IMPROVED WELDED CONNECTIONS FOR EARTHQUAKE LOADING

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SUMMARY

This paper presents an overview of fracture tests performed on welded samples representing the column/beam connection details of typical New Zealand medium-rise structural steel buildings. Material combinations and weld design alternatives have been investigated within the range typically used in a Tee-stub joint representing the welds between the beam flange and the column flange in a beam flange to stiffened column flange connection. Small-scale tests to investigate the principal influences of differing material properties and weld design alternatives on joint performance are described, as well as the first five large-scale tests to verify the findings of the small-scale tests.

INTRODUCTION

Moment-resisting connections, as found in plant, equipment and steel structures in the form of connections, are often required to perform under low cycle, high strain rate, inelastic conditions. Two recent earthquakes in the USA (Northridge, January 1994) and Japan (Kobe, January 1995) have shown that common moment-resisting connection details in both mechanical plant and buildings are prone to failure.

This research is part of a New Zealand Heavy Engineering Research Association (HERA) long-term program with the overall aim to built safer and more cost-effective structures. This paper describes the first two years of research of the project part aimed to determine the relationship between the dynamic performance of typical connection details used in New Zealand and variations in the quality of weld. The research, described in detail in [1], is complementing another seismic program of the structural division of HERA which is investigating alternative moment resisting connection details [2,3,4]. As the research work is still at an early stage and continuing the conclusions are still of a preliminary nature.

PROGRAMME OUTLINE

The research program described herein was structured into different tasks, following a review of literature from the wave of research initiated as a consequence of the Northridge and Kobe earthquakes, to investigate the adequacy of aspects of the current philosophy for earthquake resistant design in New Zealand [5]. These tasks were:

Designing or selecting a typically used Tee stub joint representing a beam flange to stiffened column flange connection, with the welds between the beam flange and the column flange.

Determining selected mechanical properties (yield, tensile, elongation, fracture toughness) for material representing the typical yield/ultimate strength ratio at the higher and lower end of the specification of a typical grade of carbon steel.

Performing fracture tests on small-scale samples, representing the column/beam connection detail, for material combinations and weld design alternatives selected.

Determining the influence of material property and weld design variations on performance of the small-scale test results.

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Performing large-scale tests to determine comparability of small-scale test results with large-scale tests.
Developing and testing alternative welded beam flange to stiffened column flange moment resisting connections, leading to an improved performance and/or lower fabrication cost through incorporating the findings of the above research tasks.

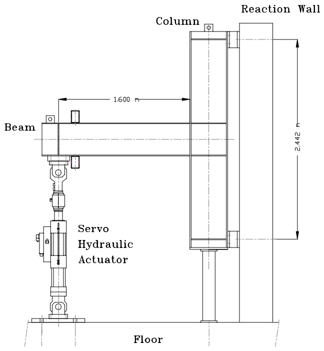

Figure 1: Large-scale test setup

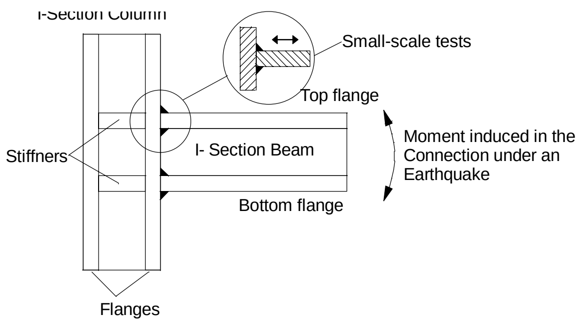

Figure 2: Small-scale test sample location within a beam/column connection

Small-Scale Test Joint Detail

In order to obtain fundamental information on the performance of the large-scale samples at lower cost, a small-scale test was devised to be performed on a standard dynamic testing machine (MTS 810). As shown in figure 2, the small-scale test represents a beam flange to column flange weld. While the large-scale test applies a moment in the connection, as expected in an earthquake, the small-scale test representing only one flange was tested under fluctuating tensile load only. It is noted in the literature e.g. [7] that up-scaling of results from small-scale samples to full scale structures is not generally applicable. However, in this case, the section thickness is kept the same thickness and only the complexity of the connection detail is reduced through the Tee-stub assemblage tested (Figure 3).

Material Combinations

U-beams

The supplier of the UBs provided data of the typical distribution of yield and yield/tensile strength for the two UBs selected. Yield strength ranges from 325MPa to 375MPa, with a range of yield/tensile ratio from .64 to .72. The testing program reflects this variation in the strength range of a particular steel grade, by testing samples of lower and higher strength in the range.

Plate for small-scale tests

Due to limitations in the capacity of the available tensile testing machine, the plate representing the beam flange was chosen to be of a slightly lower strength than the comparable beam flange. This was to avoid the need to further reduce the sample width which would cause a subsequent stronger influence of end effects. Similar to the considerations for the yield strength range of the beam, two plates representing the lower and higher yield strength of the typically supplied plate range have been selected (Table 1).

Welding consumables

The intention was to test a wide range in strength of commonly available welding consumables. The closest match of weld to parent metal strength was through a special Manual Metal Arc Welding (MMAW/SMAW) electrode of the E35 type. Table 2 lists mechanical properties and chemical composition of welding process and consumables used in the different tests.

Joint Design Alternatives

Double-sided fillet welds and complete penetration butt (groove) joints have been the basis of this comparison. Butt welds design typically asks for full penetration joints to match the strength of the beam. This is usually achieved with welds equal in size to the parent metal thickness. This is based on the typical assumption that weld metal is overmatched in strength and therefore a sufficient margin of safety against fabrication imperfection is present.

In the case of fillet welds, a considerable strength overmatch is designed into the joint. For example NZS 3404 [5] requires a strength overmatch factor of 1.25 times the specified minimum beam strength. Additionally a strength reduction factor is considered to cover weld imperfections in line with the specified weld category.

Table 1: Chemical composition and mechanical properties of steel plates representing beam

Position in Strength Range	lower	higher
Standard and Grade	AS 1594 HA 250	AS 1594 300 MOD
Specified Min. Yield/Tensile [MPa]	250 / 350	
Measured Yield /Tensile [MPa]	309 /471	353 / 505
Y/T-Ratio	0.656	0.699
Elongation [%]	23	27

Table 2: Chemical composition and mechanical properties of weld consumables

Welding Process	MMAW	GMAW	GMAW	GMAW	FCAW self -shielded
Name in Text	lower strength	standard strength	higher strength	very high strength	Standard strength
Electrode Class Min. Yield/Tensile [MPa]	EN 499 E35 355 / 470	ER70 400 / 480	ER80 470 / 550	ER110 680 / 760	E71T-8 414 /496
Measured Yield /Tensile [MPa]	378 / 481	468 / 562	561 / 642	762 / 830	390/501
Yield/Tensile-Ratio	0.786	0.832	0.874	0.918	0.778
Elongation [%]	33	27	25	20	27
Yield Mismatch Factor	1.22 (HA 250)	1.51(HA 250)	1.82(HA 250)	2.47(HA 250)	1.26(HA 250)

Details are given in [4]. In this investigation, a range of overmatch ratios was covered often influenced by the practically achieved weld sizes.

In the butt joint it is difficult to closely match the strength of the weld with that of the parent metal, as the weld metal strength of the available consumables is always greater than that of the parent metal. In the fillet weld joint, size adjustments can be readily made, however, it has to be noted that a difference always exists between theoretical required and practically achieved fillet weld sizes.

Testing Procedure

The basic principle of the small and large-scale testing is to cyclically test in the inelastic range of the materials at a loading rate which is comparable with a severe earthquake. In order to obtain clearly distinguishable results the load was progressively increased until failure. It is noted that the resulting strain rate at the critical locations of the joint has not been measured.

Small-scale tests

Most of the small-scale tests were performed load controlled at a constant frequency of 60 cycles per minute. However, in order to obtain correlation to a change in frequency, identical samples were also tested at frequencies of 1, 2, 30, and 120 cycles per minute. Figure 4 shows the small-scale test setup, while figure 5 shows the applied load over time for the double-sided fillet weld specimen M1.5A.

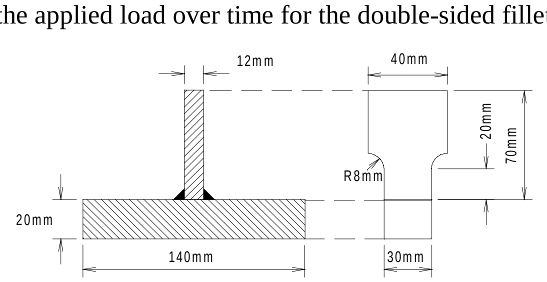

Figure 3: Small-scale test specimen geometry

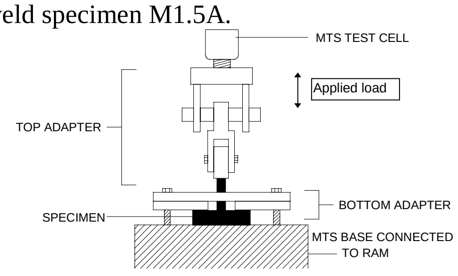

Figure 4: Small scale test set-up

Large-scale tests

The large-scale testing procedure was adapted from ATC-24 [8]. Figure 1 shows the test setup and figure 6 the deformation history as used in the testing program. It should be noted that, in order to obtain distinguishable results for relative similar samples and to generate failure conditions, the number of cycles per step and the extend of deformation has been increased compared to the recommended regime of ATC-24.

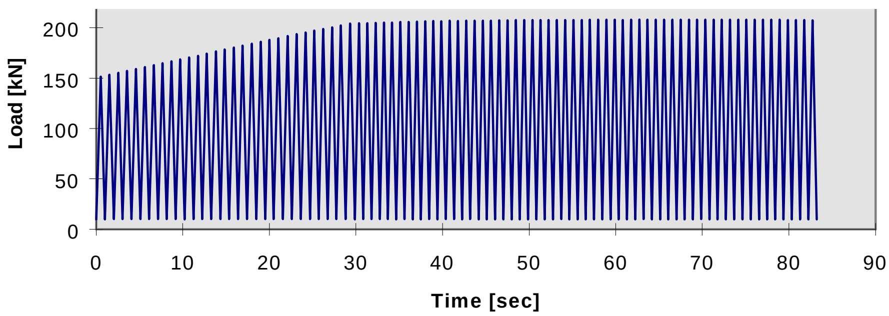

Figure 5: Load over time for the small-scale specimen M1.5A (60 cycles/min)

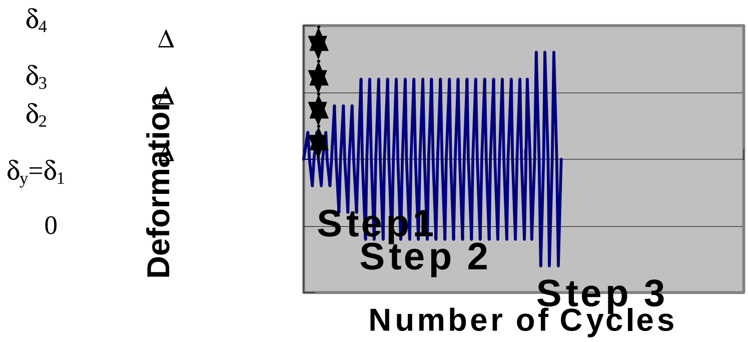

Figure 6: Deformation history for large-scale tests

The tests were performed under deformation control, with a deformation control parameter, Δ, equal to the deformation at yield, $\delta_{y,}$ of 18.8mm. As the loading rate could not be changed during the test without stopping, the loading rate was kept constant, accepting that the frequency will be reduced with an increase of deformation. The start up loading rate was chosen to reflect frequencies at step 3 (μ = 3) and 4 (μ = 4) which are typically expected at severe earthquakes. Table 3 shows the test parameters as applied.

Table 3: Test parameters for large-scale tests

Deformation step	peak displacement mm	total sum of displacements mm/cycle	loading rate mm/sec	Frequency cycles/sec [HZ]
1 (3 cycles)	Δ=18.8	75.2	150	≈ 2
2 (3 cycles)	2Δ=37.7	150.8	150	≈ 1
3 (20 cycles)	3Δ=56.6	226.4	150	≈ 0.7
4 (to failure)	4Δ=75.2	300.8	150	≈ 0.5

RESULTS

Small-Scale Tests

Figure 7 shows the applied load versus displacement for the specimen M1.5A. This is typical of the results obtained. A total of 77 tests were performed. Figure 8 shows the cycles to failure in relation to the different weld types, beam flange plate and welding consumable strength, break location and average effective mismatch factor.

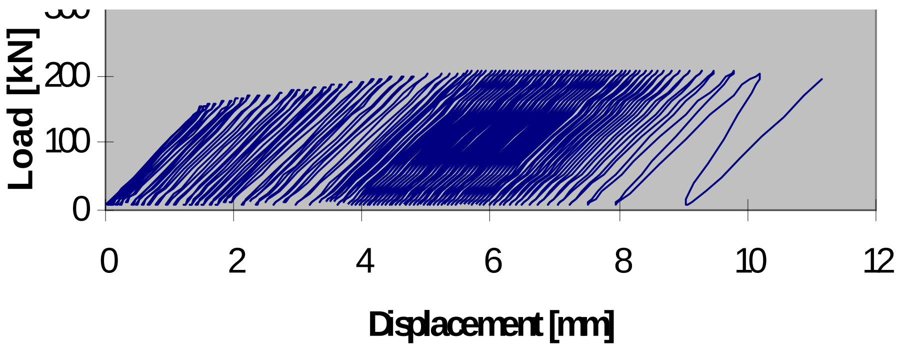

Figure 7: Load over displacement for small-scale specimen M1.5A

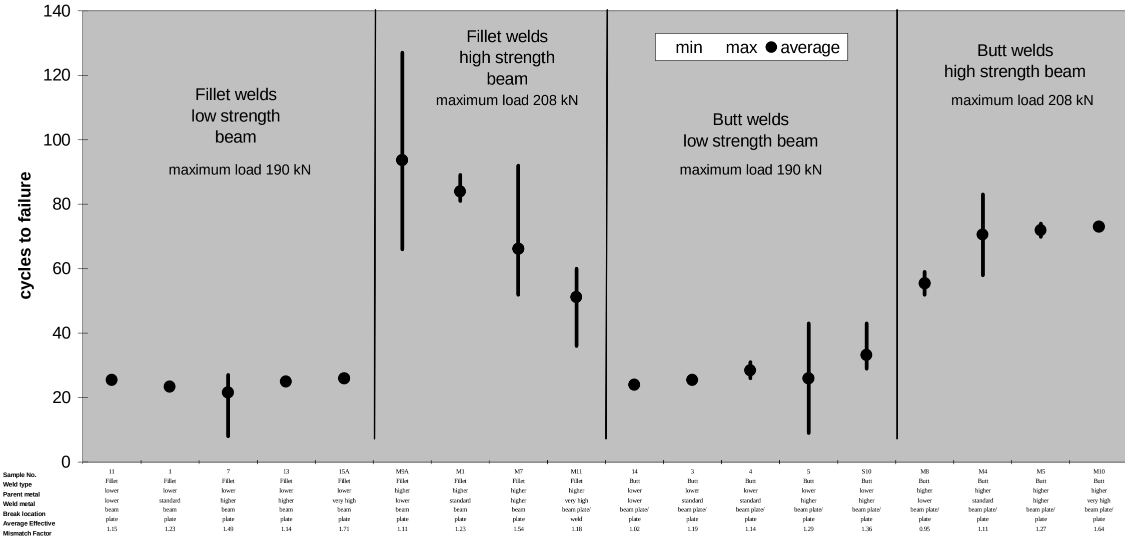

Figure 8: Results of small scale tests

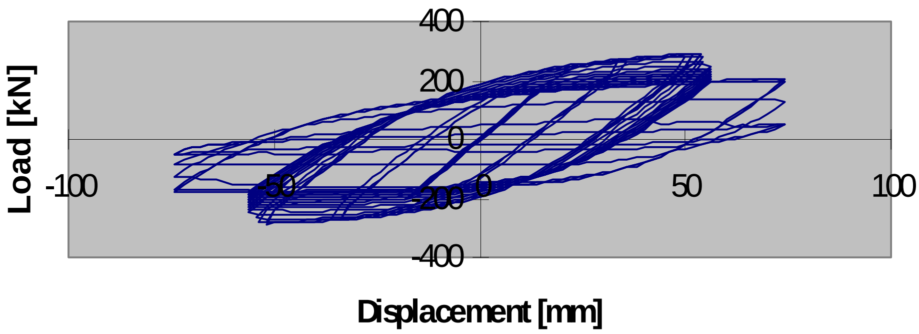

Figure 9: Tip load versus tip deflection MRC2 (double sided fillet, lower strength beam)

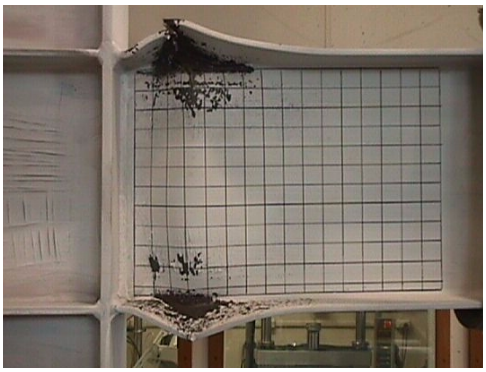

Figure 10: Sample MRC 2 after test, showing plastic hinge

Large-scale tests

To date 5 large-scale tests have been performed with the testing program to continue to further verify the relation ship between small and large-scale testing. Figure 9 shows the tip load versus tip deflection of sample MRC2. Figure 10 shows the sample after the test. Table 5 lists the test parameters and the results.

DISCUSSION OF RESULTS

Scope

It should be noted at the outset of these discussions, that the aim of the research was to better understand the effects on performance of changing joint-related details. Discussion of the results is therefore on a comparative basis between alternatives. The evaluation of overall seismic capacity is secondary and is described eg. in [4].

Small-Scale Test Series - Constant Frequency

These tests were conducted at the frequency of 60 cycles/minute. The results recorded in the tests have been correlated under different aspects. Figure 8 shows the cycles to failure of typical samples in relation to the different weld types, ‘beam’ flange plate and welding consumable strength. The number of cycles are represented as the minimum, maximum and average number of cycles for each type of welded sample.

Influence of overmatching

The samples failed typically in a ductile failure mode in the vertical plate representing the ‘beam’ flange of the connection. If samples were undermatched and showed an effective mismatch factor of below 1, break location was typically in the weld. The samples shown in Figure 8 generally were overmatched and failed in the beam with the welds obviously having sufficient strength.

Influence of parent metal strength

Independent of weld type fillet or butt, there is a strong correlation between increased performance in respect to load and number of cycles and strength of ‘beam’ plate. The higher the strength of the beam plate the better the performance. It is noted that, in each sample, the maximum load at failure due to work hardening effect and/or strain rate related effects was always above the load in the standard plate tensile test (app. 10 % for the lower strength plate and app. 14% for higher strength plate alternative).

Influence of weld metal strength

For fillet welds it appears that for the higher strength beam a clear correlation between weld metal strength and performance exists. The lower the weld metal strength the better the performance of the joint. This is note worthy as from the conventional design approach this would not have been expected. Once the fillet weld has reached the strength of the adjoining parent metal the parent metal should fail always outside the weld at the corresponding parent metal strength.

In the case of butt welds, it appears that an increase in weld metal strength leads to a slight improvement in the performance of the sample. This is also surprising, as all samples broke clearly outside the weld area and one would assume only the parent metal was tested in this case. Again it is noted that this performance trend is stronger for the higher strength beam.

Table 5: Parameters and results of large-scale tests

Sample	Weld Type	Weld Metal Strength [N/mm2]	Parent Metal Strength [N/mm2]	Loading Rate [mm/sec]	Failure Type, Location	Fmax [kN]	Cycles to Fail-ure	Throat measured on flange side	Effective weld area on flange side	Throat calculated based on measured strength values	Calculated weld area based on measured values[1]	NZS 3404 required throat no safety factors	NZS 3404 required throat with safety[2] factors	overmatch factor (effective/ calculated weld area)
MRC 1	Fillet	lower	standard	150	brittle fracture, weld	283	7	6	2022	6.1	2055.7	6.4	10	0.98
MRC 2	Fillet	lower	standard	150	ductile	290	31	10	3370	6.1	2055.7	6.4	10	1.64
MRC 3	Fillet	lower	higher	150	ductile	299	35	9	3033	6.5	2190.5	6.4	10	1.38
MRC 4	Butt	lower	standard	150	ductile	291	35	11	3707	6.1	2055.7	6.4	10	1.80
MRC 5	Unequal Fillet	higher	higher	150	brittle fracture, weld	301	12	10.5	3538.5	6.5	2190.5	6.1	9.6	1.62

Influence of weld type

It appears that fillet and butt welds perform similarly. Particularly, it can be said that the double sided fillet welds with the unfused plate face did not perform worse than the full penetration butt joints. When comparing these results to those expected from the samples under high cycle loading the results are distinctively different. Under low stress range, high cycle loading it would have been expected that the Tee-joint with the full penetration butt

[1] weld area required to just develop tension capacity
[2] overstrength factor 1.25 and strength reduction factor 0.8

joint would have performed worse than the double sided fillet (45° angle between weld and 'beam' plate) due the influence of the higher stress concentration on the weld toe (90° angle).

Some fillet weld samples (1.13, 1.14, 1.15, 7.1, 7.2, and 7.3) had the root drilled out to reduce the effective fillet weld size. These drilled samples, although reducing the effective weld area by 13% for the 1.xx tests and 6% for the 7.x tests, performed as good as the non-drilled sample sections of the same welded test specimens. This appears to indicate that once the minimum required strength of a fillet welded connection is reached an increase in weld size does not improve the performance. Due to the small sample number this observation should be verified by further testing.

Large-Scale Tests

Influence of beam strength

Only the double-sided fillet weld test MRC2 with the lower strength beam and MRC3 with the higher strength beam are directly comparable. Not only did the higher strength beam achieve an increased number of cycles, but the rate of deterioration was also slower from $\mu = 3$.

Influence of weld type and weld size

The double-sided fillet weld test sample MRC1, in which the welds were sized without the over-strength factor of 1.25 and the strength reduction factor, failed suddenly at $\mu = 3 \times 1$ in the fillet weld and the fillet weld/ beam flange interface. Local buckling of the flange just commenced prior to weld failure. MRC2, sized with the over-strength factor of 1.25 and the strength reduction factor of 0.8, failed through flange fracture at cycle $\mu = 4 \times 5$. MRC5 was an unbalanced fillet weld aimed to simulate ideal on-site welding conditions by performing most of the welding in the downhand (1F) position and only a single run on each of the two overhead (4F) fillets. The sample failed at $\mu = 3 \times 6$ through sudden, overload type fracture initiated in the small, single run, overhead weld connecting to the lower flange. Beam flange local buckling had just commenced.

MRC4, the only butt weld tested at the time of writing, using the lower strength beam failed in the flange at $\mu = 4 \times 9$ performing slightly (4 cycles) better than the comparable fillet sample MRC2.

Correlation to small-scale tests

Despite the limited number of large-scale tests performed to date, there appears to be a positive correlation between the characteristics observed in the small-scale and large-scale experiments. This was particularly true with regard to the improved performance of the higher strength beams and the fact that the fillet welds of sufficient strength performed as good as the butt weld. The influence of weld metal strength has not yet been compared.

CONCLUSIONS

The program described is at an early stage and therefore only the preliminary results are available. Despite this, some useful conclusions for the future development of seismic connection details can be drawn.

Suitability of Small-Scale Testing

Small-scale testing using test plates representing the 'beam' of identical thickness to the large-scale samples, however reducing the connection to a simple T-joint as used in this program, appears to be a useful tool to obtain fundamental knowledge of weld seismic behavior, to simplify testing and to reduce the cost of testing. Additional tests particularly to investigate further weld strength alternatives and load path effects are recommended for verification of this claim.

The small-scale tests appear to confirm previous findings [7] that higher testing frequencies, and hence higher strain rates improve the seismic performance of welded connections. Therefore it can be assumed that the pseudo-static tests usually performed (e.g. as reported in [4]) are conservative.

Beam Strength

Both small and large-scale tests indicate that the actual beam strength has a strong influence on the performance. Provided the weld stands up to the demand of beams with higher strength, it is likely that an improved performance can be achieved. This could indicate that increasing beam strength, while at the same time ensuring adequate weld metal strength, may lead to connections able to cope with a higher seismic demand.

Weld Metal Strength

For fillet welded joints increasing weld metal strength, however having the overall weld strength matching the parent metal strength, appears to reduce the performance of the joint. Therefore increasing beam strength and combine this with a suitable sized fillet weld with lower strength weld metal seems to be an interesting alternative to further investigate.

For full penetration butt joints, increasing weld metal strength appears to increase the performance of the connection. It appears that increasing the restraint of the weld rather than increasing its ductility leads to performance enhancement. Therefore to combine a beam strength and weld metal strength increase for the butt welds seems a worthwhile avenue for further investigation.

Weld Design and Size

The small and large-scale tests indicate that both double-sided fillet welds and complete penetration butt joints designed to [5] are able to adequately handle the demands placed on the joint through the beam capacity under severe seismic attack. Both weld types performed similarly. This is an important conclusion in respect to fabrication choices and costs of connections.

The study showed that the sizing of the fillet welds is critical and the current requirements of the N.Z design standard [5] are adequate if the double-sided fillet welds are of equal size on both sides. The flange fillet welds where sufficiently strong to satisfy the demands placed on them provided the welds were sized using the seismic over-strength factor of 1.25 and the strength reduction factor of 0.8 as required for weld category SP.

When sizing fillet welds it appears important to consider the load path. While the balanced double sided fillet welds (MRC3) in the large-scale test performed well, the non-balanced, double-sided fillet, which was equal in effective weld cross sectional area, did not perform as well. As only one sample has been tested so far, further work will be performed to verify this finding.

Butt joint sizing following the requirement of NZS 3404 [5] to develop the full strength of the weaker connected component is less critical to administer in fabrication. This, combined with the typically overmatched weld metal strength and the provision that the welds meet category SP quality, should be adequate to guarantee failure outside the weld area. The effect of slight thickness reduction at the weld toe as a result of undercutting combined with the likely reduction in fracture toughness in the heat affected zone, has been shown not to be critical for fracture initiation in any of the tests reported herein, similarly for the pseudo-static tests reported in [4].

Beam size limitations

Although the beam size in this study was limited to the Grade300 410 UB54 with a flange thickness of 12 mm, the results obtained were consistent with and complemented the results of the pseudo-static tests described in [4] on larger size specimen including a Grade300 610UB101 beam with a flange thickness of 15mm. However, future work should investigate the extended range of used beam sizes and grades.

OUTLOOK

The described research, funded through support from the ‘New Zealand Foundation for Research, Science and Technology’ and the Australasian structural steel industry alike, is continuing over at least the next five years. It will, in small steps, gradually cover the addressed areas and will result in improvement to the analytical predictive tools and the provisions of NZS3404 [5].

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