90	75	60	50	40	30	25	135	115	95	80	65	55	45	200	190	165	140	120	100	85
	QL1	-60	30	110	90	75	60	50	40	30	160	135	115	95	80	65	55	200	200	190	165	140	120	100

5. The maximum permissible thickness t_{max} = 200 mm > t_f = 106 mm (HD 400x900). As a result, the profile HD 400x900 can be applied in the building without any risk of brittle fracture.

Annex D Design example – column splice - bearing type (No net tension) [42]



Butting surfaces of sections in direct bearing

Splice details

Flange cover plates: 2/ 865 x 50 x 432
Flange cover plates: 2/ 260 x 30 x 435
Packs
Bolts: M27 (d_o = 30 mm; A_s =53 mm^2) class 8.8: f_{yb} = 640 MPa; f_{ub} = 800 MPa
Fitting materials: S355 steel

HD 400 x 744: h_{uc} = 498 mm; h_{uc} = 432 mm; $t_{w,uc}$ = 55,6 mm; $t_{f,uc}$ = 88,9 mm;
HD 400 x 990: h_{lc} = 550 mm; h_{lc} = 448 mm; $t_{w,lc}$ = 71,9 mm; $t_{f,lc}$ = 115 mm;

Check the column splice for the following design forces:

$\mathrm{N_{Ed,G}} = 2500\mathrm{kN}$ (permanent actions);
$\mathrm{N_{Ed}} = 4500\mathrm{kN}$ (total actions);
$\mathrm{M_{y,Ed}} = 150\mathrm{kNm}$ (about the major axis of the column);
$\mathrm{V_{Ed}} = 80\mathrm{kN}$

The procedure requiring bearing splices needs to be checked initially if the design forces and moment induce net tension in any part of the connection.

EN 1993 – 1 – 8 Clause 6.2.7.[35]: Where members are prepared for full contact in bearing, splice material should be provided to transmit 25% of the maximum compressive force in the column.

There are 2 situations in which the bearing splice can occur:

- Columns with same serial sizes – the transfer of axial forces from upper shaft to lower shaft can be in direct bearing. The flange cover plates can be arranged to connect either to the external faces of the column or alternatively they can be connected to the inner flanges.
- Column with different serial sizes – the transfer of axial forces from upper shaft to lower shaft is made through a horizontal division plate provided between the shafts. The thickness of the division plate is chosen so that the load from the upper section can be transferred to the lower section assuming a spread of load through the division plate at 45°. Flange cover plates then connect to the external faces of the column shafts.

Design procedures are summarized below [42]:

Check	1	Recommended detailing practice	
Check	2	Flange cover plates	– Presence of net tension
Check	3	Flange cover plates	– Plate resistance
Check	4	Flange cover plates	– Bolt group
Check	5	Minimum resistance	– Cover plates and bolt group
Check	6	Tying resistance	– Cover plates and bolt group

Check 1 Recommended detailing practice

Web cover plates

Height ≥ $0,5*h_{uc} = 0,5 * 498$ mm = 249 mm 260 mm

For double side webs cover plate, thickness ≥ $0,5 * t_{w,uc} = 0,5 * 55,6$ mm = 27,8 mm 30 mm

Flange cover plates

Height $h_{fp} \geq 2*b_{uc} = 2 * 432$ mm = 864 mm < 870 mm

Width $b_{fp} \geq b_{uc} = 432$ mm ≥ 432 mm

Thickness $t_{fp} \geq \min(0,5*t_{f,uc}; 10 \text{ mm}; p_1 / 4) = 2 * 432$ mm = 864 mm < 870 mm

For double side webs cover plate, thickness ≥ $0,5 * t_{w,uc} = 0,5 * 55,6$ mm = 27,8 mm 30 mm

Spaces and edges distances – according to EN 1993-1-8 Table 3.3

min $p_1 = 2.2 * d_o = 2.2 * 30$ mm = 66 mm;

max $p_1 = \min(200\text{mm}; 14 * t) = \min(200\text{mm}; 14 * 50 \text{ mm}) = 200$ mm $p_1 = 157$ mm;

min $p_2 = 2.4 * d_o = 2.4 * 30$ mm = 72 mm;

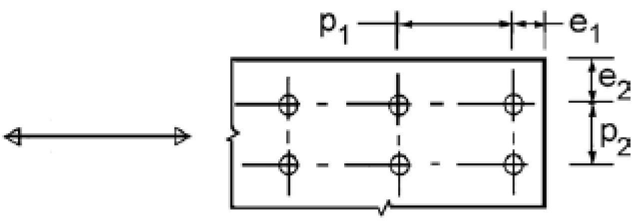
max $p_2 = \min(200\text{mm}; 14 * t) = \min(200\text{mm}; 14 * 50 \text{ mm}) = 200$ mm $p_2 = 200$ mm;

min $e_1 = 1.2 * d_o = 1.2 * 30$ mm = 36 mm;

max $e_1 = 4 * t + 40\text{mm} = 4 * 75.9\text{mm} + 40\text{mm} = 343$ mm $e_1 = 40$ mm;

min $e_2 = 1.2 * d_o = 1.2 * 30$ mm = 36 mm;

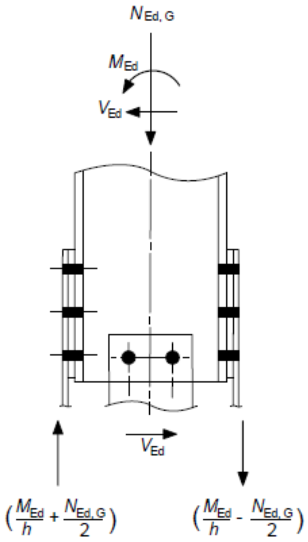
max $e_2 = 4 * t + 40\text{mm} = 4 * 75.9\text{mm} + 40\text{mm} = 343$ mm $e_2 = 116$ mm;



EN 1993-1-8 – Figure 3.1: Symbols for end and edge distances and spacing of fasteners.

Multiple packs thickness: t_{pa} = 25.9 mm

Check 2 Flange cover plates



Basic requirements:

1. If $M_{Ed} \leq \frac{N_{Ed,G}h}{2}$ then net tension does not occur; the splice need only be detailed to transmit axial compression by direct bearing
2. If $M_{Ed} > \frac{N_{Ed,G}h}{2}$ then net tension does occur; Checks 3 and 4 should be used to check the flange cover plates and their fasteners for the tensile force, N_{Ed},

   where: $N_{Ed} = \frac{M_{Ed}}{h} - \frac{N_{Ed,G}}{2}$

   M_{Ed} - is the column design moment (due to factored permanent and variable loads) at the floor level immediately below the splice.

   $N_{Ed,G}$ - is the axial compressive force due to factored permanent load only

   h - is, conservatively, the overall depth of the smaller column (for external flange cover plates) or the centerline to centreline distance between the flange cover plates (for internal flange cover plates)

Preloaded bolts should be used when net tension induces stress in the upper column flange > 10% of the design strength of that column.

Presence of net tension

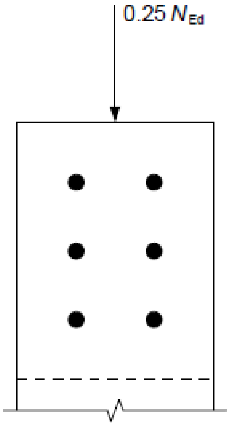
Basic requirements for no net tension: $M_{Ed} \leq \frac{N_{Ed,G}h}{2}$

$h = h_{uc}$ = 498 mm is, conservatively, the overall depth of the smaller column (for external flange cover plates) or the centerline to centreline distance between the flange cover plates (for internal flange cover plates)

$$\frac{N_{Ed,G}h_{uc}}{2} = \frac{2500\text{ kN } 498\text{mm}}{2} = 623\text{kNm}$$

$M_{Ed} = 150\text{kNm} \leq 623\text{kNm}$□ tension does not occur.

Check 5 Minimum resistance – Cover plates and bolt group



Basic requirement: $0.25N_{Ed} \leq N_{Rd}$
Cover plates: $0.25N_{Ed} \leq N_{Rd}$

$$N_{Rd} = \frac{2A_{fp}f_{y,fp}}{\gamma_{M0}}$$

Bolt group: $0.25N_{Ed} \leq 2F_{Rd,fp}$
where:
A_{fp} is the area of the flange cover plate
$F_{Rd,fp}$ is the resistance of the bolt group in one flange as calculated in Check 4

Note: Conservatively, it is assumed that the compression is transferred by the flange cover plates and bolts.

Cover plates

Basic requirement: $0.25N_{Ed} \leq N_{Rd}$

$0.25N_{Ed} = 0.25 \cdot 4500\text{kN} = 1125\text{kN}$

$$N_{Rd} = \frac{2A_{fp}f_{y,fp}}{\gamma_{M0}} = \frac{2\ 50\text{ mm }\ 432\text{MPa } 355\text{ MPa}}{1} = 15336\text{kN}$$

Bolt group: $0.25N_{Ed} \leq 2F_{Rd,fp}$

$F_{v.Rd} = \beta_p \frac{\alpha_v f_{ub} A}{\gamma_{M2}} = 0.547 \frac{0.6 \cdot 800\text{MPa} \cdot 457\text{mm}^2}{1.25} = 96\text{kN}$ is the shear resistance of a single bolt.
$F_{b.Rd} = \frac{k_1 \alpha_b d t_{pa} f_u}{\gamma_{M2}} = \frac{2.5 \cdot 0.44 \cdot 27\text{mm} \cdot 75.9\text{mm} \cdot 490\text{MPa}}{1.25} = 1339\text{kN}$is the bearing resistance of a single bolt.
$F_{Rd.fp} = nF_{b.Rd} = 6 \cdot 96\text{kN} = 1152\text{kN}$ $F_{Rd.fp} = nF_{b.Rd} = 6 \cdot 96\text{kN} = 1152\text{kN}$is the design resistance of the bold group.

where:
$n = 6$ is the number of bolts connecting one flange to the cover plate
$\alpha_v = 0.6$for property class 8.8 bolts

$$\beta_p = \begin{cases} 1 \; if \; t_{pa} < \frac{d}{3} \\ \frac{9d}{8d+3t_{pa}} \; if \; t_{pa} \geq \frac{d}{3} \end{cases}$$

For $t_{pa} \geq \frac{d}{3}$, $\beta_p = \frac{9d}{8d+3t_{pa}} = \frac{9\cdot 27}{8\cdot 27+3\cdot 75.9} = 0.547$

d = 27mm is the diameter of the bolt

$t_{pa} = 50 + 25.9 = 75.9$mm is the total thickness of the packing.

$\alpha_b = min\left(\frac{e_1}{3d_0}; \frac{p_1}{3d_0} - \frac{1}{4}; \frac{f_{ub}}{f_u}; 1.0\right) = 0.44$

$k_1 = min\left(2.8\frac{e_2}{d_0} - 1.7; 1.4\frac{p_2}{d_0} - 1.7; 2.5\right) = 2.5$

$\gamma_{M2} = 1.25$ is the partial factor for the bolt resistance.

If the length of the joint in each column $L_j = (n_1 - 1)p_1 L_j = (n_1 - 1)p_1$ is greater than 15d, the design shear resistance of all the fasteners should be reduced by a factor $\beta_{LF} = 1 - \frac{L_j - 15d}{200d}$.

$L_j = (n_1 - 1)p_1 = 2 \cdot 66\text{mm} = 132\text{mm} \leq 15d = 405\text{mm}$

Acknowledgements

This book was made possible by the constant willingness and support of the Global R&D Esch-sur-Alzette working group: Teodora Bogdan, Mike Tibolt and Liudmila Webber, key internal clients Louis Guy Cajot and Shelley Finnigan.
The peer review given by the by welding design consultant PE, ScD Duane Miller, two worldwide recognized steel fabricators Lejeunesteel (Victor Schneur) and Waltersinc (Yasser Elmarsi) and a design office Vinci Construction Grands Projets (Serge Huret), completed the technical information of this connection guide. We thank them deeply for all their efforts in improving the quality of this document.

Disclaimer

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