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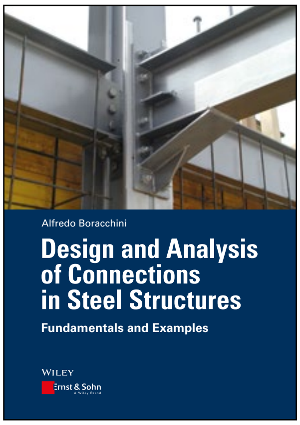

Alfredo Boracchini
Design and Analysis of Connections in Steel Structures
Fundamentals and Examples
August 2018
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The book introduces all aspects needed for connection design and analysis in steel structures. This is not treated according to any specific standard but making comparison among the methodologies and standards, e.g. Eurocode, AISC, DIN, BS. With practice examples and details.

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Preface

Structural Steel Connection Design is an engineering manual directed toward the engineering audience. The first section provides an introduction to key concepts, then progresses to provide a more in-depth description for the design of structural steel connections.

A correct approach to connection design is fundamental in order to have a safe and economically sound building. Therefore, this book will attempt to explain how to set up connections within the main calculation model, choose the types of connections, check them (limit states to be considered), and utilize everything in practice.

The focal point of the book is not to closely follow and explain one specific standard; rather the aim is to treat connections generally speaking and to understand the main concepts and how to apply them. This means that, even though Eurocode (EC) and the American Institute of Steel Construction (AISC) are the most referenced standards, other international norms will be mentioned and discussed. This helps to understand that connection design is not an exact science and that numerous approaches can be viable.

Type by type, connection by connection, detailed examples will be provided to help perform a full analysis for each limit state.

An excellent software tool (SCS – Steel Connection Studio) will be illustrated and used as an aid to assist in the comprehension of connection design. The software can be downloaded for free at www.steelconnectionstudio.com or at www.scs.pe and can be installed as a demo (trial) version (limitations about printing, saving, member sizes, and reporting), see "Software Downloads and its Limitations" (page xxiv). A professional full version can also be purchased online but the demo version is enough to reproduce the examples in the book.

The book will also try to deliver some practical suggestions for the professional engineer: how to talk about bracings to the architect, how to interact with fabricators showing an understanding of erection and fabrication, and much more.

Many countries have a deeper engineering culture about concrete structures than steel structures. This manual therefore aims to illustrate to engineers that do not design steel structures daily, some concepts that will facilitate and make their design of connections for steel structures more efficient. This will be done using a practical, rather than a theoretical, approach.

Design of steel structures can become tricky when it is about stability (buckling) and joints: this second fundamental aspect of steel constructions, which is crucial for economic performance, will be examined in detail.

The text, figures, charts, formulas, and examples have been prepared and reported with maximum care in order to help the engineer better understand and set up his or her own calculations for structural steel connections. However, it is possible that the book contains errors and omissions, and therefore readers are encouraged to have standards at hand as their primary reference. No responsibility is accepted and taken for the application of concepts explained in the manual: the engineer must prepare and perform any analysis and design under his or her complete competence, responsibility, and liability.

For a list of errors and omissions found in the book and their corrections, please check www.steeldesign.info.

Finally, please use www.steeldesign.info to send comments, suggestions, criticisms, and opinions. The author thanks you in advance.

April 2018

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Reggio Emilia

Contents

where L_b is the bolt elongation length, taken as equal to the grip length (total thickness of material and washers), plus half the sum of the depth of the bolt head and the depth of the nut; A_s is the net area of the bolt as seen previously.

3.5 Bolt Failure in Combined Shear and Tension

In the case of combined actions, EC prescribes the formula

$$\frac{F_{v,Ed}}{F_{v,Rd}} + \frac{F_{t,Ed}}{1.4F_{t,Rd}} \leq 1$$

where the subscripts v and t stand for shear and tension, respectively, R for resistance, and E for force (d design).

DIN also requires the equation

$$\left(\frac{N}{N_{Rd}}\right)^2 + \left(\frac{V_a}{V_{a,Rd}}\right)^2 \leq 1$$

with comparable meaning of the symbols (ratio between applied and resistant forces).

For AISC, in combined shear and tension, the equation to apply for the nominal resistance R_n is

$$A_b\left(1.3F_{nt} - \frac{f_v}{\Phi F_{nv}}F_{nt}\right) \leq A_bF_{nt}$$

wherein the only new term is f_v and it represents the shear stress. In other words, the ratio $f_v/\Phi F_{nv}$ is the shear usage ratio that consequently lowers the available tensile resistance. The second term (A_bF_{nt}) shows that the resistance cannot be greater than the case where only tension is included. Summing up the given information, the shear resistance is the first to be verified, and subsequently tension will be checked decreasing the available resistance as indicated in the expression. It is necessary to report that, in order to verify combined tension and shear, Ref. [9] shows an expression comparable to the DIN equation and this method is expressly accepted by [10].

For combined actions in which the shear force is resisted by friction, see the next section.

3.6 Slip-Resistant Bolted Connections

The engineer might decide to design the shear connection as slip resistant, which means the friction between the contact surfaces of the connected elements (proportional to the compressive force that is the preload of the bolt) will resist the design forces, rather than the bolt shank by contact.

The design of a slip-resistant connection can be realized at the serviceability limit state (category B in [1]) or at the ultimate limit state (category C according to EC).

In addition to the bearing resistance (to check for all the categories), category B requires the verification of the slip resistance with serviceability loads in addition to the "classic" shear resistance of the bolt.

The design slip resistance of a preloaded bolt should be taken as per [1] (EC) as

$$F_{s,Rd} = \frac{k_s n \mu}{\gamma_{M3}} F_{p,C}$$

where n represents the number of friction surfaces, μ the slip factor, k_s the coefficient given in Table 3.9, and $F_{p,C}$ the preloading force. According to EC, μ is derivable from specific tests or, in their absence, from Table 3.10; Italian standard for construction (NTC) [11] simply prescribes the value at 0.45 "in case of white metal blasted connections protected until bolt preloading" and at 0.30 for all other cases. For a category B check, $\gamma_{M3,ser}$ is used instead of γ_{M3}. The NTC names both as γ_{M3}, assigning the value 1.25 for the ultimate limit state and the value 1.1 for the serviceability limit state.

In either category B or C, bolts have to belong to classes 8.8 or 10.9 and actually all the components needed for the assembly (bolt, nut, and washer) have to comply with EN 14399 (which is divided in several parts) for a design according to EC. For bolts conforming to these standards, the preloading force used in the previous equation is taken as (all symbols familiar)

$$F_{p,C} = \frac{0.7 f_{ub} A_s}{\gamma_{M7}}$$

Table 3.9 Slip-resistant connections [1], values of k_s.

Case	k_s
Holes with standard nominal clearance	1
Oversize holes	0.85
Short slotted holes with axis perpendicular to the direction of force	0.85
Short slotted holes with axis parallel to the direction of force	0.76
Long slotted holes with axis perpendicular to the direction of force	0.7
Long slotted holes with axis parallel to the direction of force	0.63

Table 3.10 Slip-resistant design from [1, 8], values of μ in the absence of specific tests.

Friction surface class according to [8]	μ
A	0.5
B	0.4
C	0.3
D	0.2

In the case of controlled tightening, γ_{M7} can be taken as 1.

It must be noted that in category C, according to EC, the following must also be checked:

$$\sum F_{v,Ed} \leq N_{net,Rd}$$

where $\sum F_{v,Ed}$ is the design shear force acting upon the bolts (it is the sum of single contributions by each bolt) to check. For the definition of $N_{net,Rd}$ please refer to Section 3.18.2.

As previously discussed, the stiffness coefficient is evaluated as infinite for slip-resistant connections according to EC.

3.6.1 Combined Shear and Tension

If a slip-resistant connection has a tensile force $F_{t,Ed}$ applied as well, the slip resistance per bolt should be taken as

$$F_{s,Rd} = \frac{k_s n \mu}{\gamma_{M3}}(F_{p,C} - 0.8F_{t,Ed})$$

3.7 Bolt Bearing and Bolt Tearing

Bearing of the connected parts is often the reference limit state (i.e. it rules the design) for connections subjected to tension only, for example, braces or truss elements.

Both bolt bearing and bolt tearing depend on the material, bolt diameter, plate thickness, and hole edge distance. The last two variables are certainly the most easily adjustable and in particular the increase in distance between the hole and the edge of the plate is the easiest (and cheapest) to change. The distance between the axis of the hole and the plate edge is usually designed as 1.5 times the diameter of the hole, but in the case of trusses or brace connections, the standard should be increased twice and sometimes further upgraded (up to three times the diameter; greater distances do not help the cause).

If there is more than one bolt in the direction of the load transfer, then it is necessary to evaluate the failure also with regard to the spacing of the bolts, since bearing can occur for inner bolts too and it directly depends on the spacing of the bolts.

Bolt tearing (Figure 3.6) consists in a real shear tear-out, whereas bolt bearing (Figure 3.7) is more a locally visible deformation of the material created by the contact between the bolt and the contour of the hole.

According to DIN, the bearing resistance limit for plate thickness t is obtained using the equation:

$$V_{l,Rd} = \frac{\alpha_1 f_{y,k,pl}}{\gamma_M} dt$$

where t represents the reference thickness, d the bolt diameter, $f_{y,k,pl}$ the yield strength, and α_1 a coefficient depending on the edge distance and the spacing of the bolts.

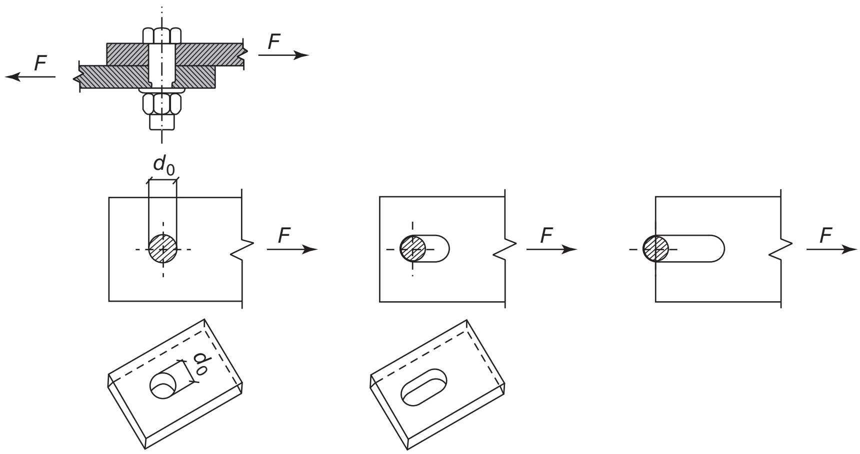
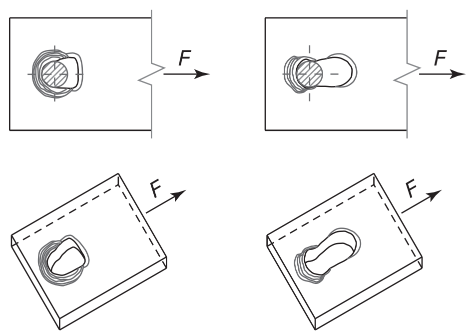


Figure 3.6 Bolt tearing.

Figure 3.7 Bolt bearing.



Considering the symbols in Figure 3.8 (and d_0, the diameter of the hole) and having $e_2 \geq 1.5d_0$ and $e_3 \geq 3.0d_0$, α_1 is derivable as (upper limit)

$$\alpha_1 = \min\left(\frac{1.1e_1}{d_0} - 0.30, \frac{1.08e}{d_0} - 0.77\right)$$

If $e_2 = 1.2d_0$ and $e_3 = 2.4d_0$, α_1 is given as (lower limit)

$$\alpha_1 = \min\left(\frac{0.73e_1}{d_0} - 0.20, \frac{0.72e}{d_0} - 0.51\right)$$

For intermediate values of e_2 and e_3:

$$1.2d_0 < e_2 < 1.5d_0$$

$$2.4d_0 < e_3 < 3.0d_0$$

and the optimal value for α_1 may be obtained by a linear interpolation of the previous cases (the worst between e_2 and e_3 will be taken).

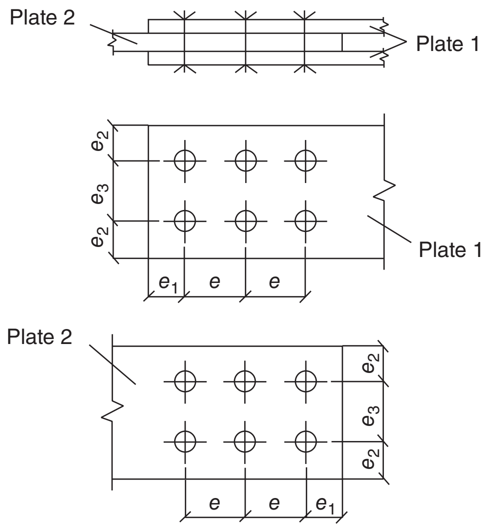


Figure 3.8 DIN symbols.

Please notice that the maximum values that can be used for e and e_1 are

$$e_1 = 3d_0$$
$$e = 3.5d_0$$

This means that greater values of end distance and spacing do not produce any further gain.

The EC method is similar but supplies coefficients for inner and end bolts, for both the direction of the load transfer and the direction perpendicular to the load. The bearing resistance of every single bolt should be compared with the limit value. Alternatively, with an easier and more operational method, it is possible to find the minimum coefficients and prudently adopt them for all the bolts.

The formula to obtain the bearing resistance is

$$F_{\mathrm{b,Rd}} = \frac{k_1 \alpha_{\mathrm{b}} f_{\mathrm{u}} t d}{\gamma_{\mathrm{M2}}}$$

with

$$\alpha_{\mathrm{b}} = \min\left(1, \alpha_{\mathrm{d}}, \frac{f_{\mathrm{ub}}}{f_{\mathrm{u}}}\right)$$

where f_{u} is the ultimate strength of the material, f_{ub} is the ultimate tensile strength of the bolt, and $F_{\mathrm{b,Rd}}$ is the EC symbol corresponding to DIN $V_{\mathrm{l,Rd}}$.

In the direction of load transfer,

$$\alpha_{\mathrm{d}} = \frac{e_1}{3d_0}$$

for end bolts and

$$\alpha_{\mathrm{d}} = \frac{p_1}{3d_0} - \frac{1}{4}$$

for inner bolts. Refer to Figure 3.9 for the meaning of the symbols.

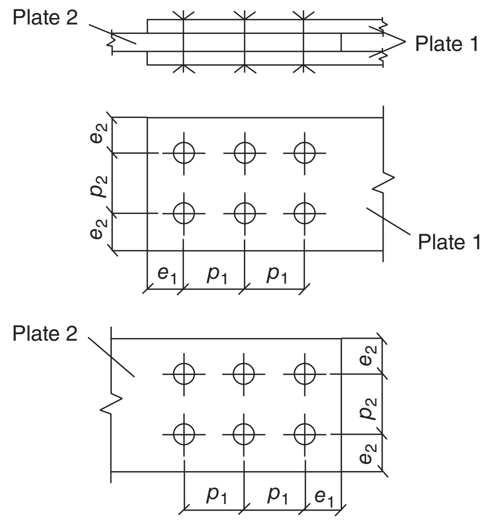


Figure 3.9 Symbols according to EC.

In the direction perpendicular to the load transfer,

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

for edge bolts and

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

for inner bolts. When using the equations above, the EC approach implicitly includes a check of the cross-section and application of these expressions also in compression controls this (tear-out would occur only in tension conditions and bearing too is usually critical in tension because in compression the edge distance is not a factor).

Changing the reference standard, an easy-to-recall formula for tear-out check (in case quick hand calculations are needed) is the AISC formula for R_n (multiply as usual by Φ to obtain the design value):

$$2(0.6F_u)L_c t = 1.2F_u L_c t$$

That is, the formula considers two resisting shear sections as in Figure 3.10 ($0.6F_u$ is exactly the shear limit).

The material collapses according to the angle shown in Figure 3.11, but the "simplified" diagram shown in Figure 3.10 is quite useful when needing to remember the equation (obtained from laboratory tests).

AISC also insists on checking the resistance against the bearing strength, which is equal to

$$2.4F_u dt$$

and the lesser value is the one that will be used as reference.

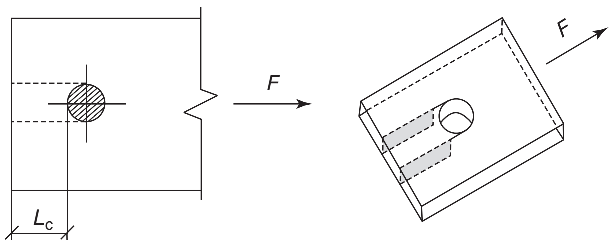


Figure 3.10 AISC representation.

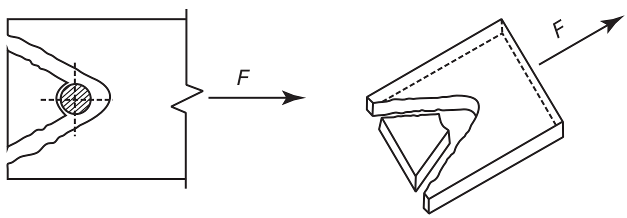


Figure 3.11 Representation of actual collapse.

To be precise, AISC prescribes the previous formulas "when the deformation at the bolt hole at service load is a design consideration" (literally from the specs). If this condition does not apply, it is possible to evaluate the strength R_n as

$$R_n = 1.5F_u L_c t \leq 3F_u\, dt$$

Instead, the Australian standard (see Ref. [6]) prescribes a resisting value equal to the minimum value between

$$a_e f_{up} t_p$$

and

$$3.2 d_f f_{up} t_p$$

where f_{up} is the plate ultimate strength, d_f the bolt diameter, r_p the plate thickness, and a_e the end distance from the hole (in the direction of the load transfer) plus half the diameter of the bolt.

To conclude the overview and certify how the phenomenon has various formulations, consider that, according to the old Italian standard UNI 10011 [12], the limit for the contact pressure value due to bearing (referred to the projected area of the cylindrical surface of the bolt) was

$$\alpha f_d$$

where α (maximum value 2.5) is equal to a/d with a being the end distance from the axis of the hole and d the bolt diameter.

Inserting the value of the pressure area ($t \times \pi d/2$), the result is a bearing strength equal to

$$a\frac{f_y}{\gamma_m} t \frac{\pi}{2} \leq 2.5d\frac{f_y}{\gamma_m} t \frac{\pi}{2}$$

The reference to the yield strength (then compensated by higher coefficients) should be noticed (typically found in older standards) for phenomena in which the ultimate strength should actually be the design reference.

Let us not forget that the verification must be done on both "sides" of the connection, usually a plate and another element (commonly the web) of the connected member. In addition, if there are two resisting sections, the double element will have the total double thickness in calculating the resistance (or half the force if this is checked against only one plate).

For a comprehensive calculation, the check must be executed in both perpendicular directions (i.e. toward both edges of the plate) since (due to eccentricity) the bolts will likely have forces in each direction.

3.7.1 Countersunk Bolts

Countersunk bolts must be checked against bearing using a reduced thickness instead of the nominal value.

Eurocode recommends reducing the reference thickness to a value equal to half the countersink depth.

3.7.2 Stiffness Coefficients

The formulas in [1] are here reported to evaluate the bearing stiffness of the components (unless the joint is designed by friction, where the stiffness is infinite):

$$k_{12}\ (\text{or}\ \ k_{18}) = \frac{24 n_\mathrm{b} k_\mathrm{b} k_\mathrm{t}\ d f_\mathrm{u}}{E}$$

where

$$k_\mathrm{b} = \min\left(1.25,\ \frac{e_\mathrm{b}}{4d} + 0.5, \frac{p_\mathrm{b}}{4d} + 0.375\right)$$

and

$$k_\mathrm{t} = \min\left(\frac{1.5 t_j}{d_{\mathrm{M16}}}, 2.5\right)$$

and n_b represents the number of bolt rows in shear, d_{M16} is the nominal diameter of an M16 bolt, t_j and f_u are the thickness and yield strength of the component, e_b is the distance of the row of bolts from the free edge in the force direction, and p_b is the distance between the bolts in the direction of the force; all other symbols have been previously defined.

3.8 Block Shear (or Block Tearing)

Block shear (or block tearing as it is called in EC) is a phenomenon that occurs due to the combined effects of shear and tension on a plate area in which bolts are present (see Figures 3.12 and 3.13).

The limit state is different from bearing both qualitatively (see Section 3.7) and quantitatively (see the formulas in that section). The block shear is a global type

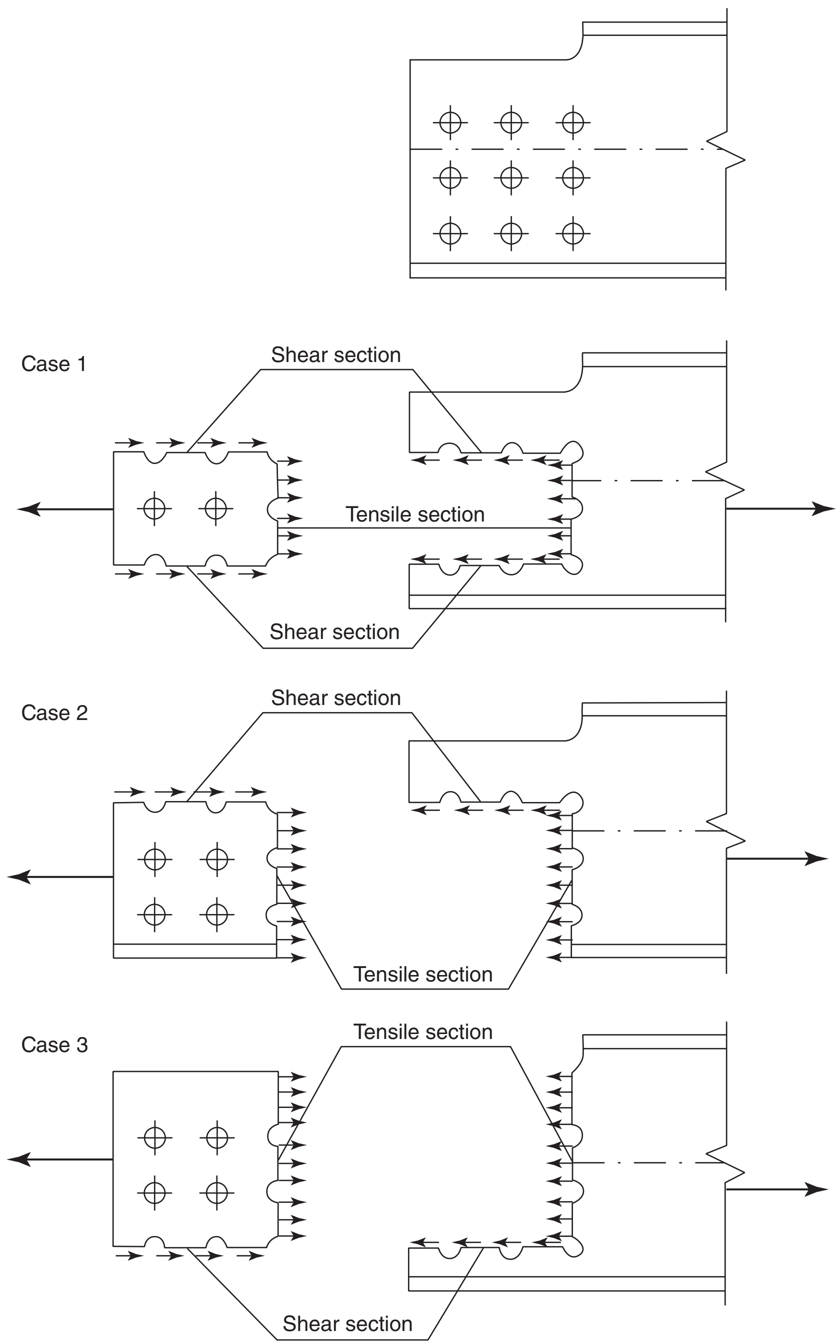


Figure 3.12 Block shear possible modes.

Figure 3.13 Laboratory test dramatically shows the block shear phenomenon. Source: Photo courtesy of J. Swanson and R. Leon, Georgia Tech.

of collapse over a group of bolts: one section is stressed by shear and another is perpendicularly stressed by tension, thus causing a breaking mechanism as in Figure 3.12 (the weakest of the three cases).

Eurocode offers the following formula to estimate the resistance:

$$k\frac{f_u A_{nt}}{\gamma_{M2}} + \frac{\left(f_y/\sqrt{3}\right) A_{nv}}{\gamma_{M0}}$$

where A_{nt} is the net area for tension and A_{nv} the net area for shear. Eurocode also requires halving the first term if there is eccentricity (therefore k is 0.5 if there is eccentricity, otherwise it is 1).

The AISC approach is similar (with the tension part that must be halved for a nonuniform stress) but the second term, in analogy with the plate verification, is taken as the lesser between $0.6F_uA_{nv}$ and $0.6F_yA_{gv}$ with A_{gv} the gross shear area.

Operationally, it would be necessary to check the block shear in both directions unless there is either only a simple axial force or shear with no eccentricity. This is currently poorly addressed in provisions that do not provide any special instructions on how to perform the check in those cases.

The same simplification just viewed that a 0.5 coefficient must be used with stresses that are not uniform (which could be either high or insignificant) and is coarse. It should also to be noted that the formulas for block shear according to the British Standards (BS) are quite different (see Ref. [13] in addition to the official standard [14]). Reference [13] also adds some interesting formulas to check shear and bending interaction of the beam web.

Hopefully, future regulatory updates can give the engineer more precise equations in this field.

It is then important to remember that the block shear collapse is classified as nonductile.

3.9 Failure of Welds

The plates in bolted joints are also largely connected by welds so it is necessary to check their resistance to various actions.

It is not the purpose of this text to delve into the different problems and various methods of analysis and implementation of welds. After an introduction of some important aspects to be kept in mind, only rudiments for basic verifications of fillet welds (which are what is commonly used in connections of standard steel structures because of their low cost and execution simplicity) will be given.

First of all, it must be kept in mind that welds have an extremely limited capacity of deformation, so overcoming the resistance of a weld usually activates a mechanism of brittle type, in particular if the weld is stressed transversely (in spite of the increased load that it can bear).

See Figure 3.14, which shows how the behavior is very fragile for a weld loaded at 90° (angle in relation to the weld longitudinal axis).

It is therefore common practice that, for small-to-medium carpentry jobs, plates are welded to completely restore the full resistance (two fillets with a throat of 5 mm restore a 10-mm plate with good approximation). This kind of design (full strength) also means that the checks are omitted in the calculation reports. For jobs of medium- to large-sized structures and for moment connections, the verification is required for both safety reasons and to avoid unnecessary oversizing. However, it is desirable that the welding failure is not the limit state that governs the design since it does not allow the redistribution of loads. If weld failure governs, the design structure would be considered fragile.

To ensure ductility so that it does not become too expensive, it is enough to size the sum of the throats of the weld equal to the thickness connected, which transmits the force (DIN recommends that each throat of a double fillet be taken as 0.7 times instead of 0.5 when the plate is S355 or equivalent, that is high resistance).

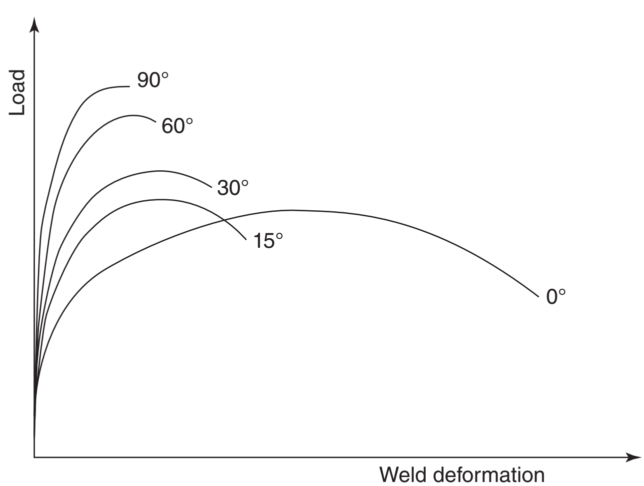


Figure 3.14 Weld deformation depending on the load angle. Source: Taken from Ref. [15].

The exact result that [16] is obtained in calculating the thickness of double fillet welds which guarantees the full strength of connected plates is, depending on the quality of the material, a throat thickness greater than 0.46 times the thickness for S235, 0.48 for S275, and 0.55 for S355. For S420 and S460 materials, each fillet must be greater than values between 0.68 and 0.74 times the thickness.

It is however interesting to note that the latest AISC indications will reduce this request by comparing the yield strength of the plate with the rupture of the weld, whereby the ductility is reached with a leg (not throat!) of $^5/_8$ the thickness for each of the two fillets instead of the $^3/_4$ required by comparing yield with yield. In fact, the AISC standards take as a reference the leg of the fillet, not the throat, and therefore the values seem to be higher (by dividing by $\sqrt{2} \approx 1.4$ the throat equivalent can be calculated). Ultimately, two throat fillets equal to 0.44 times the thickness completely reset the detail strength, ensuring the necessary ductility (which, according to AISC, even with material grade 50 is roughly equivalent to S355).

Another interesting aspect related to what was explained above as well as the economy of the welds is the fact that large welds require multiple runs (also known as "passes"). In fact, up to a throat of about 6 mm ($^1/_4$ in.) a single pass may suffice, but for greater thicknesses it is advisable to have multiple passes to achieve good welding quality. As Figure 3.15 illustrates, many passes are required to reach a slightly higher thickness. For example, a throat thickness of 9 mm requires about three passes, while one of 12 mm requires about five or six runs. This means that, to achieve a resistance equal to about 50% more than a 6 mm fillet, three times more labor is necessary (without considering that it is necessary to "clean" the various welds) and even five or six times the work for double strength. We conclude, then, wherever possible, that it is preferable to "stretch" the welded area with fillets that are not thick, rather than having very thick fillets of limited length.

A much more performing weld is the "full-penetration" weld, which will restore the strength of the connected elements but requires more preparation and control and therefore increasing costs for the fabrication shop (in contrast this solution is inexpensive for the engineer and would avoid any calculation with the simple full-penetration instruction).

It is noteworthy that it is typical to use V- or half-V-shaped (both on only one side) complete penetration welds up to 20-mm- ($^3/_4$-in.) thick plates. Beyond this limit it is convenient to use a double-V or K weld (see Figure 3.19) because, despite requiring a double preparation, it reduces the thickness of welds (consequently also the material and the labor) and guarantees a lower distortion.

A partial-penetration weld (compare Figure 3.17 with Figure 3.16) instead requires careful preparation and a calculation check based on the actual

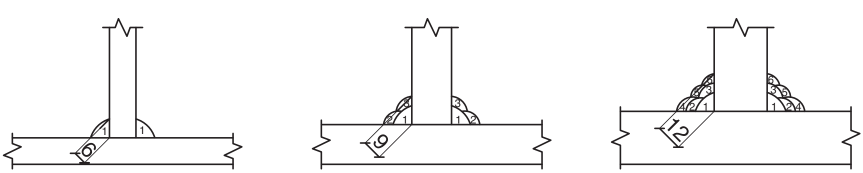


Figure 3.15 Increase of number of weld passes to have larger throats (mm).

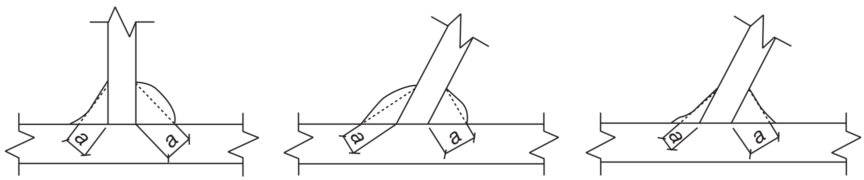


Figure 3.16 Net throat thickness of fillet welds.

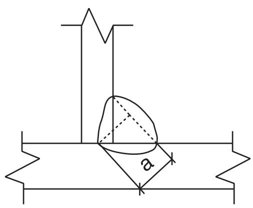


Figure 3.17 Net throat thickness of a partial-penetration weld.

thickness of the throat (similar to those for fillet welds) and it might become necessary to reduce the number of passes or when a "normal" fillet weld adding material to the connected element would interfere with some other element (a partial-penetration weld can exploit some "base material space" as a throat, therefore reducing total thickness).

When feasible, it is preferable to perform a fillet weld on both sides of the piece. A single fillet weld (that has the advantage of avoiding the rotation of the piece in the shop) can be effective for shear but not for tension in a butt joint.

About welding positions (see Figure 3.18), it must be remembered that the flat and horizontal positions are preferred to the vertical and overhead positions (where the gravity makes the operation quite cumbersome).

The welding symbols are many and various, in part depending on different geographical regions, for which the reader is referred to specialized manuals. Some of the most frequently used symbols are given in Figure 3.19. It is also significant to remember that if the indication of the weld is on one side, then the weld is on the near side according to European standards but on the far side by US standards.

As mentioned, the design checks for welds are various and will not be discussed in detail here, but some general guidelines are given below.

3.9.1 Weld Calculation Procedures

Eurocode divides the analysis according to two possible methods of calculation, the directional and simplified methods.

3.9.1.1 Directional Method

According to the directional approach, the design stress must be calculated taking tension and shear separately in both longitudinal and transverse directions, thus obtaining four values (Figure 3.20):

$\sigma_\perp$, normal stress perpendicular to the throat plane
$\sigma_\parallel$, normal stress parallel to the axis of the weld

4.4.10 Example of Base Plate Design According to Eurocode

This example designs, according to the EC (taking $\gamma_{M0} = 1.05$, $\gamma_{M1} = 1.05$, $\gamma_{M2} = 1.25$), the column base plate of an industrial building. The column is an HEA 240 and the governing load cases are the following, which are typical in similar kinds of buildings: The first one (SLU1) occurs when snow and wind (on the column strong axis of the column) act together and SLU2 originates from the wind load in the orthogonal direction that gives maximum uplift when the dead loads are factored as 1 to maximize upward forces. The lateral resisting systems are portals with rigid bases on one side and braces on the weak side (responsible for the remarkable uplift).

We consider S275 as the column material, S235 for the plates, and class 5.6 (yielding at 300 MPa, rupture at 500 MPa) for the anchor bolts.

SLU1:

$$N_{\mathrm{Ed}} = -250\ \mathrm{kN}\ \ (\text{compression})$$
$$V_{\mathrm{major\ Ed}} = 50\ \mathrm{kN}$$
$$V_{\mathrm{minor\ Ed}} = 5\ \mathrm{kN}$$
$$M_{\mathrm{major\ Ed}} = 55\ \mathrm{kN\,m}$$
$$M_{\mathrm{minor\ Ed}} = 0\ \mathrm{kN\,m}$$

SLU2:

$$N_{\mathrm{Ed}} = 110\ \mathrm{kN}\ \ (\text{uplift})$$
$$V_{\mathrm{major\ Ed}} = -5\ \mathrm{kN}$$
$$V_{\mathrm{minor\ Ed}} = 120\ \mathrm{kN}$$
$$M_{\mathrm{major\ Ed}} = -5\ \mathrm{kN\,m}$$
$$M_{\mathrm{minor\ Ed}} = 0\ \mathrm{kN\,m}$$

4.4.10.1 Uplift and Moment

If the concrete has $R_{\mathrm{ck}} = 25\ \mathrm{N\,mm^{-2}}$, that is, $f_{\mathrm{ck}} = 0.83 \times 25 = 20.75$, we get $f_{\mathrm{cd}} = 20.75 \times 0.85/1.5 = 11.8\ \mathrm{N\,mm^{-2}}$. Assuming a ratio of 4 between the area of the plate and the foundation (it will very likely be more), the result is $f_{jd} = 0.67\sqrt{4} \times 11.8 = 15.8\ \mathrm{N\,mm^{-2}}$.

We consider a base plate thickness equal to 20 mm. This means that $c = 20\sqrt{[225/(3 \times 1.05 \times 15.8)]} = 42.5\ \mathrm{mm}$, and hence $l_{\mathrm{eff}} = 240 + 2 \times 42.5 = 325\ \mathrm{mm}$, which is reduced to 300 mm (plate width), and $b_{\mathrm{eff}} = 12 + 2 \times 42.5 = 97\ \mathrm{mm}$, so an effective area that is about $29\,100\ \mathrm{mm^2}$. SCS provides a higher value because it also considers the part below the web in the computation (conservatively neglected here).

The distance of the center of compression from the center of the plate on both sides is equal to the distance between the centerline of the flanges and the axis of the column, namely $z_{\mathrm{C,l}} = z_{\mathrm{C,r}} = 230/2 - 12/2 = 109\ \mathrm{mm}$, smaller in absolute value than the eccentricity, equal to $-55\,000/250 = -220\ \mathrm{mm}$, and thus we will have one side (the left-hand side) in tension and one (the right) in compression. The only effective area in resisting compression in this manual computation, is below the right flange and it bears $15.8 \times 29\,100 = 460$ kN. It must, however, be verified that this number is not higher than the value that the flange and the web of the column can take in compression: $F_{c,fb,Rd} = M_{c,Rd}/(h - t_f)$ and, conservatively assuming a class 3 section so that the elastic resistance is considered, we have $F_{c,fb,Rd} = 675 \times 10^3 \times 275/1.05/(230 - 12) = 811$ kN. SCS correctly takes the plastic resistance and gives us 895 kN.

In the tension zone, on the left, the plate must instead be checked for tension bending (T-stub, Figure 4.32) and the column web in tension near the flange.

Now, assuming the geometry in Figure 4.31, we run some checks noting that, in order to strictly apply the formulas of EC, the anchor bolts should not have a pitch that is greater than the width of the profile.

Alternatively, stiffeners as in Figure 4.33 could be welded to comply with hypotheses for T-stub equations in EC and allow moving the anchors externally. Another alternative (the method used by SCS) would be to make the calculation following the references cited in Section 3.10.7.

Assume a throat weld size of 6 mm for flanges, $m_x = 70 - 0.8 \times 6\sqrt{2} = 63$ mm. Then we have

$$\ell_{eff,cp} = \min(2\pi 63, \pi 63 + 150, \pi 63 + 2 \times 75) = 288 \text{ mm}$$

$$\ell_{eff,nc} = \min(4 \times 63 + 1.25 \times 65, 75 + 2 \times 63 + 0.625 \times 65, 0.5 \times 300,$$
$$0.5 \times 150 + 2 \times 63 + 0.625 \times 65) = 150 \text{ mm}$$

$$\ell_{eff,1} = \ell_{eff,2} \left(= \sum \ell_{eff,1} = \sum \ell_{eff,2}\right) = 150 \text{ mm}$$

Let us check if there is prying action: The elongation length of the anchor is eight times the diameter plus the thickness of the grout and base plate plus the washer and half nut (estimated about 15 mm); hence $8 \times 24 + 20 + 50 + 15 = 277$; this is to be compared with L_b (see Section 3.10.4), which according to EC becomes (A_s of M24 = 353 mm^2) $8.8 \times 63^3 \times 353 \times 2/(300 \times 20^3) = 647$ mm, and

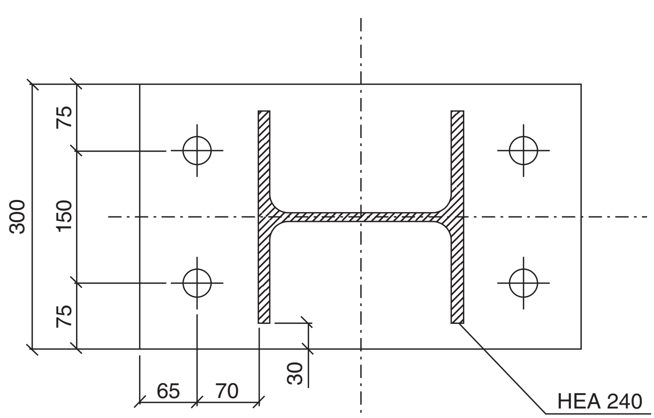


Figure 4.31 Geometry.

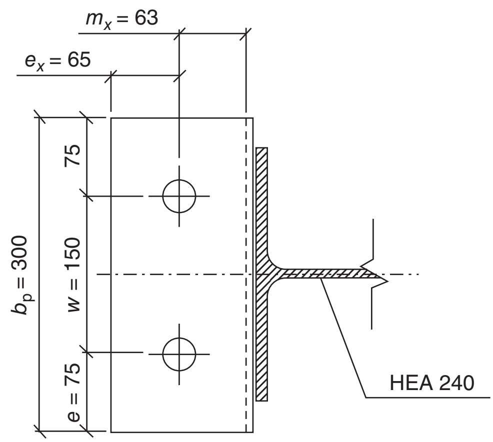


Figure 4.32 T-stub parameters.

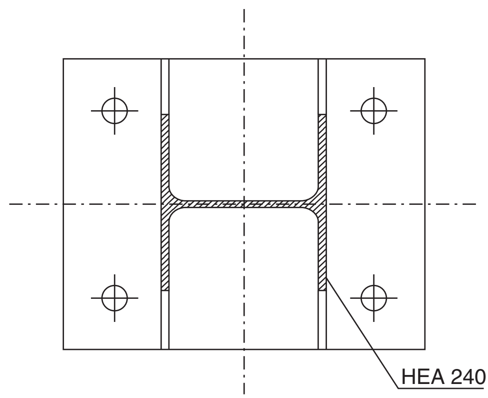


Figure 4.33 Possible solution (not used in the example) to comply with EC assumptions when anchors are outside the column width.

therefore prying action is possible. In fact, some sources (e.g. Refs. [4, 10]) deny the possibility that prying occurs in a base plate, so the engineer could directly apply this hypothesis. However, taking the worst possible situation as per recent instructions (see Section 3.10.4), we have: $F_{T,1-2,Rd} = 2(0.25 \times 150 \times 20^2 \times 225/1.05)/63 = 2(3.21 \times 10^6)/63 = 102$ kN.

We use here 225 MPa instead of 235 MPa because the thickness is >16 mm.

About the resistance of the anchor bolts (M24) on one side, it is $F_{T,3,Rd} = 2(0.9 \times 353 \times 500/1.25) = 2 \times 127.1 = 254$ kN. Then $F_{T,2,Rd} = (2 \times 3.21 \times 10^6 + 65 \times 2 \times 127.1 \times 10^3)/(63 + 65) = 179$ kN, and thus $F_{T,Rd} = \min(102, 179, 254) = 102$ kN. It is underlined that the engineer also has to check (not in the scope of this text) that the foundation and the soil can resist the design loads. The T-stub resistance in tension is therefore 102 kN and this is the actual reference value to take since the web column in tension does not govern the design.

The lever arm z is equal to $z_{C,r} + z_{T,l} = 109 + 185 = 294$, and hence the resisting moment is $M_{j,Rd} = \min(102 \times 294/((109/-220) + 1), -460 \times 294/((185/-220) - 1) = 59.4$ kN m (the tension side governs the design), bigger than the design action (93% design ratio).

Regarding the second combination (SLU2), both sides work in tension (eccentricity is −45 mm) and the resistant force is again 102 kN on each side. For the arm $z = 185 + 185 = 370$ mm, so $M_{j,\mathrm{Rd}} = \max[102 \times 370/(185/-45 + 1), 102 \times 370/(185/-45 - 1)] = -7.4$ kN m, and hence the utilization ratio is 68%.

4.4.10.2 Shear

Since the uplift is remarkable and it will highly stress the anchors, we choose to let a shear key take care of the shear. The first combination has a shear value that is slightly over 20% of the axial force so the friction could absorb it. However, SLU2 cannot rely on friction because of the uplift and, therefore, we weld a profile on the bottom part of the base plate that will likely be placed only in base plates where braces land.

Let us assume a piece of HEA 120 that is 150 mm long. The part inside the grout thickness must not be considered in design so the effective depth to take is $150 - 50 = 100$ mm.

If we design a pit as in Figure 4.34 (that will have grout inside after installing the column) and we consider a ratio between the concrete area (facing the lug in the pit) and the steel area that is about 2.5 in both directions (the depth and the width of HEA 120 are quite similar), the resulting contact pressure is $f_{jd} = 0.67\sqrt{2.5} \times 11.8 = 12.5\ \mathrm{N\,mm^{-2}}$, which means a resisting force of about $12.5 \times 113 \times 100 = 141$ kN in the weak-axis direction and a little more (the flange is 120 mm wide) in the strong axis, both above the design loads (the exact calculation in SCS gives us a maximum design ratio of 83%).

Shear and bending in HEA 120 must also be checked. The design bending moment can be evaluated as (in the SLU2 case, which is the one supposedly governing) $120(50 + 100/2) = 12$ kN m, to be compared with a weak moment resistance of $49\,000 \times 275/1.05 = 12.8$ kN m. The eccentricity (which could be considered in different ways, even zero) has to be set manually in SCS.

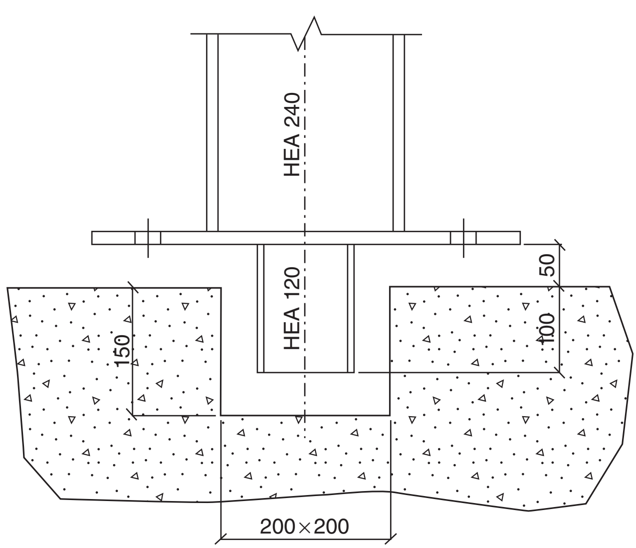


Figure 4.34 Shear lug pit detail.

It is left to the reader to complete the remaining checks.

From SCS calculations we note that the anchors could also work in shear (at 36%) because the interaction with tension is not a problem (61% exploitation, bolt eccentricity set equal to 0).

4.4.10.3 Welding

For the column, the engineer may recommend double fillets with a 6 mm throat for the flanges and 4 mm for the web. This is almost a full-strength weld, with resulting benefit in ductility.

Similarly, in a simplified manner, a 4-mm throat is designed in the weld all around the shear lug (which we just saw working with a good design ratio).

4.4.10.4 Joint Stiffness

The stiffness in the part in compression is $k_{C,r} = k_{13} = 30\,200\sqrt{(300 \times 97)}/(1.275 \times 210\,000) = 19.3$ mm. For the part in tension $k_{T,l} = 1/(1/k_{15} + 1/k_{16}) = 1/(1/(0.425 \times 150 \times 20^3/63^3) + 1/(2 \times 353/277)) = 1/(1/2.0 + 1/2.6) = 1.1$ mm. For SLU1, then, $e_k = (109 \times 19.3 \times 185 \times 1.1)/(1.1 + 19.3) = 93$ mm and $\mu = (1.5 \times 55/59.4)^{2.7} = 2.43$, and hence $S_j = 210\,000 \times 294^2/(2.43\ \ (1/19.3 + 1/1.1)) \times (-220)/(93 - 220) = 1.4 \times 10^4$ kN m.

For SLU2, some coefficients change: $\mu = 1$, eccentricity $e = -45.5$, $z = 370$, and $e_k = (185 \times 1.1 - 185 \times 1.1)/(1.1 + 1.1) = 0$ mm, and thus $S_j = 210\,000 \times 370^2/(1(1/1.1 + 1/1.1)) \times (-45.5)/(0 - 45.5) = 1.6 \times 10^4$ kN m.

4.4.10.5 Comparison with AISC Method for SLU1

Carrying out the design for the same base plate (500×300) according to AISC for the combination SLU1, we get $m = 141$ mm, $n = 54$ mm, $n' = 59$ mm, and hence, with $\lambda = 1$ (conservative), $\lambda n' = 59$ mm. The governing length is therefore 141. Assuming, as we just did, foundations with dimensions at least two times the base plate, $f_{p,max} = 0.6 \times 0.85 \times 0.83 \times 25\sqrt{4} = 21$ N mm^{-2}, and thus the critical eccentricity is $500/2 - 250\,000/(2 \times 21 \times 300) = 230$ mm. Since for SLU1 the eccentricity is $55\,000/250 = 220$ mm, the small eccentricity equations apply, so $h' = 500 - 2 \times 220 = 60$ mm and $f_p = 250\,000/(300 \times 60) = 14$ N mm^{-2}, and therefore the contact pressure is verified with a 66% approximate ratio.

We must now evaluate the bending moment on the plate, loaded by the contact pressure, as $(14 \times 60)(141 - 60/2) = 93\,200$ N mm/mm. When the material is S235, the required thickness becomes, $t = \sqrt{(4 \times 93\,200/(235 \times 0.9))} = 42$ mm, or, considering S275 as the material, 39 mm. As is clear by the numbers, the approach (the AISC method only considers the part in compression for small eccentricities) and the results are quite different. To get a better performance by an AISC-based design, the engineer could shorten the long side (500) of the plate, consequently lowering m and possibly widening the plate to make room for the anchors if geometrically necessary.

4.5 Chemical or Mechanical Anchor Bolts

Mechanical and chemical anchors ("Hilti" probably being the most famous producer) are usually adopted when connecting to a structure that already exists so that precasting bolts is not possible. “Small” parts such as stairs or door frames might also be fixed with this kind of anchor since they do not provide heavy loads. This has the additional advantage of a much wider tolerance of a precast anchor bolt group because the anchors are installed only at the end, with the steel part already in its position.

When connecting to vertical walls, the designer should keep in mind that threaded bars going through the walls can also be utilized.

From a design point of view, in addition to the limit states of the steel parts, that is, the anchors themselves and the plate (refer to the previous section about base plates), all the checks typical of the concrete must be performed since they are usually the ones governing the design.

In most cases it is advisable to follow the charts provided by anchor producers that sometimes even distribute free software to support the design of their products.

It is suggested to carefully evaluate the different types of anchors as anchor bolts with the same diameter and from the same vendor may have different bearing capacities. This clarification of the type, as well as the diameter and the length, must be clearly indicated in the design documents to prevent fabricators from buying just the cheapest available.

4.6 Fin Plate/Shear Tab

The *fin plate* (or *web side plate* or *single shear plate* or *shear tab*) is a connection made with a vertical plate usually welded to the main member (a column or a beam) and bolted to the secondary member (a beam). There are rare cases where the connection is made with two parallel plates with the web of the beam inserted between them (or there are two plates welded to the beam web that are bolted to the fin plate) but the erection issues are apparent though there is a design advantage (bolts work with two shear planes), so this combination is commonly avoided.

The secondary element is a secondary beam if the main member is a primary beam while it is a primary (or secondary) beam if the main member is a column.

The fin plate is recognized as a hinge even if it is able to develop small bending moments. The rotation capacity of the fin plate is not commonly checked (according to “traditional” methods) if the secondary member has deflections in the acceptable range (which is supposed to be the rule). According to the EC, the calculation of the rotation stiffness should, however, be done. Published by the influential and authoritative European Convention for Constructional Steelwork (ECCS) with design examples based on EC, Ref. [13] bypasses the verification ensuring the rotation capabilities as in Section 4.6.3 and coupling it with good ductility (see Section 4.6.4). This approach seems acceptable.

The shear tab in I- or H-shaped profiles (i.e. IPE, HE, W, UB, UC) connects the web of the secondary member and thus is highly effective in conveying shear while it is much less efficient with axial loads since the whole area is not effective (shear lag phenomenon, see Section 3.19.1), as shown in Figure 2.6, having the

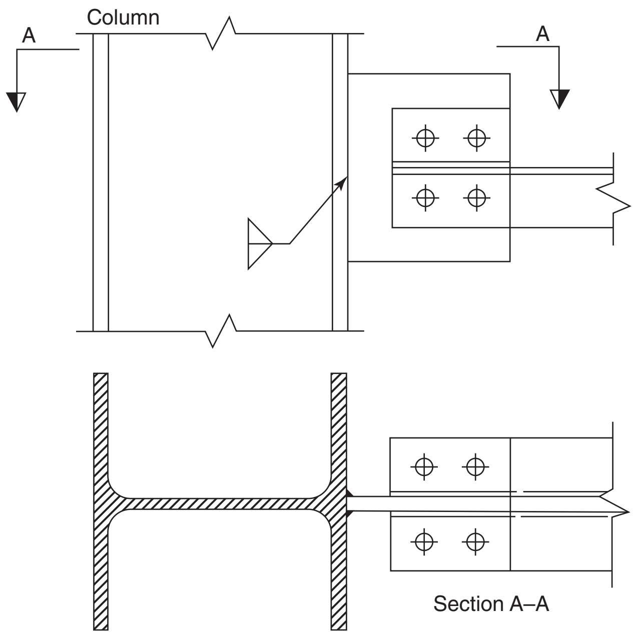


Figure 4.35 Angle brace or strut connected by using an additional angle (called “lug angle” in the EC) in order to transmit higher axial forces.

forces to converge into the web. However, having different shaped profiles (U and L type) as secondary members and possibly by adopting details such as bolting the second leg of the angles or the flanges of the channels, the fin plate becomes highly effective for large tensile and compression loads and it is widely used to connect bracings (Figure 4.35). It is actually effective for braces designed in compression even when only the flange or web is bolted as these braces do not usually transmit big actions, the instability being the element that governs the design of the brace itself.

4.6.1 Choices and Possible Variants

Here we discuss in detail the possible variants for this joint, such as the position of the pin (hinge), the location of the plate, and the notches (copes) in the secondary member. Most of the considerations can be applied later while discussing other connection types without mentioning the options in detail.

4.6.1.1 Pin Position

1. It is possible (actually the most frequently chosen option) to locate the theoretical pin at the axis of the main member. This scheme minimizes the stress in the column (only concentric axial load is transferred) or main beam (no torsion) but it creates a moment in the bolt group due to its eccentricity from the connection axis.
2. It is possible to locate the theoretical pin at the center of gravity of the bolt group. This scheme minimizes the stress in the bolts but worsens the one in

the main element for the occurrence of moments in the latter. If a beam is the primary member, possibly not balanced on the other side by another secondary, the induced torsion is likely a problem, and hence this choice is not recommended in such cases.

3. It is possible to locate the theoretical pin at any position within the range set by the options above. It might, for example, be convenient to take the axis at the contact point between the column flange and the fin plate when the column is connected with the strong axis: The eccentricity in the bolt group is smaller at the cost of a bending moment in the column, which, though, working with its strong axis, can oppose consistent strength (to check with numbers, obviously).

4.6.1.2 Location of Plate Welded to Primary Member

1. It is possible to weld the plate only to the web (if it is a beam or if it is a column oriented according to the weak axis) or to the flange (column strong axis); see Figure 4.36 for examples.

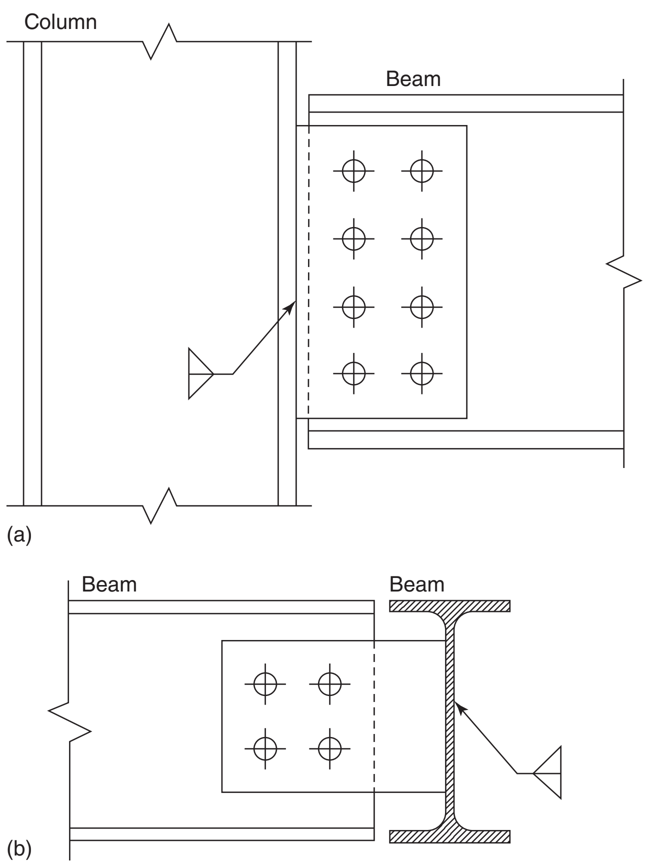


Figure 4.36 Classical solution with the shear tab welded to the primary member flange (a) or web (b).

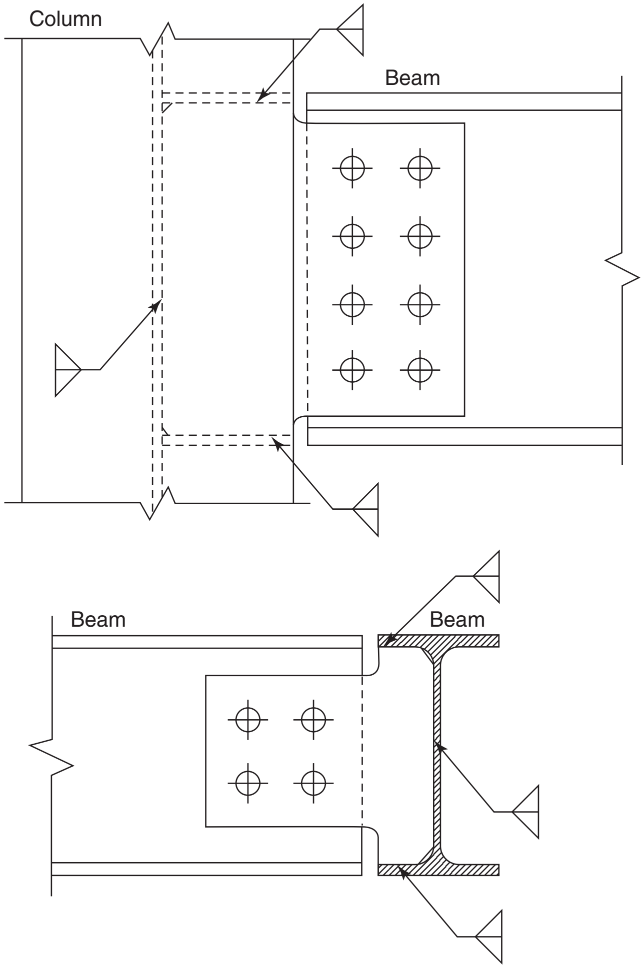


Figure 4.37 Shear tab welded also to the primary member flanges (or stiffeners).

2. It is possible to weld the plate to the web and both flanges (in columns and beams). To do this in columns, some additional stiffeners might be needed as in Figure 4.37.
3. It is possible to weld (Figure 4.38) the plate to the web and one flange (usually the top flange) of the main member (column or beam).

4.6.1.3 Notches (Copes) in Secondary Member

Sometimes, for the onset of problems related to the eccentricity, there is the need to bring the secondary element near the axis of the main one. Depending on the geometry (usually the top of the steel is the same but it is not the rule), it might be necessary to cope/notch the beam:

1. Only the top flange is notched (Figure 4.39, left side).

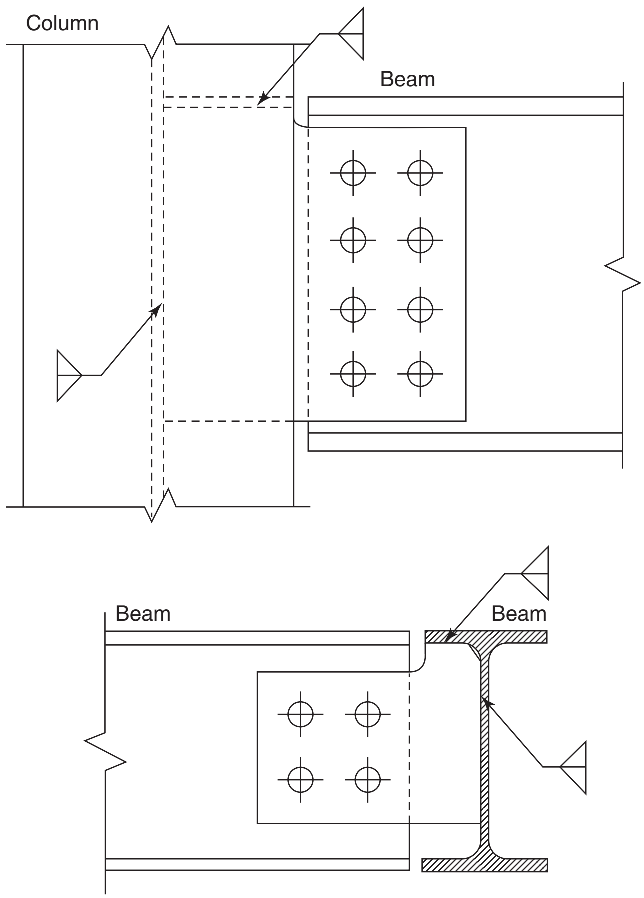


Figure 4.38 Shear tab welded to the primary member top flange (or stiffener).

2. Both the flanges are notched as in the right side of Figure 4.39, which represents an infrequent case of a secondary beam that is deeper than the primary beam, sometimes found when the primary beam is a wide-flange type or the primary is, say, part of a composite construction (so it does not need to be very deep).
3. Only half of the top and bottom flange is notched. This might help in inserting the beam during erection (Figure 4.40).
4. Both flanges have two half notches on each side, as in Figure 4.41. This will allow inserting the beam during erection.

Notching (to allow the secondary beam to be closer to the primary member so that eccentricity is lowered) brings additional costs and so should be avoided if the checks are satisfied without coping the beam.

4.6.1.4 Reinforcing Beam Web

It may happen that the verification of the secondary beam web is not satisfied because of bearing or other limit states, especially in the case of

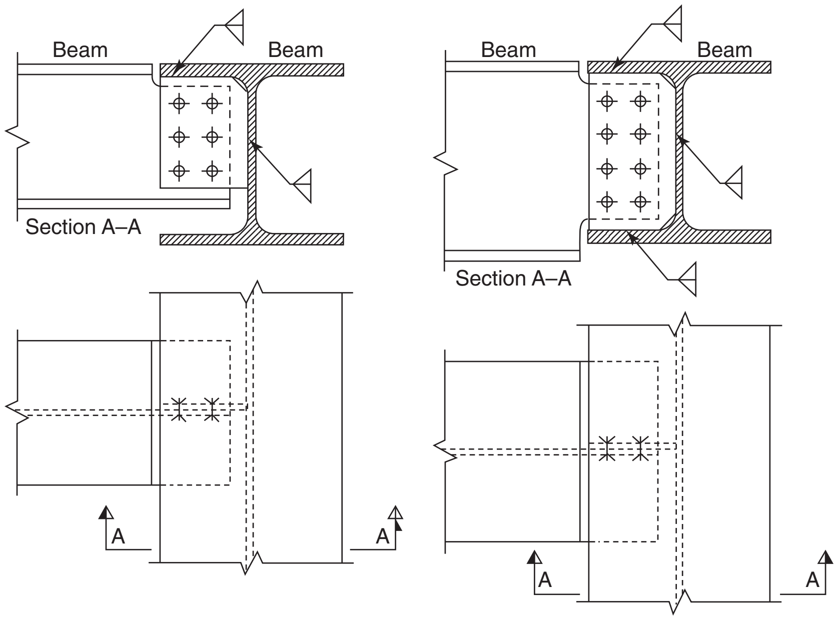


Figure 4.39 Notched configurations.

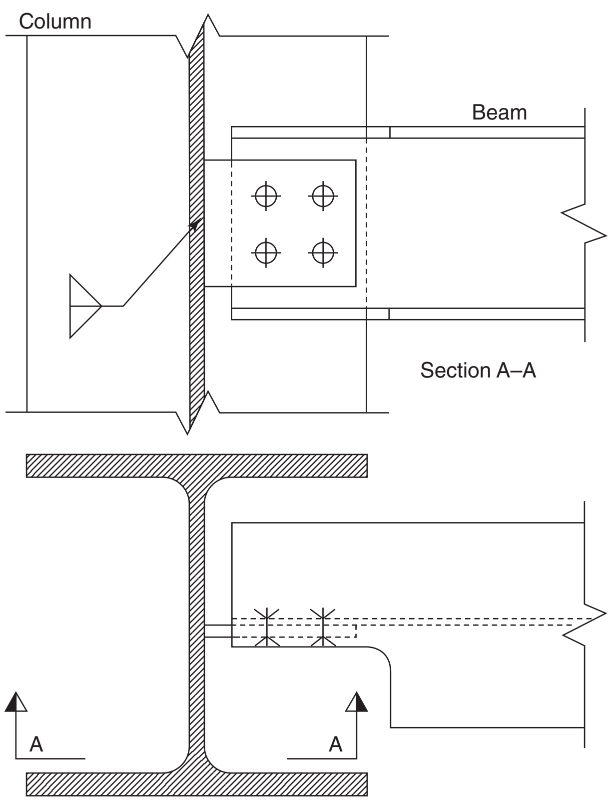


Figure 4.40 Notched flanges can help erection.

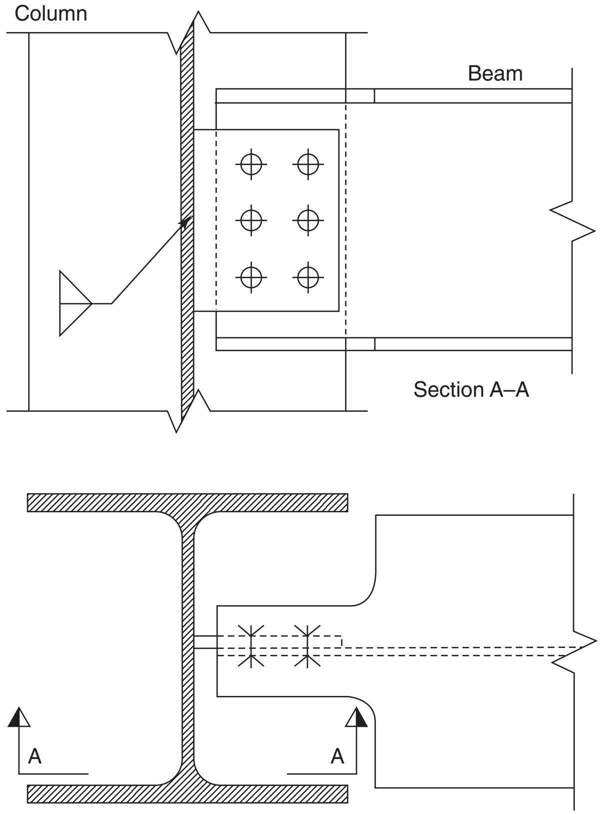


Figure 4.41 Both sides of the flanges are notched to allow positioning during erection.

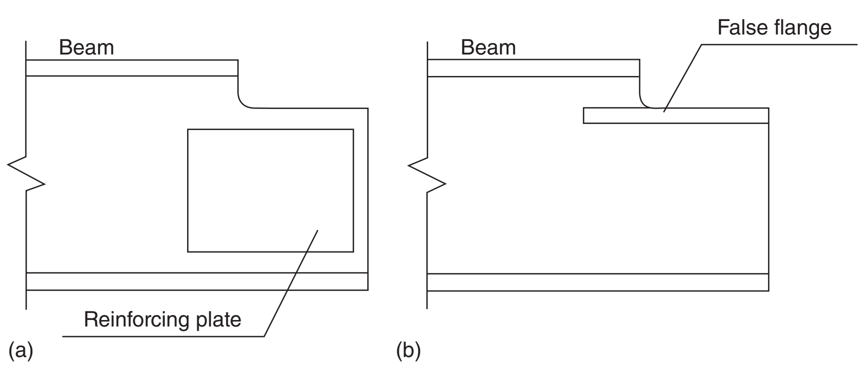


Figure 4.42 (a) Reinforcing plate and (b) false flange.

relevant horizontal forces or eccentricity. In these instances, a reinforcing plate (Figure 4.42a) can be welded to the web, taking into account that if the structures are later galvanized, suitable measures must be provided (Section 6.14).

If the beam is notched, Ref. [1] prescribes to horizontally extend the reinforcing plate beyond the limit of the notch for a length equal to the depth of the notch itself.

Another solution ("false flanges") is to weld plates perpendicular to the web in order to re-create flanges where the beam has been notched (Figure 4.42b).

4.6.2 Limit States to Be Considered

The design must deal with the following (taking into account eccentricities):

- Bolt shear
- Bearing for plate and beam web
- Block shear for plate and beam web
- Plate resistance
- Plate buckling (see Section 3.21, in particular Section 3.21.2)
- Secondary-member resistance taking into account bolt holes and possible notches/copes
- Local resistance of the main member
- Weld resistance.

4.6.3 Rotation Capacity

A method to check the rotation capacity in fin plates can be found in [13, 14]. The geometrical meaning is quite intuitive and consists of avoiding any contact between the parts.

The joint rotation capacity (Figure 4.43) can be considered satisfied if the following is verified:

$$z > \sqrt{(z-g)^2 + (l)^2}$$

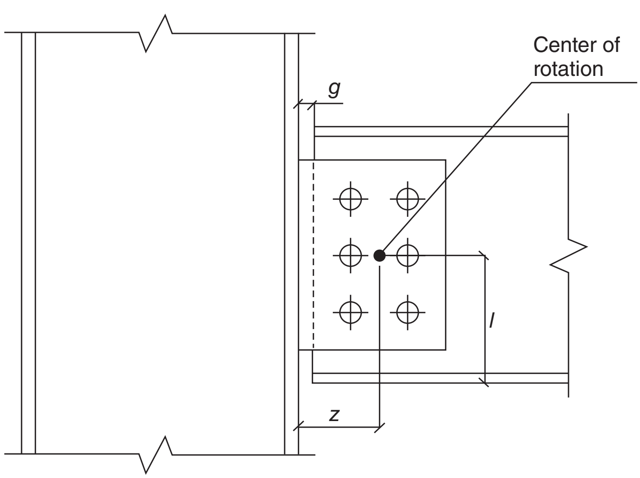


Figure 4.43 Symbols in the formulas to check the rotation capacity.