Seismic application of fillet and partial penetration butt welds

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ABSTRACT

Fillet and partial (incomplete) penetration butt welds are often the most cost-effective weld details for structural steel connections in seismic-resisting systems. Appropriately sized and executed, double-sided, balanced fillet welds and partial penetration butt welds can offer the same performance as complete penetration butt welds under both semi-static and low cycle fatigue (seismic) loadings. The weld sizing criteria are explained in NZS 3404 standard, including for use in seismic connections between members in ductile responding systems. Despite this, there is a misperception among design engineers about the performance of fillet welds under seismic load. Most overseas design specifications still call for complete penetration butt welds to be used in seismic full-capacity connections. This paper presents literature review and test results achieved as a part of HERA Seismic Research Programme in partnership with the Universities of Auckland and Waikato, Auckland University of Technology and the University of Michigan. It demonstrates that the sizing criterion for fillet welds used in the seismic connections currently included in NZS 3404 is conservative, especially the use of the overstrength factor in determining the demand from the principal load path elements in connections results in oversized fillet welds. It also discusses the performance of partial penetration welds as an alternative to fillet welds.

1 INTRODUCTION

As defined in the NZS 3404 standard (Standards New Zealand 1997/2001/2007), a steel connection consists of connection elements, including connection components and connectors at the intersection of two members. A weld is a connector element in a steel connection that works as a link between one member or connection component to another. A primary function of a weld is to effectively transfer the force between the connection elements and members. Weld failure would significantly damage a connection that could trigger the collapse of a part of or whole building and put human life at risk. Accordingly, designing welds to a high level of reliability is essential to prevent connection damage.

Designing economical welds that can be easily fabricated and inspected is very important to improve the productivity and efficiency of steel construction. With the significant use of structural steel in multi-storey commercial and residential buildings, steel fabricators are seeking ways to implement the most cost-effective fabrication. It starts with the use of fabrication-optimised design solutions, including weld details such as fillet welds.

Full penetration butt welds are mostly more expensive than fillet and partial penetration butt welds because they need longer joint preparation and execution time, in many cases back gouging and non-destructive testing. Changing the weld type to partial penetration butt welds and fillet welds for seismic application could save money and time for fabricators. In addition to changing weld type, finding the optimum size of welds could reduce welding operation and material cost and benefit the environment. Numerous research studies investigate the fatigue performance of welds under cyclic loads to explore the governing rules of weld failure and to prevent weld fatigue cracks. The studies concluded that choosing the correct size of the fillet weld prevents weld failure and shifts the cracks to the parent material, which is a desirable failure mode in welded joints (Saiprasertkit 2014; Saiprasertkit et al. 2012; S. Xing et al. 2017; Xing et al. 2016; Shizhu Xing et al. 2017).

NZS 3404 allows 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. According to Clause 12.9.1.2.2 in NZS 3404, fillet and partial penetration butt welds between elements of primary category 1,2 or 3 members or between elements of primary and secondary category 1, 2 or 3 members shall be designed using an overstrength factor for the elements in the principal load-carrying path of the connection where capacity design is not undertaken. This means the nominal section capacity of the member is multiplied by an overstrength factor to obtain the weld design action. Where capacity design is undertaken, the capacity design derived design actions, i.e. overstrength design actions obtained through the capacity design process, are used. Table 12.2.8(1) of NZS 3404 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 size welds, resulted from applying the overstrength factor, are costly, particularly for thick plates.

Similar to other steel design standards, NZS 3404 standard discourages using partial penetration butt welds for seismic applications. Consequently, SCNZ Steel Connection Guide-Part 1 & 2 (Steel Construction New Zealand (SCNZ) 2007) have not included partial butt welds in details of the industry-standard form of welded and endplate moment connections. Clause C12.9.3.2 in Commentary to NZS 3404 standard explains that incomplete penetration butt welds or one-sided fillet welds have an inherently brittle failure under tension loads. This type of failure is undesirable for seismic applications. According to the literature review, the gap or notch between partial penetration butt welds introduces stress concentration to the weld roots, and they are susceptible to fatigue failure and brittle fractures (Kurobane et al. 2004; Saiprasertkit 2014;

Saiprasertkit et al. 2014). However, as mentioned above, some literature indicates that appropriate partial weld detailing can suppress brittle root failure and move the cracks outside the weld.

In the most recent work by Chandramohan et al. (2022), experimental tests along with key findings and research developments about the performance of fillet welded joints under static, cyclic and fire loading have been summarised.

This paper presents a review of the research works conducted by HERA Seismic Research Programme to address both the cost-effective and reliable design of fillet and partial penetration butt welds in the steel construction industry of New Zealand.

1.1 HERA's Seismic Research Programme

In the late 1990s, New Zealand Heavy Engineering Research Association (HERA) directed a 6-year-long research program to evaluate the performance of steel connections under earthquake loads in New Zealand. The program has investigated both economic and technical aspects of connections to make them safer and cost-effective to design and construct. The outcomes were incorporated in HERA Steel Connection Guide, NZS 3404, AS/NZSA 1554.1 and other relevant standards. It has added a degree of resilience to our steel design procedure required to sustain the seismic event without noticeable damage, as witnessed by the excellent performance of structural steel systems in the severe earthquakes of 2010/2011 in Canterbury, 2013 in the upper South Island and 2016 again in the upper South Island and Wellington.

The new HERA Seismic Research Programme has been initiated since 2016. The programme examines welding manufacturing challenges to improve steel construction productivity and employs advanced FEA methods to provide technical solutions and deliver optimum weld sizes. The day-to-day research work has been done by PhD students supported by HERA Foundation together with Master and Undergraduate students. This programme involves cooperation between HERA, the University of Auckland, Auckland University of Technology, the University of Waikato, and international partners. HERA was granted Endeavour Fund from the Ministry of Business, Innovation and Employment with the title "Developing a Construction 4.0 Transformation of Aotearoa New Zealand's Construction Sector" in 2022. Subsequently, some of the HERA Seismic Research Programme's activities have been integrated with the Circular Design Research Sub-programme under the aforementioned granted fund.

2 RESEARCH WORK

2.1 Use of EFPBWs in welded moment connections

The effective full penetration of T-butt weld (EFPBW) is a partial penetration weld introduced in EN 1993-1-8 standard (Europian Standard 2005) (See Fig. 1). According to this standard, EFPBWs could have the same performance as full penetration butt welds under loads if they satisfy two conditions; firstly, the summation of throat thicknesses of welds on both sides be greater than the stem plate thickness (t) and secondly, the maximum allowable gap size between weld roots be the smaller of $t/_5$ and 3mm.

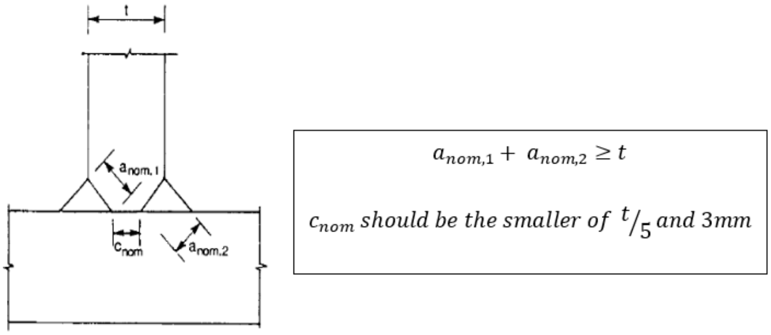


Figure 1: Effective full penetration of T-butt welds from EN 1993-1-8 standard

Three T-shaped large-scale welded moment-resisting connections were tested at The University of Auckland under cyclic loading. The beam flanges were welded to the column flange by EFPBWs following the requirements of EN 1993-1-8 standard. The plate thicknesses of beam flanges were 11 mm, 20 mm and 32 mm for specimens 1, 2 and 3, respectively. Fillet welds were used for welding the beam web to the column flange. The column was 460 UB 74.6 in all specimens.

The test setup configuration, arrangement of strain gauges and loading history was in accordance with SAC protocol (SAC Joint Venture 1997). The actuator applied load to the tip of the beam, and two pinned supports were embedded at each end of the column (See Fig. 2). The beams' dimensions and loading history details have been given in Taheri (2020).

All specimens developed plastic hinges in the beam flanges, dissipating a considerable amount of energy before forming plastic hinges and providing sufficient rotation angle to be qualified for seismic application. Weld failure (root failure) was not observed during the tests in all samples, indicating that the weld details and size of EFPBWs were adequate to suppress weld failure. Indeed, the cracks shifted to the plastic hinge zone or the weld toe at the very end of the cyclic loading regime.

One of the samples was sliced after the test, and the weld details were measured accurately. Also, some T-shaped small-scale samples were made to examine the feasibility of achieving the maximum allowable gap sizes following EN 1993-1-8 standard under production conditions used by New Zealand fabricators. Based on the small- and large-scale test results, the requirements for EFPBWs were updated to be applicable in New Zealand.

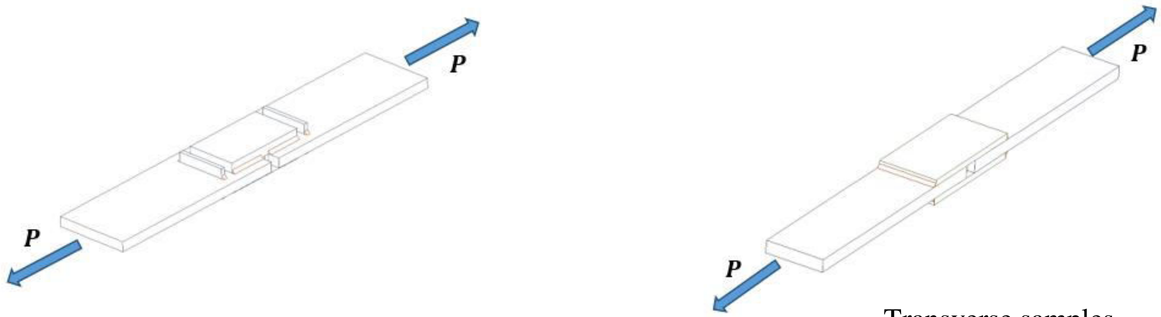
More complete test results and explanations have been given in HERA Report R8-43 (HERA 2021), Taheri (2020) and Taheri et al. (2020).

Figure 2: Test setup based on the SAC protocol

2.2 Finding the property of weld materials for sizing the welds

The objective of this study was to compare the empirical and theoretical values of the shear strength of welds and to check the real angle of weld fracture in shear samples. A more accurate value for weld shear strength could lead to the development of more realistic weld design criteria for fillet and partial penetration welds under cyclic loads. Main steel design codes assume the theoretical shear strength of weld to equal 0.6 multiplied by the ultimate tensile strength of weld derived from the von Mises yield criterion under pure shear conditions. However, this assumption gives a conservative value for the shear capacity of weld when loaded parallel to one leg. Moreover, the steel design standards consider the theoretical throat dimension with an angle of 45° for the fillet welds with equal leg sizes as the critical failure angle. Many studies and experimental tests have shown that it is not a correct assumption when the load is not parallel to the weld length (angle loading conditions) (Lu et al. 2015; Miazga et al. 1989; Nie et al. 2012)

AWS B4.0:2016 standard (American Welding Society 2016) introduces two different specimen types for weld shear testing, including longitudinal and transverse specimens. Figure 3 illustrates the position of welds versus the load direction in each specimen type. The tensile machine should be applied to pull the main plates from both sides until rupture happens in the weld. Then, the weld shear capacity is calculated by dividing the peak load by the weld's theoretical failure shear plane to find the weld's shear capacity.



Longitudinal samples

Transverse samples

Figure 3: Longitudinal and transverse shear weld specimens.

Five longitudinal and five transverse shear weld specimens for each fillet weld size of 6mm and 10 mm (in total, twenty samples) were tested at the University of Auckland. The flux-cored arc welding process was used to fabricate all the samples. More details about specimen dimensions and fabrication have been given in Taheri (2020).

The test results revealed that the empirical value of weld shear strength was greater than the theoretical value ($0.6 \times ultimate\ tensile\ strength\ of\ weld$) for the design weld size for all samples. There was a significant difference between the average value of weld shear strength for longitudinal and transverse samples. Indeed, the average weld shear strength of the longitudinal samples was about 70% and 80% less than that of the transverse samples for fillet weld sizes of 6 mm and 10 mm, respectively. The reason for this inconsistency between weld shear strength of longitudinal and transverse samples could be attributed to the existence of stress concentration at the end of weld in longitudinal welds. Moreover, the transverse weld shear strength was overcalculated because of an unrealistic theoretical throat thickness dimension, which was not on the plane of 45 degrees. According to the experimental tests, the average angle of fracture in welds was 52 and 18 degrees for longitudinal and transverse samples, respectively (see Fig. 4). This result was consistent with the findings of other studies (Lu et al. 2015; Miazga et al. 1989; Nie et al. 2012). The study recommended using transverse samples with real weld fracture angles to calculate the shear strength of welds and to investigate more closely the effect of the actual weld size compared with the idealised weld size. This includes the effect of penetration of the weld face into the parent metal and the actual leg lengths being greater than the specified leg length.


Longitudinal shear weld specimens


Transverse shear weld specimens

Figure 4: The typical weld failure for longitudinal and transverse specimens.

In another research carried out by Forster (2022), twenty-five transverse shear weld samples and twenty-one round-shaped weld tensile samples were tested at the University of Auckland. AWS B4.0:2016 standard was followed for making and testing the specimens. Two fabricators and different welding processes were employed in the fabrication of samples. The tests aimed to compare the empirical and theoretical weld shear strength and examine the effects of production conditions on the weld strength.

The results indicated that the shear strength established based on testing of transverse samples made by both fabricators was greater than the theoretical values for the specified weld size estimated based on the von Mises criterion. Table 1 illustrates the ratio of the lower second standard deviation of transverse weld shear strength to the average ultimate tensile strength for each welding process.

As is evident from the results in Table 1, the average value of this ratio for all welding processes is about 1.0. Indeed, welds' shear strength is approximately equal to their tensile strength. However, this ratio for cruciform samples (the tests will be explained in the next section) were 0.74, 0.68 and 0.63 for 6 mm, 8 mm and 12 mm fillet welds, respectively. It demonstrated that the weld sizes, multiple passes of welding and sample configuration (or weld confinement conditions) would affect the results of the shear strength of welds. Subsequent measuring of the actual weld size and profile showed that both these give consistently more weld area than the area generated by the minimum specified weld size and profile, which is a significant contribution to the increased strength observed.

Table 1: Transverse Shear vs tensile averages strength for each welding process

	2LSD transverse Shear strength/ Average UTS
Metal Core (Fabricator 1)	0.91
Gasless FluxCore	0.93
ARC	1.08
Solid Wire	1.18
Metal Core (Fabricator 2)	0.89

The research highlighted the impact of having good welding quality systems and following a correct welding procedure on having consistent weld shear strength. Also, the average real fracture angles in welds for all transverse shear weld samples (the samples with weld defects excluded) was 20.56 degrees, which was close to the results of the first series of tests done by Taheri (2020).

2.3 Small-scale tests of cruciform joints made by optimum size of fillet and partial penetration butt welds

Multiple cruciform joints were fabricated by fillet and partial penetration welds to be tested at the University of Auckland. A new weld sizing criterion was developed to find the optimum sizes of fillet and partial penetration butt welds. The objective was to evaluate the performance of the welds under cyclic and static loading. The first group of samples was tested by Forster (2022) under static loading, and the second group with identical weld details will be tested soon under cyclic loading.

The new weld sizing criterion was developed analytically in collaboration with Prof. Pingsha Dong's team at the University of Michigan. The formula consists of different parameters as below;

$$2 \times \tau_{u\,(WM)} \times a_{\theta(crit)} \geq F_{u\,(BM)} \times t \times S.F. \tag{1}$$

where,

t is the stem plate (loaded plate) thickness in the cruciform joints

$\tau_{u\,(WM)}$ is the ultimate shear strength of the weld material (from transverse weld shear samples)

$F_{u\,(BM)}$ is the ultimate tensile strength of the base metal (from ultimate tensile tests)

$a_{\theta(crit)}$ is the actual throat dimension in the critical angle

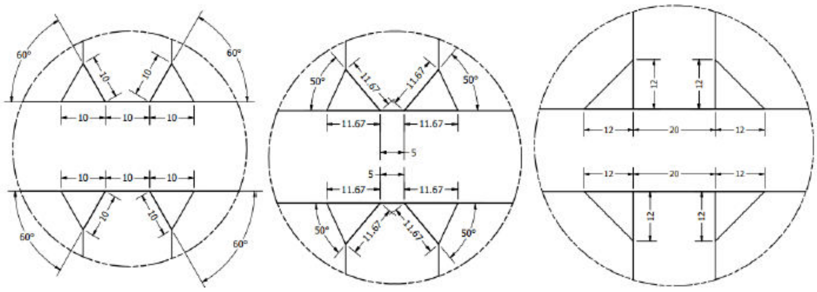
$S.F.$ is the safety factor

This criterion modifies some pre-accepted assumptions in the steel design codes to achieve more precise weld dimensions. The actual angle of weld failure is calculated using the finite element solution and traction stress method (Dong 2003) to find real weld throat dimensions instead of theoretical throat size. Also, the transverse shear weld samples were tested to obtain the real shear capacity of the weld in lieu of the theoretical value of $0.6 \times F_{uw}$ where F_{uw} is the nominal tensile strength of weld metal.

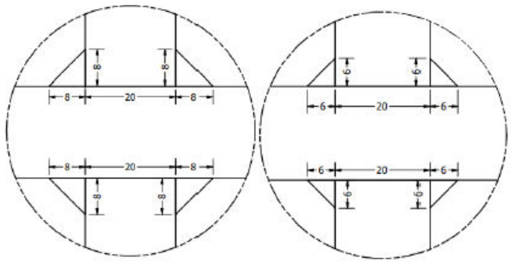
The provided formula for fillet weld sizing in the tests was based on the full capacity of the members in transverse shear load conditions, and finite element analysis is required to recognise the actual fracture angle in the welds. A similar formula for quantitative fillet weld sizing criterion without the need for finite element analysis for longitudinal and transverse shear conditions was presented in the HERA webinar by Dong (Dong 2021).

Indeed, as long as inequality (1) above is satisfied, failure happens outside the weld into the loaded plate. The safety factor in this research was considered to be 1.0. The stem plate thickness (loaded plate thickness) in all cruciform joints was 20 mm.

Six cruciform joints with an optimum fillet weld size of 12 mm were made for testing under static loading. In addition, six samples for each undersized fillet weld sizes of 6 mm and 8 mm were fabricated to check the mode of failure of samples, which would be predicted to be weld failure for all these samples. Also, six cruciform joints for each partial penetration butt weld with 5 mm and 10 mm gap sizes (between the roots of welds) and with optimum weld dimensions were made for the tests. Two fabricators were used to make all the samples with the metalcore weld process. The designed weld details have been illustrated in Figure 5.



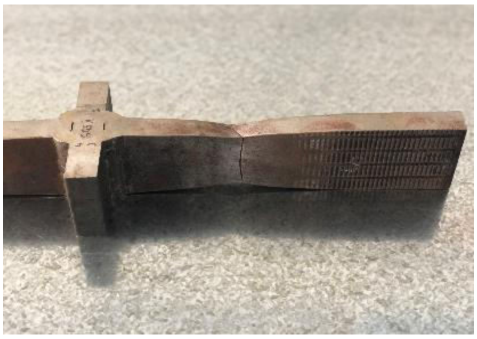
Optimum weld details from left to right, 10 mm gap PPBW, 5 mm gap PPBW, 12 mm fillet weld



Undersized fillet weld details, from left to right 8 mm and 6 mm fillet welds

Figure 5: Weld profiles for cruciform joint samples

According to the test results, loaded plate failure occurred for all cruciform joints with partial penetration butt welds, and weld fracture was the failure mode for all the cruciform joints with undersized fillet welds. However, inconsistency was observed in the failure mode of samples made by 12 mm fillet welds. Two cruciform joints out of six samples failed in the loaded plates. The reason for this inconsistency is still unclear. However, it could be attributed to the lower shear strength of larger welds made by multiple welding passes, as mentioned in the previous section. The results indicated that the safety margin in the weld sizing criterion should be increased for sizing fillet welds. The mode of failure of samples has been illustrated in Figure 6. For more information about the tests, refer to Forster (2022).



Loaded plate failure in samples made by partial penetration welds

Weld failure in some samples made by fillet welds

Figure 6: Mode of failure in cruciform samples

2.4 Seismic tests of replaceable active links made by fillet and partial penetration butt welds

Replaceable active links are getting more popular to be used as seismic members in eccentrically braced frames. The increasing application of replaceable active links can be attributed to simple substitutability by unbolting the damaged link after an earthquake. They also give more freedom to designers for member selection due to discontinuity from the collector beam. However, they are expensive seismic members because their fabrication involves many details and welding efforts. This is especially true when a custom welded I section should be fabricated for a replaceable active link.

The main aim of this research work was to examine the feasibility of replacing full penetration butt welds with the optimum size of fillet and partial penetration welds in order to make fabrication more cost-effective. Consequently, four pairs of active links with grade 300 structural steel plates were fabricated and tested at the University of Auckland. The length and dimensions of all replaceable active links were identical. The welds for the first pair of samples were designed following NZS 3404 standard and HERA P4001:2013 report without involving overstrength factors. As explained in the previous section, finite element solutions and the proposed weld sizing criterion were used to find optimum fillet welds between the active link flanges and endplates in the second and third pair of samples. The same approach was followed to find the optimum fillet weld size between the web and flange of active links in all samples, together with the web and endplate welds in the third paired samples. Effective full penetration butt welds were designed for the weld between active link flanges and endplates in fourth pair samples.

Figure 7 demonstrates the test setup for active links. One side of the active links was fixed to the stub columns, and the cyclic loading was applied to the other side using a 1000 kN actuator. With some modifications, the pseudo-statically cyclic loading was extracted from Richards et al. (2006).

Figure 7: Replaceable active links test set up

The failure mode in all samples was web buckling followed by developing cracks initiated from the middle of webs at the centre of the active links. Plastic hinging was observed in the flanges of the active links about a central quadrant of the buckled web, as is evident in Figure 8.

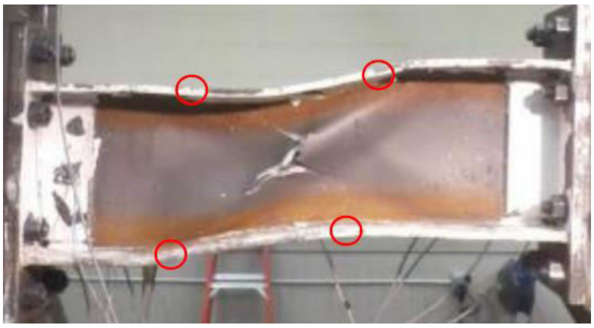


Figure 8: Specimen after the test

No weld failure was observed during the tests until the rupture developed to the longitudinal welds between the web and flanges of active links at the very end of the loading regime, with a high number of cycles. It indicated the weld performance was acceptable, and weld details in all samples were adequate to prevent premature failure of active links. For more information about the tests, refer to Chan (2022).

3 ONGOING RESEARCH PROJECTS

In continuation of previous research work to understand the behaviour and strength of fillet and partial penetration welds in the loaded joints, HERA has been involved in some other research projects. A series of T-shaped samples designed by different fillet weld sizes were fabricated for testing at Auckland University of Technology. The test setup is being prepared, and the experiments will commence soon. These tests aim to investigate the possibility of revising overstrength and safety factors value to reduce the inherent conservatism that has been included in the weld sizing criterion in the NZS 3404 standard. These experiments will be undertaken using cyclic loading protocols.

In other research work, a group of T-shaped samples, representing the joint between beam flanges and high-strength steel endplates in endplate moment connections, were fabricated and will be tested soon. One of the objectives of this research is to examine the performance of fillets and effective full penetration butt welds in this joint.

Moreover, as mentioned before, a series of cruciform samples made by fillet and partial penetration welds should be tested under cyclic loading in future. Also, two pairs of active links made by full penetration butt welds using the backing bar for the joint between active link flanges and endplate will be tested at the University of Auckland. The test setup and specimen dimensions are identical to the tests in Section 2.4. The main objective of the tests is to evaluate the seismic performance of active links and welds while keeping the backing bar in place instead of removing it.

4 CONCLUSIONS

Fillet and partial penetration butt welds sized to NZS 3404 and executed to AS/NZS 5131 and Construction Category 3 are adequate and conservative for use in the seismic connections. These details should be specified in lieu of full penetration butt welds where possible and/or practical. Fillet welds are also more suitable for the application of robotic welding technologies. Specifying the optimum size of welds will further reduce fabrication time and cost.

The test results demonstrated that the current assumptions in NZ steel design codes could be modified to yield less conservative weld sizes. The weld sizing criterion in NZS 3404 standard involves theoretical shear weld strength, safety factor, overstrength factor and theoretical throat size. The effects of the angle of loads with respect to the weld length are also ignored in the provided formula versus other principal design codes. Ignoring angle loading could result in about 10% larger weld sizes by using NZS 3404 standard in transverse loading conditions. However, working with the weld sizing criterion in NZS 3404 standard is less complicated than other standards for designers. Future cyclic tests of cruciform and T-shaped samples aforementioned in this paper could lead to more realistic values for safety and overstrength factors. Given that as made weld area is typically significantly greater than the minimum specified weld area, there is also serious scope for reviewing the current strength reduction factor of 0.8 for these welds to determine if a less conservative (i.e. larger value) factor is appropriate. That will be a focus of future work and any changes will require more investigation into the effects of weld gap and weld misalignment on the behaviour of these welds when connecting elements subject to inelastic cyclic action.

Effective full penetration butt welds, as described in HERA report R8-43, give conservative partial penetration weld sizes. Test results proved the adequate performance of this type of weld for seismic applications. EFPBWs are likely more economical for plate thicknesses between 25 mm to 40 mm, depending on weld and preparation techniques, than fillet and full penetration welds.

Overall, improving current weld sizing criteria is important for more cost-effective weld sizes. Alternatively, establishing easy-to-use tables with optimum fillet and partial penetration welds for seismic connections could also mitigate high fabrication costs and time.

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6 REFERENCES

American Welding Society 2016. *Standard methods for mechanical testing of welds (AWS B4.0: 2016).* Danvers, MA, United States: Approved by the American National Standard Institute (ANSI).

Chan, A. 2022. Cyclic testing of built-up, extended end plated replaceable links for more cost-effective weld design (Master Thesis). The University of Auckland, Auckland, New Zealand.

Chandramohan, D. L., Roy, K., Taheri, H., Karpenko, M., Fang, Z., & Lim, J. B. P. 2022. A State of the art review of fillet welded joints. *Materials, 15*(24), 8743.

Dong, P. 2003. A robust structural stress method for fatigue analysis of ship structures. Paper presented at *the ASME 2003-22nd International Conference on Offshore Mechanics and Arctic Engineering*, American Society of Mechanical Engineers.

Dong, P. 2021. Principles and methods for design of welded structures . *Welding Innovations Network*, HERA's Webinar, Manukau City, New Zealand.

Europian Standard. 2005. *Eurocode 3: Design of steel structures Part 1-8: Design of joints (EN 1993-1-8).* Brussels, Belgium: Approved by European Committee for Standardisation (CEN).

Forster, M. 2022. A new weld sizing criteria for fillet welds and partial penetration butt welds with the use of traction shear stress method (Master Thesis). *The University of Auckland*, Auckland, New Zealand.

HERA. 2021. The use of effective full penetration of T-butt welds in welded moment connections. In *HERA Report R8-43*, Manukau City, New Zealand.

Kurobane, Y., Azuma, K., & Makino, Y. 2004. Applicability of PJP groove welding to beam-column connecions under seismic loads. *Connections in Steel Structures V*.

Lu, H., Dong, P., & Boppudi, S. 2015. Strength analysis of fillet welds under longitudinal and transverse shear conditions. *Marine Structures, 43*, 87-106.

Miazga, G. S., & Kennedy, D. J. L. 1989. Behaviour of fillet welds as a function of the angle of loading. *Canadian Journal of Civil Engineering, 16*(4), 583-599.

Nie, C., & Dong, P. 2012. A traction stress based shear strength definition for fillet welds. *The Journal of Strain Analysis for Engineering Design, 47*(8), 562-575.

SAC Joint Venture. 1997. *Protocol for fabrication, inspection, testing, and documentation of beam-column connection tests and other experimental specimens (Rep. No. SAC/BD-97)*. In *Rep. No. SAC/BD-97*. Sacramento,California: SAC Steel Project.

Saiprasertkit, K. 2014. Fatigue strength assessment of load-carrying cruciform joints in low- and high-cycle fatigue region based on effective notch strain concept. *Welding in the World, 58*(4), 455-467.

Saiprasertkit, K., Hanji, T., & Miki, C. 2012. Fatigue strength assessment of load-carrying cruciform joints with material mismatching in low- and high-cycle fatigue regions based on the effective notch concept. *International Journal of Fatigue, 40*, 120-128.

Saiprasertkit, K., Sasaki, E., & Miki, C. 2014. Fatigue crack initiation point of load carrying cruciform joints in low and high cycle fatigue regions. *International Journal of Fatigue, 59*, 153-158.

Standards New Zealand. 1997/2001/2007. *Steel structures standard (NZS 3404:Part 1 and 2:1997).* Wellington, New Zealand: Approved by the Steel Structures Committee (P 3404) for the Standards Council.

Steel Construction New Zealand (SCNZ) 2007. *Steel Connections Guide - Part 1 & 2- Steel Connect (SCNZ 14.1:2007; SCNZ 14.2:2007)*. Manukau City, New Zealand.

Taheri, H. 2020. Seismic Evaluation of Welded Moment-resisting Connections (PhD Thesis). *The University of Auckland*, Auckland, New Zealand.

Taheri, H., Clifton, G. C., Dong, P., Karpenko, M., Raftery, G. M., & Lim, J. B. P. 2020. The use of effective full penetration of T-butt welds in welded moment connections. *Welding in the World, 64*(9), 1503-1519.

Xing, S., & Dong, P. 2017. A fatigue failure mode transition criterion for sizing load carrying fillet welded connections. In Z. W. Nikbin, K., P. C. McKeighan, & D. G. Harlow (Eds.), *Fatigue and Fracture Test Planning, Test Data Acquisitions and Analysis* (Vol. ASTM STP1598). West Conshohocken, PA: ASTM International.

Xing, S., Dong, P., & Threstha, A. 2016. Analysis of fatigue failure mode transition in load-carrying fillet-welded connections. *Marine Structures, 46*, 102-126.

Xing, S., Dong, P., & Wang, P. 2017. A quantitative weld sizing criterion for fatigue design of load-carrying fillet-welded connections. *International Journal of Fatigue, 101*, 448-458.