Experimental Study of Fillet Welds for Seismic Actions

S. Ramhormozian, M. Zhang
Auckland University of Technology, Auckland, New Zealand.

C. Clifton
University of Auckland, Auckland, New Zealand.

M. Karpenko, H. Taheri
Heavy Engineering Research Association, Auckland, New Zealand.

ABSTRACT

The New Zealand Steel Structures Standard NZS3404 uses a simplified method to predict the design capacity of fillet welds and hence to size the weld. From current research into the performance of these fillet welds under transverse and longitudinal loading to failure, the current weld design provisions are likely to be conservative compared with those in overseas high seismically active countries of comparable technical capability in fabrication. This paper describes a research project to identify the actual capacity of the fillet welds, as the most common type of welds, by experimental cyclic testing, to propose more realistic design provisions for fillet welds in seismic resisting systems compared to those specified by NZS3404. This is a collaboration between AUT, UoA, HERA and the University of Michigan, with the aim of making the seismic fillet welds design and fabrication more cost-effective while ensuring the continuing adequate performance of these welds, particularly under severe seismic actions.

The experimental cyclic testing is being conducted at AUT labs. These include fillet weld samples of different sizes being loaded cyclically in the elastic and post-elastic range using dynamic hydraulic actuators. This paper provides information about the test setup, specimens' configurations, and cyclic loading regime along with the results and recommendations.

1 INTRODUCTION

A weld serves as a connector element within a steel connection, facilitating the linkage between one member or connection component and another. The role of a weld is to efficiently transmit forces between structural members. Any failure in welding could result in damage to a connection, potentially leading to the collapse of a section or the whole building. Therefore, designing welds with a high degree of reliability is essential.

Enhancing the productivity and efficiency of steel construction depends to a significant extent on designing welds that are not only economical but also easily fabricated and inspected. Given the extensive utilization of structural steel in multi-storey commercial and residential buildings, steel fabricators are actively exploring strategies to streamline fabrication processes while maintaining cost-effectiveness. Full penetration butt welds typically incur higher costs compared to fillet welds due to their requirement for lengthier joint preparation and execution time and non-destructive testing (Taheri, Karpenko et al. 2023). For seismic applications, using double sided fillet welds instead of full penetration butt welds may result in significant savings in both time and expenses for fabricators. This may also have a positive influence on environmental sustainability.

New Zealand steel standard (NZS3404 1997/2001/2007) recommends a simplified method to size the fillet welds. The nominal capacity of fillet welds per unit length of the weld ($\boldsymbol{V_w}$) are calculated as $\boldsymbol{V_w=0.6f_{uw}t_tk_r}$, in which $\boldsymbol{f_{uw}}$ is the nominal tensile strength of weld metal; $\boldsymbol{t_t}$ is design throat thickness on angle 45° for an equal leg length fillet weld, regardless direction of the imposed force on weld; and $\boldsymbol{k_r}$ is the reduction factor for the length of a welded lap connection. The factor $(1/\sqrt{3})=0.58\approx0.6$ reflects the fact that the von-Mises yield criterion (pure shear failure along the design throat thickness) is adopted when developing the above-mentioned weld sizing equation. It is worth noting that von-Mises yield criterion predicts when the material exceeds its elastic limits and is not primarily developed to predict the ultimate stress and/or rupture state of the material. Hence, adopting von-Mises yield criterion but using ultimate tensile strength (instead of tensile yield stress) of the material reflects assumptions and approximations considered and which are appropriate for material that behaves in a ductile manner in the beyond yield range (Picón and Cañas 2009).

The aim of this research is to examine the accuracy of the NZS3404 method of sizing fillet welds under seismic actions with the aim of optimizing and modifying it to make fillet welds more cost effective without affecting their performance. This includes testing different sizes of fillet weld in t-stub connections under cyclic loading ranging from elastic to inelastic capacities of the connections. This paper briefly explains the experiments along with their results.

2 TEST SAMPLES

The tests were conducted with the assumption that the test specimens represent the connection between the flange of a steel beam and the column flange in a moment resisting frame welded beam-column connection. The beam's flange is represented by a web plate with dimensions of 60mm × 16mm, while the column's flange is represented by a thick flange base plate with dimensions of 60mm × 60mm. This represents the very rigid column base support, through the column flange and into tension/compression stiffeners to take the out of balance, moment induced beam longitudinal actions into the column. The test includes four groups of samples, each featuring different fillet weld sizes: 8mm, 10mm, 12mm, and 16mm. Additionally, two more groups of samples are included, with weld size of 10mm and 12mm, in which 1.5mm and 3mm gaps were introduced. Hence in total, there were six groups of samples, with each sample group tested with five repeats. Figure 1 below shows the tee-stub tested drawings.

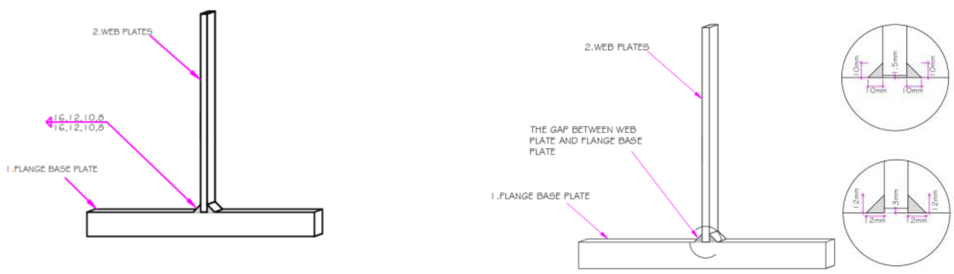

Figure 1: drawings of the test samples; samples without gap (left) and samples with gap (right)

3 TEST SETUP

The test setup including a specimen (See Figure 2) consists of six main parts, i.e., two clamping angles, one Web-plate, one Flange base plate, and two cover plates. The web-plate is welded to the flange base plate which together make a sample (a T-stub sample shown in figure 1). The clamping angles are connected to the actuator's endplate through their short legs each one by 2×M30 High Strength Friction Grip (HSFG) Property Class (PC) 8.8 bolts and connected to the Web-plate through their long legs by 10×M24 HSFG PC8.8 bolts (and for some experiments by M20 PC10.9 bolts). The Web-plate is welded to the middle of the Flange base plate. The flange base plate is covered by the cover plate at both sides. The cover plates also cover the backup plates. The cover plate is connected to the strong floor by 2×M36 HSFG PC8.8 threaded rods and nuts. The test setup, excluding the specimens, is designed to remain elastic at 1000kN force which is the maximum force that may be imposed by the dynamic actuator.

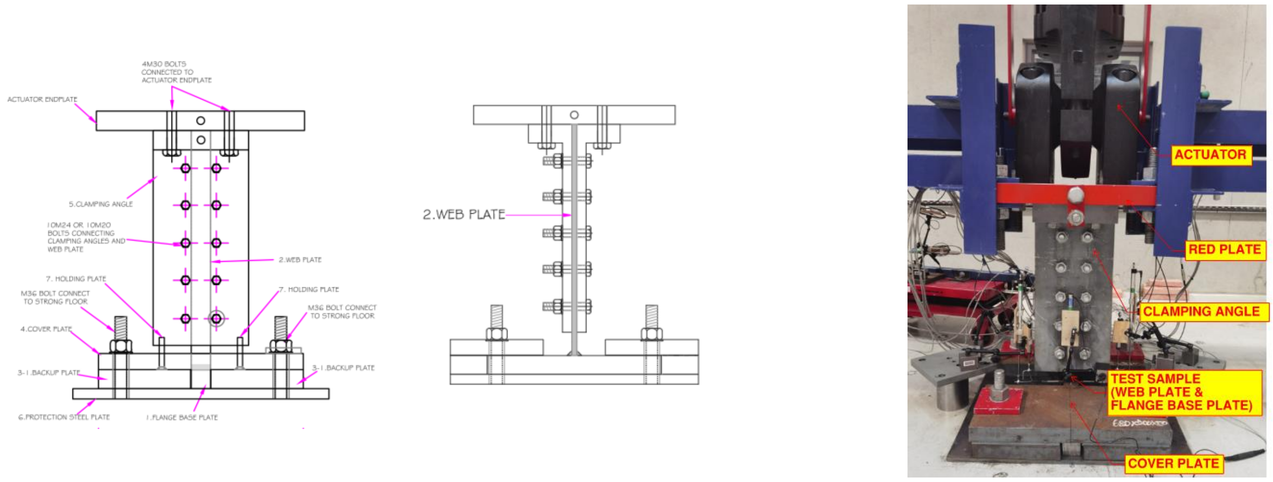

Figure 2: the test setup: drawings of two side views (left) and actual setup (right)

4 LOADING REGIME AND INSTRUMENTATION

The loading regime adopted for the tests is derived from (AISC 2016), with modifications made to fit the specific purpose of these tests. The original loading regime procedure falls under the section for prequalification and cyclic qualification testing provisions of beam-to-column moment connections. The loading protocol outlined in (AISC, 2016) involves a multiple-step test-loading history. In this protocol, the deformation is defined as a drift, which had to be converted into axial elongation of the stem. Subsequently, the simultaneous displacement of the actuator was determined. A trial test was conducted to calibrate the displacement time history. Eventually 13 levels of elongation were defined associated with small interstorey drifts (0.00047 Radian) up to very large interstorey drifts (0.04 Radian) each one with few repeating cycles. This was to ensure the samples were tested cyclically in the elastic (linear range) and then suitably up into the inelastic (non-linear range. Figure 3 shows the cyclic loading regime used in the experiments.

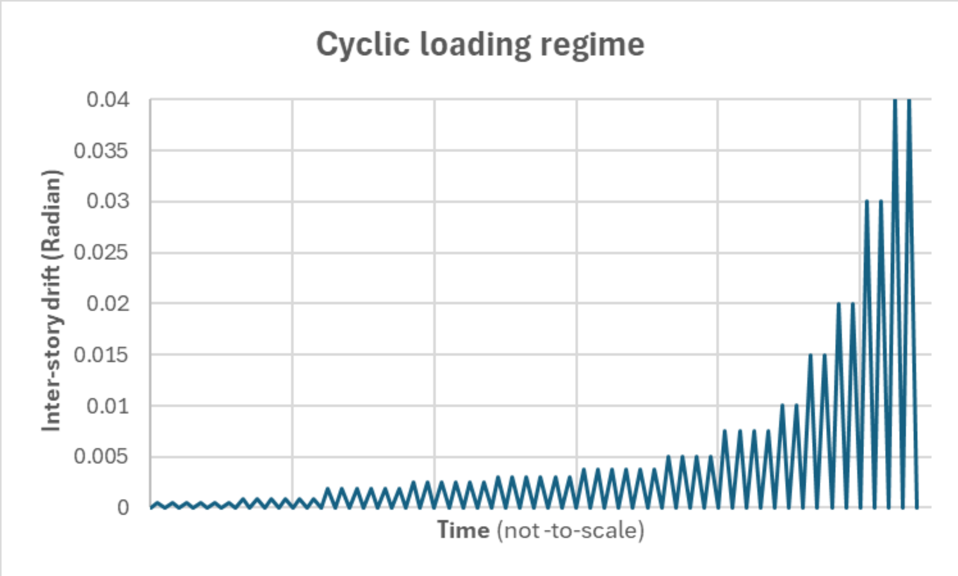

Figure 3: Cyclic loading regime used in the experiments

For the instrumentation, the strain gauges, LVDTs, load cells, and cameras were deployed to record as comprehensively as possible the data, which was collected by the built-in FlexLogger software of the National Instruments Data Acquisition System (DAQ). The key measurement remained the 1000kN MTS dynamic actuator's built-in load cell, which recorded the load imposed on the weld. One time scale and zero starting point was used for all channels.

5 OBSERVATIONS, RESULTS, AND DISCUSSION

All of the samples were physically inspected prior to the cyclic tests. To measure the dimensions of the samples, tools such as the steel ruler and Digital Vernier Caliper Gauge were utilized. To document the weld dimensions, both leg lengths and throat were measured, each comprising six points' readings. These points were positioned 10mm from each edge of the weld, with one additional point at the centre of the weld. Etching tests were conducted on both sides of the samples prior to the cyclic tests. Two methods were employed to determine the throat thickness of the samples, one based on physical dimension measurements and the other one based on the photos taken from the etched surface of the welds.

Three typical test termination modes were observed, as follows;

- Failure occurred due to fracture in the weld.
- Failure occurred due to fracture in the web.
- Termination of the test due to sample slipping (note that the means of transferring load from the actuator to the specimens was friction).

All the samples with weld fractures were either equal to or less than 10mm and there were no instances of fractures in samples with 12mm and 16mm weld sizes. Upon comparing the ratio of actual capacity to theoretical capacity, it became evident that for these samples, the actual weld capacity remains significantly higher than the theoretical capacity predicted by the NZS3404:1997. The fracture angles were primarily observed to range from 13 ° to 33°. This is notably different from the assumed fracture surface angle used in the current weld design according to NZS3404, which is set at 45°.

All the samples that fractured in the stem (i.e. web plate), and not weld, had weld sizes of 12mm and 16mm. When comparing the ratio of actual demand to theoretical capacity, all sample ratios were over 1.2, except for the 16mm weld size samples, however it is worth noting that given the fracture occurred in the stem and not the weld, it is not possible to precisely identify the weld fracture load (capacity) of such samples, but only to conclude they are stronger than the maximum recorded values from the tests. These ratios also demonstrate that the actual capacity of the weld, as determined in this test, is higher than the theoretical capacity calculated from the weld design provisions in NZS3404:1997.

As for the samples in which slipping occurred, this was because the stem began buckling inelastically, and the bottom of the clamping angles experienced a horizontal load expanding/separating them while resisting against the inelastic buckling of the stem. In most cases, stem buckling indicates that the stem was loaded beyond its yield capacity. In such tests, the deformations experienced by the samples are expected to be significantly more severe than a weld in a seismic resisting system during a severe earthquake. It is also worth noting that given the tests were terminated due to sliding, it is not possible to precisely identify the weld fracture load (capacity) of such samples but to conclude they are stronger than the maximum recorded values (i.e. sliding force) from the tests. When comparing the actual weld demand with the theoretical capacity of the weld for these samples, the actual weld demand was found to be larger, even with tests being terminated due to sliding, except for three cases. Considering the overall test results and the weld size for these samples, it is highly probable that the 16mm weld size sample would be strong enough to avoid fracturing in the weld. The ratios of actual demand to theoretical capacity also demonstrate that the actual capacity of the weld is expected to be significantly higher than the theoretical capacity derived from the weld design in NZS3404:1997.

To sum up the key findings, the actual capacity of the welds compared with their theoretical design capacity according to NZS3404 (i.e. theoretical capacity of a weld size equal to the tested weld according to the measurements of each sample), for the samples that were fractured in welds (hence the actual weld capacity could be calculated) was 1.8 to 2.2 times higher than the theoretical capacity. This ratio was 1.3 to 2.1 for the samples that fractured in stem (note that the weld capacity is higher than this as it is not fractured).

All these suggest that the tested welds exhibited significantly higher capacity compared with that predicted by the NZS3404. While this suggests potentials to modify the NZS3404 design equations for fillet welds, it is worth noting that the NZS3404 design provisions for fillet welds are not direction-dependent, meaning that different directions of the imposed load (e.g. transverse or longitudinal) are regarded the same. However, in fact the direction of the load does influence the weld capacity. For example, (Kato and Morita 1974) reported that under static conditions the transversely-loaded fillet weld capacity are of the order of 40% greater than those for longitudinally loaded fillet welds. There has been further research looking into the load direction influence on the fillet welds capacity e.g. (Lu, Dong et al. 2015). Hence careful considerations are needed to recommend changes to NZS3404 design provisions for fillet welds.

6 CONCLUSIONS

From the cyclic test results, it is evident that the fillet weld fracture did not occur in these experiments when the weld size was equal to or greater than 12mm for the 16mm thick stem used in these experiments. All experiments that led to weld fracture suggested significantly higher capacity for the weld compared with that recommended by the NZS3404. There was no experiment suggesting the welds are under designed following NZS3404. Hence, it is recommended through this research, to consider potential modifications to the NZS3404 design recommendations to possibly making them less conservative and more cost-effective. This may be achieved through careful considerations and analysis of the results of this research and previous research results which is underway by the authors.

7 ACKNOWLEDGEMNTS

Generous support and kindness received from John Jones Steel for donating and shipping the samples is much appreciated. Support and kindness of Heavy Engineering Research Association (HERA) and Konnect Fastening Systems is also acknowledged. The support from New Zealand Ministry of Business, Innovation and Employment (MBIE) through an Endeavour Fund for the Research Programme (Sustainable Earthquake Resilient Buildings for a Better Future - PROP-83779-ENDRP-AUT) is greatly appreciated. The efforts and help of the technical and laboratory staff at AUT as well as PhD student Mr Kevin Yip, helping in conducting experimental testing is also acknowledged.

8 REFERENCES

AISC (2016). Seismic provisions for structural steel buildings, American Institute of Steel Construction.

Kato, B. and K. Morita (1974). "Strength of transverse fillet welded joints." Welding Research Supplement 59s-64s.

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

NZS3404 (1997/2001/2007). Steel structures standard, incorporating Amendments 1 and 2, Wellington [N.Z.]: Standards New Zealand.

Picón, R. and J. Cañas (2009). "On strength criteria of fillet welds." International Journal of Mechanical Sciences 51(8): 609-618.

Taheri, H., M. Karpenko, C. Clifton, S. Ramhormozian, P. Dong, J. Lim and K. Roy (2023). "Seismic application of fillet and partial penetration butt welds."