DESIGN RULES FOR FILLET WELDS IN EUROCODE 3 AND AISC

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INTRODUCTION

The present design rules for fillet welds in Eurocode 3 lead to very thick fillet welds for higher strength steels S420, S460 and S690. Other design rules, e.g. the AISC specification [6] and the Canadian standard CSA-S16-09 [8] give much smaller fillet weld sizes for these steel grades.

Another important difference is that the strength of the filler metal is directly taken into account in the design equation, while in Eurocode 3 Part 1-8 (EN1993-1-8, [3]) the strength of the filler metal is indirectly taken into account. EN1993-1-8 prescribes matching weld metal, while many other codes also allow undermatching weld metal for fillet welds.

Allowing undermatching weld metal for fillet welds enables the designer to choose an optimal combination of parent metal, filler metal and welding parameters to comply with the requirements for ductility and strength.

A summary is given of previous and present design rules and the results of recent research. Proposals for modification of the Eurocode 3 design rules for fillet welded joints are included, focussing on high strength steel.

1   FROM IIW (1974) TO EUROCODE3 DESIGN RULES FOR FILLET WELDS

In 1976, the International Institute of Welding (IIW) published its recommendation "Design rules for arc welded connections in steel submitted to static loads" [1]. The range of application of these IIW design rules is limited to statically loaded welded connections in carbon and low alloy steels having a minimum tensile strength less than 600 N/mm^2, a ratio of yield to ultimate strength $\leq 0,8$ and a minimum elongation $\varepsilon \geq 12\%$. The welds are assumed to be made by arc welding (i.e. covered electrode, gas shielded or submerged arc welding).

During the drafting of Eurocode 3, a re-evaluation was carried out of the test results of the IIW international test series, where more than 700 tests were carried out in 11 countries. Also later tests were considered in the database. To extend the scope of Eurocode 3 to higher strength steel, research was carried out on fillet-welded connections in S460 [5], [13]. Design rules for S420 and S460 were published in Annex D of ENV1993 [3].

In EN1993-1-8 the scope of the main document was extended to cover also steels up to S460 and the rules for welds in S460 were included in the main document. Because of the large scatter of the test results in S460 and the limited number of tests, the resulting design rules give very thick fillet welds [4], [13].

In recent years several research projects have been performed aiming to develop design rules for structures in steel S690. This has resulted in proposals for modification of EN1993-1-8 and EN1993-1-12. The most important modifications are the proposal to modify the resistance part of the design equation by including a direct link to the weld metal strength, and to allow undermatched weld metal.

2   DESIGN OF FILLET WELDS IN EUROCODE 3

Eurocode 3 gives two methods to calculate the design strength of fillet welds: the directional method and the simplified method.

2.1 Simplified method (4.5.3.3 in EN1993-1-8)

According to the simplified method, before called the mean stress method, the design resistance per unit length is:

$$F_{w.Rd} = f_{vw.d} \cdot a \quad \text{with} \quad f_{vw.d} = \frac{f_u / \sqrt{3}}{\beta_w \, \gamma_{M2}} \tag{1}$$

where:

$f_{vw.d}$ = the design shear strength of the fillet weld
f_u = the nominal ultimate tensile strength of the weaker part joined
γ_{M2} = partial safety factor for welded connections ($\gamma_{M2} = 1,25$)
β_w = correlation factor (see *Table 1*)

2.2 Directional method (4.5.3.2 in EN1993-1-8)

In the directional method, before called the stress component method, the forces transmitted by a weld are resolved into normal stresses and shear stresses on the throat section of the weld. The normal stress $\sigma_{//}$ parallel to the weld axis does not need to be considered. The requirement is:

$$\sigma_c = \sqrt{\sigma_\perp^2 + 3\tau_\perp^2 + 3\tau_{//}^2} \le \frac{f_u}{\beta_w \, \gamma_{M2}} \quad \text{and} \quad \sigma_\perp \le 0,9 \frac{f_u}{\gamma_{Mw}} \tag{2}$$

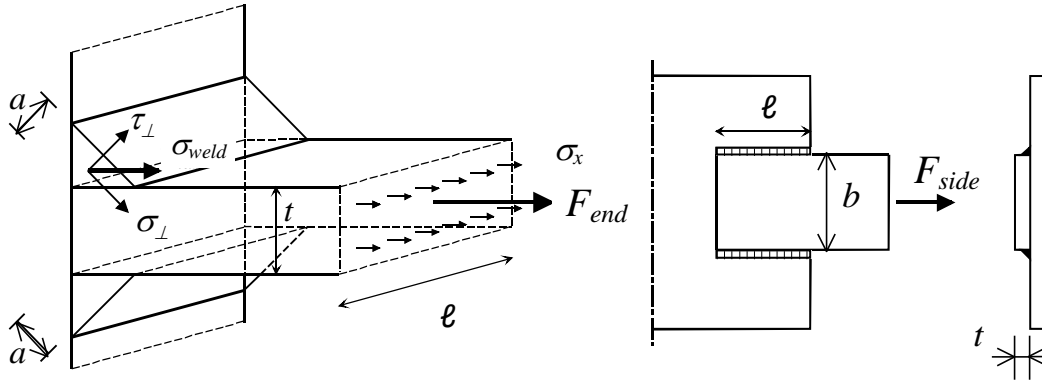*Fig. 1.* End fillet welds and side fillet welds.

For end fillet welds as in *Fig. 1* $\sigma_\perp = \tau_\perp = \dfrac{\sigma_{weld}}{\sqrt{2}}$ and $\tau_{//} = 0$. With equation (2) it follows:

$$\sigma_c = \sqrt{\left(\frac{\sigma_{weld}}{\sqrt{2}}\right)^2 + 3\left(\frac{\sigma_{weld}}{\sqrt{2}}\right)^2} \le \frac{f_u}{\beta_w \, \gamma_{M2}} \tag{3}$$

$$\sigma_{weld} \le \frac{f_u}{\beta_w \, \gamma_{M2} \sqrt{2}} = f_{w.u.end} \tag{4}$$

For double end fillet welds it follows:

$$a \ge \frac{F_{end}}{2\ell \, f_{w.u.end}} = \frac{\sigma_x \cdot t}{2 \, f_{w.u.end}} \tag{5}$$

The greatest weld size is found if in the connected plate $\sigma_x = f_y$. In *Table 1* the required weld sizes are given for this case.

For side fillet welds is $\sigma_\perp = \tau_\perp = 0$ and $\tau_{//} = \tau_{weld}$. With equation (2) it follows:

$$\tau_{weld} \le \frac{f_u}{\beta_w \, \gamma_{M2} \sqrt{3}} = f_{w.u.side} \tag{6}$$

Values for $f_{w.u.side}$ and $f_{w.u.end}$ are given in *Table 1*.

Table 1. Values of β_w, $f_{w.u.end}$, $f_{w.u.side}$ for steels in Table 3.1 of EN1993-1-1, Tables 1 and 3 of EN1993-1-12 and weld thickness for full strength double end fillet welds.

Steel grade	S235 / S235W	S355 / S355W	S355 N/NL	S355 M/ML	S420 N/NL/ M/ML	S460 N/NL/ M/ML	S690 Q/QL/ QL1	S690 Q/QL/ QL1	S690 Q/QL/ QL1
Plate thickness t (mm)	≤ 40	≤ 40	≤ 40	≤ 40	≤ 40	≤ 40	≤ 50	≤ 50	≤ 50
f_y (N/mm^2)	235	355	355	355	420	460	690	690	690
f_u (N/mm^2)	360	510	490	470	520	540	770	770	770
f_{eu} *) (N/mm^2)							770	640	440
β_w	0,80	0,90	0,90	0,90	1,00	1,00	1,00	1,00	1,00
$f_{w.u.end}$ (N/mm^2)	255	321	308	295	294	305	436	362	249
$f_{w.u.side}$ (N/mm^2)	208	262	251	241	240	249	356	296	203
For double end fillet welds with $\sigma_x = f_y$	a ≥ 0,46 t	a ≥ 0,55 t	a ≥ 0,58 t	a ≥ 0,60 t	a ≥ 0,75 t	a ≥ 0,75 t	a ≥ 0,79 t	a ≥ 0,95 t	a ≥ 1,39 t

*) f_{eu} is the nominal weld metal strength.

2.3 Discussion

- In case of side fillet welds, the simplified method and the directional method give the same throat thickness. In case of end fillet welds, there is a difference of a factor $\sqrt{3}/\sqrt{2}=1,22$. This gives for the amount of weld metal to be deposited a factor 1,5.
- From *Table 1* it appears that the design strength $f_{w.u.end}$ and $f_{w.u.side}$ for higher strength steels than S355 is lower than for S355. This is not logical.
- For steel grades up to S460, the strength of the filler metal shall at least be matching the strength of the parent metal. The difference in the strength between the parent metal and the weld metal is implicitly taken into account by the β_w-value. In practice the amount of overmatching depends on the steel grade: in lower strength steel there is usually more overmatching than in higher strength steel.
- For steel grades higher than S460 up till S700, EN1993-1-12 prescribes the use of the tensile strength of the applied filler metal in the design equations. Filler metal strengths grade 35, 42, 55, 62, and 69 with nominal tensile strengths f_{eu} = 440, 500, 640, 700 and 770 N/mm^2 are allowed. For high strength steels it is not easy to have an overmatched weld metal that also fulfils the requirements for economic welding, ductility etc. Undermatched weld metal can then be a good solution, if the lower strength of the weld metal can be compensated by thicker welds.
- Because of the high strain concentrations in fillet welds, see *Fig. 2*, the ductility of the weld metal is very important to allow local yielding and redistribution of stresses. The ductility may have a decisive influence on the strength of the fillet-welded connection [13]. Factors governing the strength and ductility of the deposited weld metal are:
  a. the chemical composition of the filler metal
  b. the chemical composition of the parent metal
  c. the welding process and welding parameters (e.g. the parameters that govern the cooling rate, preheating, the thickness of the electrode, etc.)
  d. the weld thickness
  e. the geometry of the weld (weld discontinuities; stress and strain concentrations).
- Another reason for the low design stresses of fillet welds in S420 and S460 according to the present design rules in Eurocode 3, was the limited number of available test results and the scatter in those results.
- It is noted that for several steel types the design values for the ultimate strength of steel in EN1993-1-1 are lower than in the ENV1993-1-1. This has direct consequences for the design of fillet welds as can be seen in *Table 1*. Whether this is justified is not clear. It was never before.
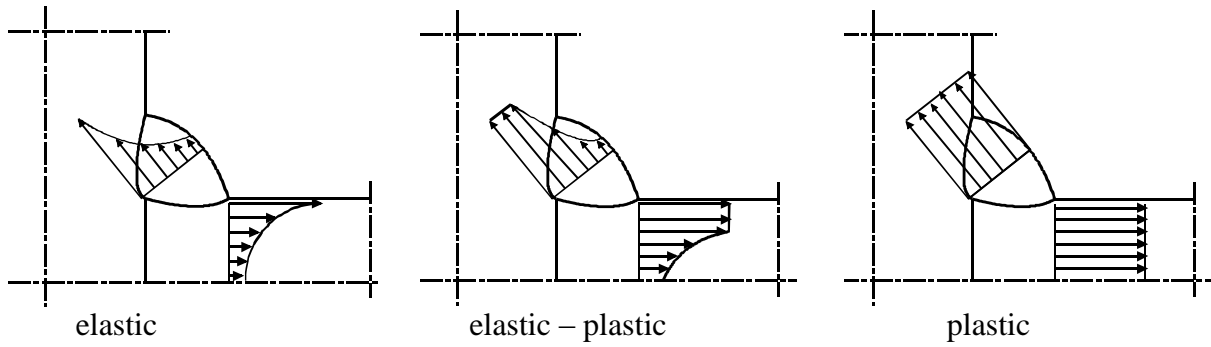
| elastic | elastic – plastic | plastic |

Fig. 2. Stress and strain concentrations, local yielding and redistribution of stresses in fillet welds.

3 DESIGN OF FILLET WELDS ACCORDING TO AISC

In the AISC Specification for Structural Steel Buildings [6] design rules for fillet welds are given in Chapter J, Design of Connections. At several places reference is made to the American Welding Society Codes, e.g. to the AWS Structural Welding Code – Steel [7].

3.1 General provisions

The design strength ϕR_n of welded joints is defined as the lower value of the base material strength $\phi F_{nBM} A_{BM}$ and the weld metal strength $\phi F_{nw} A_{we}$, where:

F_{nBM} = nominal stress of the base metal, ksi (N/mm^2); for tensile force $F_{nBM} = F_u$

F_{nw} = nominal stress of the weld metal, ksi (N/mm^2)

A_{BM} = cross-sectional area of the base metal, in.^{2} (mm^2)

A_{we} = effective cross-sectional area of the weld, in.^{2} (mm^2)

ϕ = resistance factor.

The effective cross-sectional area of the weld is the throat thickness times the effective length of the weld (the same as in Eurocode 3). For the design of fillet welds, filler metal with a strength level equal to or less than matching filler metal is permitted to be used.

In AWS D1.1/D1.1M table 3.1, prequalified base metal – filler metal combinations for matching strength are given. In *Table 2* a summary is given for Shielded Metal Arc Welding (SMAW).

Table 2. Summary of AWS D1.1/D1.1M prequalified base metal – filler metal combinations for matching strength for SMAW.

Group	Minimum yield strength range (N/mm^2)	AWS electrode specification for SMAW	Nominal electrode tensile strength in ksi (N/mm^2)	Nominal electrode yield strength in ksi (N/mm^2)
I	≈240 to ≈315	E60xx - E70xx	60 - 70 (414-483)	48 - 58 (331-400)
II	≈315 to ≈415	E70xx	70 (483)	58 (400)
III	≈415 to ≈450	E80xx	80 (552)	67 (462)
IV	≈485	E90xx	90 (621)	77 (531)

Matching is on the basis of yield strength. In the calculation of the resistance the introduction of overmatching weld metal is not allowed. The sections along the legs need not to be checked since in tests rupture was never observed in this section, provided that the weld metal is not overmatched.

In order to achieve good structural behaviour, the quality of the weld is important. The AWS codes contain many provisions to ensure this. E.g. much emphasis is put on low hydrogen electrodes and prequalified base metal - filler metal combinations. The values of ϕ, F_{nBM} and F_{nw} and limitations thereon are given in Table J2.5 of the AISC specification.

3.2 Design of end fillet welds and side fillet welds in AISC

The design formula for the design strength of fillet welds is:

$$\phi R_n = \phi F_{nw} A_{we} \qquad \text{with} \qquad F_{nw} = 0,60 F_{EXX} (1,0 + 0,50 \sin^{1,5} \theta) \tag{7}$$

where:

ϕ = resistance factor = 0,75

F_{EXX} = electrode classification number, i.e. minimum specified strength, ksi (N/mm^2)

θ = angle of loading measured from the weld longitudinal axis, degrees. For end fillet welds θ is 90 degrees and for side fillet welds θ is 0 degrees

A_{we} = effective area of weld throat, in.^{2} (mm^2)

For end fillet welds and side fillet welds the design strength becomes:

$$\phi R_{n,endfillet} = 0,675 F_{EXX} \, A_{we} \quad \text{and} \quad \phi R_{n,sidefillet} = 0,450 F_{EXX} \, A_{we} \tag{8}$$

For full strength end fillet welds as in *Fig.1* this gives with matching electrodes:

$$\phi R_{n,plate} = \phi_t \, F_y \, A_{plate} = 0,9 \, F_y \, \ell \, t \tag{9}$$

$$\phi R_{n,endfillet} = 0,675 F_{EXX} \, A_{we} = 0,675 F_{EXX} \, \ell \cdot 2a \tag{10}$$

With $\phi R_{n,endfillet} = \phi R_{n,plate}$ it follows:

$$a = \frac{0,9 F_y \, t}{2 \cdot 0,675 F_{EXX}} = 0,667 \frac{F_y}{F_{EXX}} t \tag{11}$$

Table 3. Comparison of weld thickness for end fillet welds according to AISC and Eurocode 3 for matching electrodes.

AISC – SMAW matching			Eurocode 3 (directional method)		
F_y (N/mm^2)	F_{EXX} (ksi – N/mm^2)	a	Steel grade (N/mm^2)	$F_{w.u.end}$ (N/mm^2)	a
235	60 – 414 / 70 – 483	0,38t / 0,33t	S235	255	0,46 t
355	70 – 483	0,49t	S355 / S355 N/NL / S355 M/ML	321 / 308 / 295	0,55t / 0,58t / 0,60t
420	80 – 552	0,51t	S420 N/NL/M/ML	294	0,71t
485	90 – 621	0,52t	S460 N/NL/M/ML	305	0,75t
			S690 f_{eu}=770 / S690 f_{eu}=640 / S690 f_{eu}=440	436 / 362 / 249	0,79t / 0,95t / 1,39t

For side fillet welds the differences between the AISC specification and Eurocode 3 are smaller. The main reason is the strength difference between end fillets and side fillets. For AISC it is a factor 1,5 while in EC3 it is $\sqrt{3}/\sqrt{2} = 1,22$. Nevertheless, for the higher strength steels S420 and S460 the AISC specification still gives considerably smaller sizes (~20%).

Side and end fillet welds have different stiffness and deformation capacity limits, e.g. Butler, Pal and Kulak [12]. This has consequences for the structural behaviour of combinations of welds in different directions, e.g. in lap joints in gusset plates. Reference is made to J2 of [6], section 4(c).

4 THE SWEDISH CODE BSK94 (1997)

In the Swedish code BSK94 [11] the design strength for fillet welds follows from:

$$f_{wd} = \frac{\phi\, f_{wuk}}{1,2\,\gamma_n} \qquad \text{with} \qquad f_{wuk} = \sqrt{f_{ub} \cdot f_{euk}} \qquad (12)$$

where:

f_{ub} = nominal ultimate strength of the base metal
f_{euk} = nominal ultimate strength of the filler metal
ϕ = reduction factor depending on the weld type. For fillet welds $\phi = 0.9$
γ_n = partial coefficient with respect to safety class γ_n varies between 1,0 and 1,2

For side fillet welds the design strength is:

$$F_{R//} = 0,60\, a\, \ell\, f_{wd} \qquad (13)$$

For fillet welds with a different direction the design strength is:

$$F_{R\alpha} = \frac{a\, \ell\, f_{wd}}{\sqrt{2 + \cos 2\alpha}} \qquad (14)$$

For end fillet welds this gives:

$$F_{Re\,nd} = 0,71\, a\, \ell\, f_{wd} \qquad (15)$$

It is noted that in BSK94, the design strength of fillet welds depends on the strength of the base metal and on the strength of the plate metal. In fact, the formulation is between the Eurocode and the AISC specification.

5 PROPOSALS FOR MODIFICATIONS IN EUROCODE 3

5.1 The design resistance of fillet welds

In a previous paper [16] the main directions for new design rules for fillet welds were proposed:

- The resistance function for the fillet weld strength should be depending on the electrode strength, either as it is in AWS D-1.1 and AISC-LRFD or some "mixture", e.g. as in the Swedish code BSK94.
- Undermatching filler weld metals for fillet welds should be allowed. Undermatching filler metals will give thicker welds, but on the other hand, improved weldability and the better ductility will result in better overall structural behaviour.
- The design strength of fillet welds should depend on the weld quality, both in terms of ductility (possibilities for local yielding and redistribution of stresses) and in terms of geometry (shape and absence of weld defects). Whether this dependence can be practically introduced in the design rules needs further consideration.

In the meantime EN1993 Part 1-12 on high strength steels [10] was published which prescribes weld design based on the filler metal strength (filler metal grade).

In 2012 Christina Rasche finalized her PhD on the Load Bearing Capacity of Fillet Welded Connections of High Strength Steels [15]. She performed more than 100 tests on fillet welded connections mainly in S690 with a variation of filler metal strengths. In her analysis she also considered test results from several other sources. Numerical analyses and statistical evaluations according to EN 1990 Annex D were performed.

5.2 Proposal for improvement [15]

A proposal for improvement of the correlation factor β_w according to EN 1993-1-8 and EN 1993-1-12 was developed for high strength steels S460 and S690:

- For S460 N/NL/M/ML: $\beta_w = 0,85$ instead of 1,00 as in the present EN 1993-1-8
- For S690 N/NL/M/ML: $\beta_w = 1,10$ instead of 1,00 as in the present EN 1993-1-12

This gives more logical results for S460, resulting in a smaller difference with the AISC specification. It is noted that the test results in [15] show better ductility than the test results in [5] that were the basis for the present correlation factor β_w for S460. As indicated before: better ductility gives better redistribution of stresses in the throat of the weld and therefore better strength.

5.3 Proposal for modification [15]

The following proposal for improvement (modification) of the present design resistance for fillet welds is presented for the directional method [15]:

$$\sigma_c = \sqrt{\sigma_\perp^2 + 3\tau_\perp^2 + 3\tau_{//}^2} \leq \frac{0,25 f_u + 0,75 f_{eu}}{\beta_w \, \gamma_{M2}} \tag{16}$$

where:

f_u =   the nominal ultimate tensile strength of the weaker part joined
f_{eu} =   the nominal weld metal strength
γ_{M2} =   partial safety factor for welded connections ($\gamma_{M2} = 1,25$)
β_w =   correlation factor depending on the nominal weld metal strength:
   - for weld metal G42/E42 β_w = 0,89   - for weld metal G46/E46/T46 β_w = 0,85
   - for weld metal G69/T69 β_w = 1,09   - for weld metal G89 β_w = 1,19.

With this modified model for the design resistance a safe and more economic design of fillet welded joints particularly of high-strength steels and different filler metals is possible.

5.4 Connections to unstiffened flanges

In 4.10 (3) of EN1993-1-8, it is stated that for an unstiffened flange of and I- or H section the following criterion should be satisfied.

$$b_{eff} \geq (f_{y,p} / f_{u,p}) b_p \tag{17}$$

Otherwise the joint should be stiffened. The background of this requirement is of course the assurance for deformation capacity. At increasing load it guarantees that the connected plate will yield before rupture in the connection. But deformation capacity is not always a requirement. The rule that if (17) is not fulfilled, the connection should be stiffened, is new. It was not in the IIW recommendations of 1974 or any later ECCS recommendation and not in the ENV1993-1-8.

5.5 Minimum weld strength for ductility

In 4.9 of EN1993-1-8, it is stated that in welded joints where deformation capacity is required the welds require sufficient strength not to rupture before general yielding in the adjacent parent material. For unstiffened flanges a rule is given, see previous section. But for e.g. joints like the ones in *Fig. 1*, no rules are given. In the IIW recommendations of 1974 and later ECCS recommendations and national standards the rule was that in such cases the welds should at least be designed for 70% of the full load. In the ENV1993-1-8 this value was raised to 80%. The main reason was that for modern steels and for higher strength steels than S355 the relation f_u / f_y is lower than in the steels covered in previous recommendations and standards (S235 – S355).
A simple and effective solution is similar to the requirement for unstiffened flanges. The welded joint should be designed for at least:

$$\sigma_x = \alpha f_{y,p} / f_{u,p} \tag{18}$$

Where α is a calibration factor, e.g. 1.1. This approach is similar to the one for net section deformation capacity verification of bolted lap joints.

6   CONCLUSIONS

-   Present design rules for fillet welds in Eurocode 3 may give unnecessary thick welds for higher strength steels.

- Design rules where the filler metal is taken directly into account, allow more freedom for the designer to choose a more optimal combination of base metal, filler metal and welding parameters to comply with requirements for ductility and strength.
- Undermatching filler metal may have advantages regarding weldability and better quality of welded joints, possibly leading to better ductility and redistribution of stresses and therefore leading to a higher strength.
- Ductility especially in the root of the weld is essential for redistribution of stresses by local yielding end thereby achieving an optimum strength.
- Proposals for modification of Eurocode 3 design rules have been formulated.

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