Comparison between weld sizing methods included in steel structure standards

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ABSTRACT

The weld sizing methods given in structural steel design standards are based on some simplifications to facilitate the design process, which can lead to an overly conservative design in some instances. This paper examines the weld sizing criteria of structural steel standards, such as EN 1993-1-8 and ANSI/AISC 360, and compares them with the weld design philosophy of NZS 3404, AS4100, and AS/NZS 5100.6. The paper also references the experimental tests performed under HERA's Seismic Research Programme, in cooperation with partner universities of Auckland, Waikato, Michigan, and AUT, to find the appropriate weld sizes in T joints. The results reveal that the current fillet weld sizing criterion included in NZS 3404 is overly conservative. Therefore, it provides the rationale for the introduction of the "equivalent complete penetration butt welds" for T joints to the draft NZS 3404:2024. The equivalent fillet and/or partial penetration compound welds offer the same capacity as complete penetration butt welds, but at significantly lower fabrication costs.

1 INTRODUCTION

Fillet welds are a common type of weld used in structural welded connections, particularly for the lap, T, or cruciform joints (Mellor et al., 1999). The reasons for the popularity of this type of weld rather than complete or incomplete penetration butt welds could be attributed to less fit-up time, no required edge preparation, and the feasibility of welding by electrodes with large diameters (Woerner et al., 2006).

Standards limitations, technical restrictions, and the cost of weld materials can affect the design engineer's decision to choose fillet welds over other types of welds. The size of the fillet welds directly impacts the weld material costs, and it is one of the crucial parameters that affect the final cost of welded steel structures (Jármai and Farkas, 1999). Therefore, the design of fillet welds should satisfy the strength criteria stipulated in design codes while also being cost-effective.

The required fillet weld size designed according to the NZS 3404 standard is usually larger than the one calculated based on other steel design standards such as EN 1993-1-8 (European Standard, 2005) and ANSI/AISC 360 (American Institute of Steel Construction, 2005). This is particularly the case for the welds in the primary load paths for seismic resisting connections in ductile structures, where the overstrength factor is used in the design of the welds in NZS 3404 standard. Hence, many design engineers prefer to use complete penetration butt welds instead of large fillet welds, especially for thick plates. This results in higher fabrication costs for steel structures in New Zealand.

According to Packer et al. (2016), designing fillet welds under general static loads seems quite simple conceptually, however, external forces give rise to complex stress distributions within the weld. Indeed, non-uniform internal stress distributions in welds primarily result from some variables, such as varying weld geometry along the weld length, excessive penetration, or lack of fusion, connection geometry influences, and residual stresses. Accordingly, all available steel design codes consider some simplifications to remove the complexity in the calculation of fillet weld sizes, such as:

- The cross-section of the fillet weld with equal leg sizes is a 45°right triangle, as shown in Figure 1.
- Any excessive penetration or lack of fusion is neglected. In other words, the ideal triangle is assumed along the weld length in the calculations.
- The fillet weld is weakest in shear and is always assumed to fail in this mode.
- The shear failure happens on a plane through the throat of the weld assumed 45° from the weld leg (for welds with equal legs).
- Uniform stress distribution on the throat plane is assumed in the weld sizing criterion.
- The ultimate shear strength of a fillet weld (τ_{uw}) is taken as 0.6 times the tensile strength of the weld metal (f_{uw}) based on the von Mises failure criterion for pure shear ($\tau_{uw} = 0.6f_{uw}$).

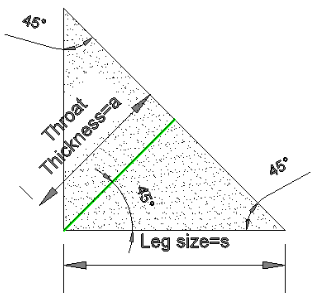


Figure 1: Cross-section of the fillet weld with equal legs.

The simple design philosophy of fillet welds states that the distributed or demand stress on the critical throat plane created due to external forces in fillet welds shall be equal to or smaller than the strength criterion of weld material (Picón and Cañas, 2009). The limit state strength or resistance of the fillet weld should be taken as full yielding of the weld to develop the tensile strength along the failure plane. However, even though many standards employ the classical yield criterion, the uniform stress, in the form of peak load extracted from experimental tests divided by the assumed weakest section area, is used as the strength criterion of the fillet weld in practice. For example, Equation 1 provided by AWS B4.0 (American Welding Society, 2016) standard has been developed based on longitudinal and transverse weld shear strength tests for the calculation of the ultimate shear weld strength.

$$\tau_{uw} = \frac{P_u}{a \times L} \quad (1)$$

where τ_{uw} is the ultimate shear strength of the fillet weld; P_u is peak load prior to failure obtained from the standard shear tests; L is the total length of fillet weld; and a is theoretical throat dimension at 45°.

However, the assumptions in this equation are not supported by experimental testing or numerical modelling. Numerous studies (Lu et al., 2015, Miazga and Kennedy, 1989) reported the actual failure plane angle in fillet welds is about $15°$ to 22.5° instead of 45°. Moreover, there is a considerable discrepancy in the ultimate shear strength of fillet welds between longitudinal and transverse specimens (Lu and Dong, 2020; Lu et al., 2015; Nie and Dong, 2012). The uncertainty for the ultimate weld shear strength in tests would be a reason for using the theoretical value of $0.6f_{uw}$ for shear capacity of fillet welds in many design codes. The theoretical shear strength of the weld is based on the von Mises criterion on the pure shear conditions and yields a conservative value for weld strength.
This study examines the fillet weld design criterion developed based on New Zealand standards to recognise why it gives larger fillet weld sizes than other steel design codes. The research outcome is being used to update the fillet weld sizing criterion used in the standards to give more cost-effective weld sizes while still ensuring reliable performance.

2 FILLET WELD DESIGN CRITERIA BASED ON EN 1993-1-8 STANDARD

There are two methods for designing fillet welds according to EN 1993-1-8 standard (European Standard, 2005). The first method, known as the simplified or mean stress method, operates under the assumption that the average shear stress on the critical failure plan of the weld should be equal to or less than the shear resistance of fillet welds. In other words, the size of the fillet weld is adequate if the resultant of all forces per unit length on the weld is equal to or less than the design resistance of the weld per unit length.

$$F_{w,Ed} \leq F_{w,Rd} \quad (2)$$

where $F_{w,Ed}$ is the design value of the weld force per unit length; and $F_{w,Rd}$ is the design weld resistance per unit length and can be calculated as below:

$$F_{w,Rd} = \frac{0.6 f_u \times a}{\beta_w \gamma_{M2}} \quad (3)$$

where f_u is the nominal ultimate tensile strength of the weaker part joined; a is the theoretical throat dimension at 45°; β_w is the correlation factor; and γ_{M2} is the partial safety factor.

EN 1993-1-8 standard employs the ultimate shear weld resistance ($\tau_{uw} = 0.6 f_{uw}$) in its fillet weld design criterion indirectly. It uses $0.6\ f_u$ which is the shear strength of the weakest part of the joint. Since the base material typically is the weakest part in the joint, f_u usually refers to the ultimate tensile strength of the parent material. However, the correlation factor (β_w) is used for correlating the strength of base metal to matching weld strength. It increases the strength of the base material to be close to that of weld metal (Picón and Cañas, 2009). The correlation factor only depends on steel grade, varying from 0.8 for lower strength to 1.0 for higher strength steel grades (see Table 1). Generally, it is conservative if overmatched electrodes are used, which is a requirement of NZS 3404 for seismically governed connections.
The strength of fillet welds, in this method, is not dependent on the direction of the load. Instead, it considers the pure shear stress on the weld throat plane as the minimum shear capacity of the fillet weld when subjected to a load parallel to the weld axis. Accordingly, the size of welds designed based on the simplified criterion is notably conservative for transverse loading conditions.
The second method of fillet weld sizing in EN 1993-1-8 standard is called the directional method. This method was established based on the von Mises yielding criterion. In the directional method, a uniform distribution of

stresses, including normal and shear stresses, is assumed to act on the weld failure plane. In other words, the strength of fillet welds is calculated under combined stresses induced by external loads in any direction. These stresses are illustrated in Figure 2 and described below:

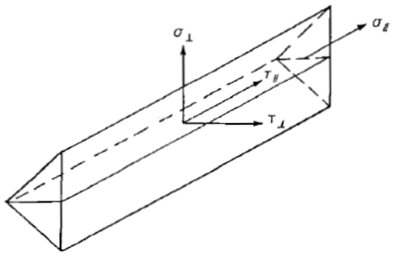


Figure 2: Stresses on the throat section of a fillet weld after EN 1993-1-8 standard.

- $\sigma\perp$ is the normal stress perpendicular to the throat plane,
- $\sigma\|$ is the normal stress parallel to the axis of the weld,
- $\tau\perp$ is the shear stress (in the plane of the throat) perpendicular to the axis of the weld,
- $\tau\|$ is the shear stress (in the plane of the throat) parallel to the axis of the weld.

The 1993-1-8 standard ignores the effects of normal stress parallel to the weld axis ($\sigma_\parallel$) in the fillet weld design for static loading conditions because of low impact on the weld capacity. Therefore, if an infinitesimal cubic volume element on the failure plane of a fillet weld is assumed, the normal stress perpendicular to the throat plane ($\sigma\perp$) and shear stresses perpendicular and parallel to the weld axis are acting on that element.
The von Mises failure criterion states that yielding occurs when the maximum distortion energy is equal to the distortion energy at yielding in a uniaxial tensile test. The final form of von Mises criterion considering principal stresses is simplified as below:

$$\frac{1}{2}[(\sigma_1-\sigma_2)^2+(\sigma_2-\sigma_3)^2+(\sigma_3-\sigma_1)^2]={\sigma_0}^2 \tag{4}$$

where $\sigma_1, \sigma_2, \sigma_3$ are principal stresses; and σ_0 is yielding stress of the material.

The principal stresses for generalised state stress at a point in 3D is calculated from Equation 5:

$$ {\sigma_p}^3-I_1{\sigma_p}^2+I_2\sigma_p-I_3=0 \tag{5}$$

where I_1, I_2, I_3 are the stress invariants.

In the case of considering an element on the weld failure plane, the stress invariants can be obtained as below:

$$I_1=\sigma_\perp, \quad I_2=-({\tau_\perp}^2+{\tau_\parallel}^2)\,, \quad I_3=0$$

By substituting the stress invariant values in Equation 5, then the principal stresses are:

$$\sigma_1=\frac{\sigma_\perp}{2}+\sqrt{\left(\frac{\sigma_\perp}{2}\right)^2+({\tau_\perp}^2+{\tau_\parallel}^2)}, \quad \sigma_2=\frac{\sigma_\perp}{2}-\sqrt{\left(\frac{\sigma_\perp}{2}\right)^2+({\tau_\perp}^2+{\tau_\parallel}^2)}, \quad \sigma_3=0$$

By putting principal stresses in equation 4, the final weld sizing criteria based on von Mises failure criterion is:

$$[{\sigma_\perp}^2+3({\tau_\perp}^2+{\tau_\parallel}^2)]^{0.5}\le\sigma_0 \tag{6}$$

According to Kamtekar (1982), σ_0 is the yield stress of the weld metal in Equation 6 by having elastic-perfectly plastic uniaxial stress-strain curve (see Figure 3(a)). If weld metal has a strain-hardening type stress-strain curve, their behaviour should be taken as a graph with effective stress and strain when material is subjected to the multi-axial stress. Then, σ_0 can be replaced with σ_0' in Equation 6 where σ_0' is effective stress. In uniaxial tension test of weld metal, the obtained curve is similar with Figure 3(b) and the rupture happens in ultimate tensile stress. Therefore, it is assumed that the effective stress is equal to ultimate tensile stress for weld metal, and it is used in Equation 6.

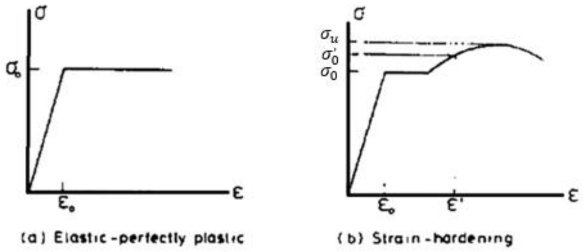


Figure 3: Stress-strain curves from Kamtekar (1982).

Therefore,

$$[\sigma_\perp{}^2 + 3(\tau_\perp{}^2 + \tau_\parallel{}^2)]^{0.5} \leq \sigma_u \quad (7)$$

Accordingly, although the von Mises failure criterion is based on the yield strength of the material, the Eurocode standard considers the limiting combined stress equal to $\frac{f_u}{\beta_w \gamma_{M2}}$. This limiting stress is higher than yielding strength for lower steel grades and, interestingly, is less than yielding strength for higher strength steels, as shown in Table 1 (Henderson, 2016). Finally, the directional method provides the equations below:

$$[\sigma_\perp{}^2 + 3(\tau_\perp{}^2 + \tau_\parallel{}^2)]^{0.5} \leq \frac{f_u}{\beta_w \gamma_{M2}} \quad (8)$$

$$\sigma_\perp \leq \frac{0.9 f_u}{\gamma_{M2}} \quad (9)$$

Equation 9 is for examining the tensile failure of weld or base metal under normal or direct stress. The limiting direct stress values for various steel grades are given in Table 1.

Table 1: limiting stresses in fillet welds based on EN 1993-1-8 standard from Henderson (2016).

Steel grade		S235	S275[1,2]	S355[1]	S420[1]	S460[1]
	β_w	0.8	0.85	0.9	1.0	1.0
Ultimate strength	f_u	360	410	470	520	540
Limiting combined stress	$f_u/(\beta_w\gamma_{M2})$	360	386	418	416	432
Limiting direct stress	$0.9f_u/\gamma_{M2}$	259	295	338	374	389

1 Subgrade M has minimum tensile strengths which vary with thicknesses below 100 mm
2 Subgrades M and N have a minimum tensile strength of 370 MPa

3 FILLET WELD DESIGN BASED ON ANSI/AISC 360 STANDARD

Weld sizing criteria in the American steel design standard is like the simplified method in the Eurocode standard with a modification to include the effects of angle of loading and ductility in the strength of fillet weld. This modification was proposed based on the empirical research carried out by Miazga and Kennedy (1989) and Lesik and Kennedy (1990). It is in the form of a simple loading angle-dependent function as shown below:

$$f(\gamma) = (1.0 + 0.50\, sin^{1.5}\, \gamma) \quad (10)$$

where γ is the angle of loading measured from the longitudinal weld axis, [degree].

This function works as a directional-enhancement factor which should be multiplied to the ultimate shear capacity of fillet weld. Thus, ultimate shear weld resistance is taken as:

$$F_{nw} = 0.60F_{EXX}(1.0 + 0.50\, sin^{1.5}\gamma) \tag{11}$$

where F_{EXX} is electrode classification number that shows minimum specified tensile strength (MPa).

The average shear stress on the assumed critical failure plane is assumed as same as the simplified method in Eurocode. Considering the design strength ($\emptyset$) or allowable strength ($\frac{1}{\Omega}$) safety factors, the final fillet weld design criteria can be written as below:

$$P \leq \left(\emptyset\ or\ \frac{1}{\Omega}\right) * 0.60F_{EXX} * (1.0 + 0.50\, sin^{1.5}\gamma) * a * L \tag{12}$$

where P is the resultant force on the weld; a is the throat size of weld at $45°$; and L is the length of weld.

This standard prohibits using overmatching weld metal (a slight overmatching is permitted) in the welded joints because of increasing the residual stresses during the welding process, which leads to increase cracking tendencies in and around the fillet weld (Miller, 2006). According to Table J2.5 in ANSI/AISC 360 standard (American Institute of Steel Construction, 2005), the filler metal should be selected so that to have a strength level equal to or less than the matching filler metal for the fillet welds.

4 FILLET WELD DESIGN BASED ON NZS 3404 STANDARD

Both steel design standards in New Zealand and Australia suggest an identical fillet weld sizing criterion. According to NZS 3404:part 2 standard, the general failure criterion of fillet welds is as below (see Figure 4):

$$\sqrt{\left[{V_n^*}^2 + K_V\left({V_{Vt}^*}^2 + {V_{Vl}^*}^2\right)\right]} = \emptyset K_W V_W \tag{13}$$

where V_n^* is design force per unit length of weld normal to the plane of the fillet weld throat; V_{Vl}^* is design shear force per unit length of weld longitudinal to the plane of the fillet weld throat; V_{Vt}^* is design shear force per unit length of weld transverse to the plane of the fillet weld throat; V_W is the nominal capacity of a fillet weld per unit length; $\emptyset$ is the strength reduction factor which can be selected from Table 3.3(1) in NZS 3404 standard for SP and GP categories; and K_V & K_W are constant factors.

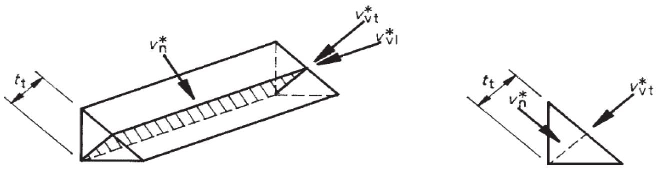


Figure 4: Design actions on a fillet weld after NZS 3404 standard.

Like Eurocode standard, the bending moments at the face of the fillet welds and normal forces applied longitudinally to the weld sections, as shown in Figure 5, are ignored in the failure criterion due to minor effects.

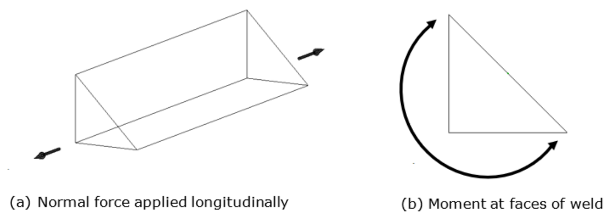


Figure 5: Design actions not considered in assessing the strength of a fillet weld (after NZS 3404: Part 2 standard).

According to NZS 3404 standard, the nominal capacity of the fillet weld per unit length (V_W) is calculated by considering failure shear stress ($0.6F_{uw}$) on the critical throat plane area:

$$V_W = 0.6 f_{uw} t_t k_r \tag{14}$$

where f_{uw} is the nominal tensile strength of weld metal; t_t is design throat thickness on angle $45°$; and k_r is the reduction factor for the length of a welded lap connection (L_w) according to Table 9.7.3.10(2) in NZS 3404 standard.

Indeed, same as the simplified method, the minimum nominal shear capacity of the weld is $0.6f_{uw}$. The k_r factor is used for decreasing the effective length of long welds in the lap joins to account for non-uniformity of stress distribution on the weld due to the existence of stress concentration, shear lag and rotational loads induced by distortion of elements.

There is still some ambiguity in the NZS 3404 standard regarding selecting the correct values for constant factors of K_V and K_w to use in Equation 13, and further explanation is needed. NZS 3404: Part 2 standard adopted values of $K_V = 1.0$ and $K_w = 1.0$ for its fillet weld sizing criterion to facilitate ease of use by designers and minimise design errors. Assuming $K_V = 1.0$ and $K_w = 1.0$, the design force per unit length (V_w^*) is a vectorial sum of the resolved design force components per unit length on the effective throat area of the weld. Thus, the suggested fillet weld design criterion by NZS 3404 standard is:

$$\sqrt{V_n^{*2} + {V_{Vt}^*}^2 + {V_{Vl}^*}^2} \leq \emptyset * (0.6 f_{uw} t_t k_r) \tag{15}$$

Indeed, by adopting $K_V = 1.0$ and $K_w = 1.0$, the general form of the fillet weld sizing criterion in NZS 3404 standard is as same as the simplified method in Eurocode standard (by ignoring the safety factors and substituting weld shear strength instead of base material shear strength and correlation factor). Like the simplified method, the effect of loading angles is disregarded in the fillet weld capacity. Consequently, the weld sizes are conservative, particularly in the case of transverse loading conditions.

NZS 3404: Part 2 standard explains that in AS 1250-1972 standard, which the weld design criterion developed based on the vector addition method, the factor of K_V was considered equal to 1.0. However, AS 1250-1981 and NZS 3404:1989 standards assume $K_V = 3$ and $K_w = 1.0$. In fact, prior to 1992, both New Zealand and Australian standards were intended to change their fillet weld design criterion to that of directional method in the Eurocode standard based on direct application of the von Mises criterion. Because of the ambiguity around these constant factors and difficulty in understanding of weld sizing criterion by steel designers in both countries, which is often incorrectly applied in practice, K_V and K_w were changed to one again in steel design codes after 1992.

NZS 3404 standard permits using double-sided, balanced fillet welds for seismic connections. The sizing procedure for fillet and partial penetration welds includes the overstrength factor ($\emptyset_{oms}$) in designing welds between elements that are in the primary load path of the connection. It leads to larger and more conservative fillet welds to ensure the adequate performance of welds under seismic loads. Table 12.2.8(1) of NZS 3404 standard provides the value of the overstrength factors for beams, braces and columns based on different steel grades and for different member categories. Although there is no limitation to the maximum size of fillet welds in the standard, the larger weld sizes, resulted from applying the overstrength factor, are costly, particularly for thick plates.

Unlike the ANSI/AISC 360 standard, there is no constraint for overmatching weld metal strength in the NZS 3404 standard. However, the matching prequalified welding consumables with steel type have been given in Tables 4.6.1(A) in AS/NZS 1554.1:2014 standard. It is also noteworthy that the tensile strength of weld metal must be equal to or more than the tensile strength of the base metal ($f_{uw} \geq f_u$) for the seismic design of members subject to inelastic actions (category 1, 2 or 3 members).

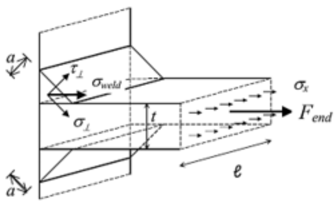
5 EQUIVALENT FULL PENETRATION BUTT WELDS IN T JOINTS

Equivalent full penetration butt welds (EFPBWs) include fillet, partial penetration, or compound welds with equivalent strength as same as full penetration butt welds. Indeed, EFPBWs should have enough strength to allow developing the full capacity of the weakest plate in a joint. As explained in the previous section, NZS 3404 standard yields larger weld sizes rather than other standards, mostly due to ignoring load angling effects and considering overstrength factors in its weld sizing criterion. This is particularly the case, in T joints when the transverse loading on welds is the dominant load combination. It can increase the fabrication costs for seismic connections and environmental damages. Accordingly, it is beneficial to find the optimum size of welds in T joints for avoiding large size of welds. This section intends to summarise the results of some experimental works and research papers to find reliable size of welds which would be used in updating draft of NZS 3404:2024 standard.

Gresnigt (2014) examined the design rules of fillet welds for both Eurocode 3 and AISC standards and specified the differences of weld sizing criteria between these two standards. The publication provides a table (see Table 2) that compares the adequate size of end fillet welds in accordance with AISC and Eurocode standards for various steel grades and matching electrodes. The suitable weld throat dimension can be picked based on the loaded plate thickness (t) for T joints readily by going through this table.

Table 2: Comparison of adequate weld throat sizes based on Eurocode 3 and AISC standards from Gresnigt (2014).

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



The similar calculations for finding the adequate size of throat thickness for end fillet welds in welded moment connections carried out by Taheri (2020) considering different steel grades and loaded plate thicknesses (t) based on NZS 3404 standard. Table 3 gives sufficient throat sizes for electrode strength of 490 MPa and the associated overstrength factor should be included into the formula.

Table 3: Weld throat thickness for the end fillet welds according to NZS 3404 standard for welded moment connections.

Steel grade	**300, 300L0, 300 L15, 300 S0**			**350, 350 L0, 350 S0**		
Thickness (t) [mm]	< 11	≥ 11 To ≤ 17	> 17	< 11	≥ 11 To < 40	≥ 40
f_y [MPa]	320	300	280	360	340	330
f_u [MPa]	440	440	440	480	480	480
Throat thickness for end fillet weld	$t_t \geq 0.68\, \emptyset_{oms} t$	$t_t \geq 0.63\, \emptyset_{oms} t$	$t_t \geq 0.59\, \emptyset_{oms} t$	$t_t \geq 0.76\, \emptyset_{oms} t$	$t_t \geq 0.72\, \emptyset_{oms} t$	$t_t \geq 0.\ 7\emptyset_{oms} t$

In other study, Xing and Dong (2017) collected the failure mode data of a series of cruciform joint samples tested under high-cycle fatigue loading conditions. Eventually, the probabilities of weld toe and root failure versus normalised weld size of s/t were calculated as shown in Figure 6 (where s and t are weld size and loaded plate thickness respectively). According to this graph, if the normalised weld size be approximately 0.93, then the probability of weld toe failure, which is the desirable mode of failure for seismic connections, is 95%.

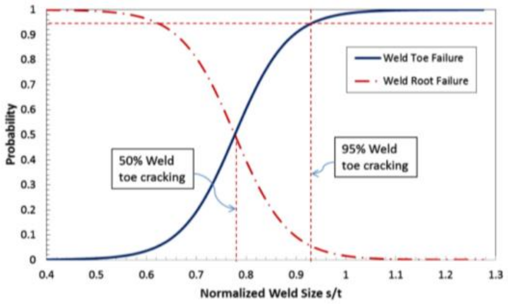


Figure 6: Probabilities of weld root and toe failure versus weld size from Xing and Dong (2017).

Furthermore, a series of experimental tests have been undertaken as a part of HERA Seismic Research Programme to understand the performance of fillet and partial penetration welds in different joints subjected to static and cyclic loading. The new HERA Seismic Programme commenced in 2016 by testing three T-shaped large-scale welded moment-resisting connections at The University of Auckland (see Figure 7). The beam flanges were welded to the column flange by EFPBWs following the requirements of EN 1993-1-8 standard. The varying beam flange thicknesses (11 mm, 20 mm, and 32 mm) across the specimens allowed for a comprehensive analysis of EFPBW performance under seismic load conditions. HERA report R8-43 (HERA, 2021) was developed based on the test results and a table was provided for finding adequate weld sizes based on different beam flange thicknesses. For more information about test results and analysis, refer to this report.

Figure 7: Large-scale tests of welded moment connections made by EFPBWs under seismic loading.

Later, a series of cruciform joints made by optimum size of fillet and partial penetration butt welds were tested under static loading. The designed joint details have been illustrated in Figure 8. Six cruciform joints with fillet weld size of 12 mm together with 6 samples for each type of partial penetration butt weld containing 5 mm and 10 mm gap sizes were fabricated by using two different fabricators. The stem plate thickness was 20 mm with steel grade 300. Based on the test results, the failure mode in all the samples made by partial penetration butt welds was plate failure while 4 out of six samples made by fillet welds failed in the welds. The test results revealed the sufficient size of weld that could shift failure from the weld to the loaded plate. For more information about the tests and result, refer to Forster (2022).

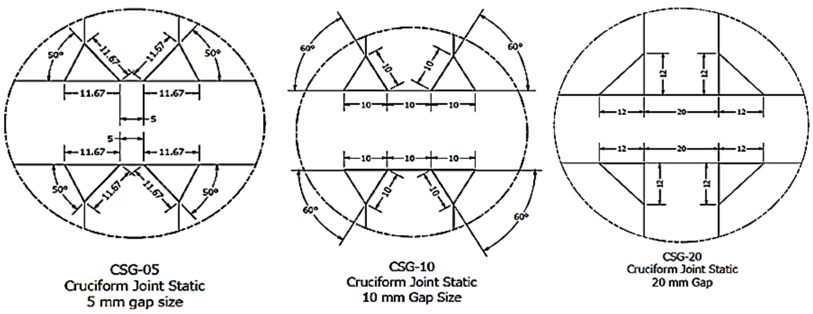


Figure 8: Cruciform joint tests under static loading.

In continuation of HERA Seismic Research Programme, thirty T-shaped samples designed by different fillet weld sizes of 8, 10, 12 and 16 mm were fabricated for testing at Auckland University of Technology (see Figure 9). The loaded plate thickness was 16 mm with grade steel of 300. The aim of the tests was investigating the possibility of revising overstrength and safety factors to reduce the inherent conservatism that has been included in the weld sizing criterion in the NZS 3404 standard. These experiments were undertaken using cyclic loading protocols. According to the results, all the samples made by 12mm fillet welds failed in the plate and the mode of failure in most of the samples with 10 mm was weld fracture. It showed the boundary size for the weld that can supress the weld failure and shift the cracks out of the weld is between 10mm to 12mm. The results of this study will be published by master student, Mark Zhang, soon.

Figure 9: Testing of T-shaped samples under cyclic loading for steel grade 300.

Furthermore, twenty-four T-shaped samples, made by steel grade 350 for stem plate and HSS material for the base plate, were tested at Auckland University of Technology (see figure 10). The typical weld metal with ultimate tensile strength of 490MPa were used for welding the samples. The thickness of stem plate was 16 mm and CPBWs, EFPBWs and 12 mm fillet welds were used in fabrication of T joints. No fracture occurred in the weld during the tests. The PhD student, Kevin Yip, is currently working on this project and the results of the tests will be published in the first half of 2024.

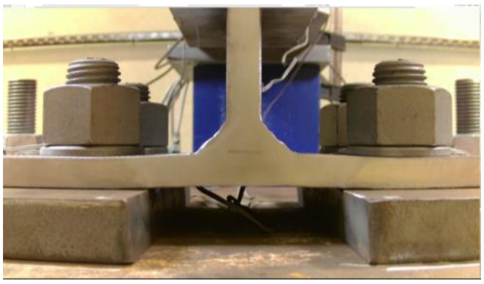


Figure 10: Testing of T-Shaped samples made by HSS base plate and steel grade 350 stem plate under cyclic loading.

From this work, Table 4 summaries the relationship between adequate single weld throat size with the loaded plate thickness in a T joint, based on the findings in different studies. The provided formulas associated with the AISC, Eurocode 3 and NZS 3404 standards have been calculated for steel grade 350 in Table 4. For the NZS 3404 standard, an overstrength factor of 1.25 was assumed, and calculations were performed for plate thicknesses ranging from 11 mm to 40 mm to determine the required weld size. Additionally, the formula associated with HERA report R8-043 in this table was obtained for the plate thickness of 8mm (with grade steel 350) which it gives a conservative value among other thicknesses. It is noteworthy that a series of built-up active links made by EFPBWs and fillet welds between active link flanges and web with endplate and fillet welds between active link web and flanges, with steel grade 300, were tested under seismic loading. Weld failure wasn't observed during the tests. The results demonstrated that smaller throat sizes are sufficient to suppress weld failure, even when there is combined transverse and longitudinal loads on the welds. More information is accessible at Chan (2022).

Table 4: Summary of the results for steel grades 300 and 350.

Source /Reference	**AISC Static loading**	**Eurocode 3 Static loading**	**NZS 3404 Static loading**	**High cycle fatigue loading**	**Low cycle \Seismic loading**	**Static loading**
Gresnigt (2014) Steel grade 350	$t_t \geq 0.49$ t	$t_t \geq 0.58$ t	-	-	-	-
Taheri (2020) Steel grade 350	-	-	$t_t \geq 0.95$ t	-	-	-
Xing and Dong (2017)	-	-	-	$t_t \geq 0.66$ t	-	-
HERA report R8-043	-	-	-	-	$t_t \geq 0.62$ t	
Cruciform joint tests Steel grade 300	-	-	-	-	-	$t_t \geq 0.53\,t$ (W/O safety factor)
T-shaped sample tests Steel grade 300	-	-	-	-	$t_t \geq 0.53\,t$ (W/O safety factor)	-
T-shaped sample tests Steel grade 350	-	-	-	-	$t_t \geq 0.59\,t$ (W/O safety factor)	-

6 CONCLUSIONS

The current weld sizing criterion included in the NZS 3404, AS 4100 and AS/NZS 5100.6 ignores the effects of direction of loading on the weld shear capacity by assuming the constant factors as $K_V = 1.0$ and $K_w = 1.0$. Thus, using current weld sizing criterion results in larger fillet weld sizes in case of applying loads in any direction except parallel to the weld axis. Particularly, in transverse loading conditions, the size of fillet welds is overly conservative.

There is a significant difference in the calculated throat size of welds (required to avoid weld failure) between experimental tests, the AISC and Eurocode 3 standards, and the NZS 3404 standard, particularly for seismic welds. The NZS 3404 standard yields the most conservative throat sizes because it uses the overstrength factor and ignores load angling effects on the capacity of welds. The following multipliers are recommended for finding the total required throat size in T joints:

- For welds connecting members forming a yielding region, the multiplier is 1.2 for grade 300 steel and class 490 MPa weld metal and 1.35 for grade 350 steel and class 490 MPa weld metal,
- For welds connecting members not forming a yielding region, the multiplier is 1.0 for grade 300 steel and class 490 MPa weld metal and 1.2 for grade 350 steel and class 490 MPa weld metal,
- NOTE: A multiplier of 1.35 can be used to cover all the above cases.

Therefore, the adequate total throat dimension for fillet welds on both sides of the stem plate can be calculated without much effort by only multiplying the loaded plate thickness to the proposed multipliers.

The above weld sizing criteria is applicable to fillet, partial penetration, and compound welds with an equal throat thickness used in semi-static and seismic applications to the NZS 3404 standard. It leads to a more efficient and economical weld design. These welds also have performance equivalent to the butt welds on the same joints and can be used to replace butt welds. They can also be considered for the high-cycle fatigue applications, but further application-related assessment will be required. Using multipliers for weld design should be limited to the typical form of multi- and single-storey steel structures in accordance with current New Zealand practices.

7 ACKNOWLEDGEMENT

The authors would like to acknowledge HERA Foundation for providing scholarships for the students and the following companies for their support: John Jones Steel Ltd, D&H Steel Construction Ltd, Grayson

Engineering Ltd, Konnect Fastening Systems. The efforts and help of the technical and laboratory staff at AUT and UoA in conducting experimental testing, are also acknowledged.

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