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Seismic test of "effective full penetration of T-butt welds" in welded moment resisting connections.

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ABSTRACT

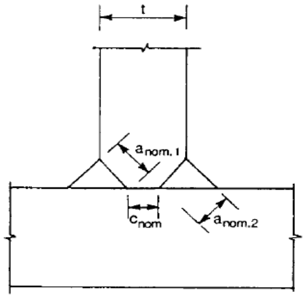
The fully welded moment resisting connection is a type of rigid steel connection that is used widely in moment frames in buildings, bridges and offshore structures. Welded moment connections have been qualified by many standards and guidelines such as NZS 3404, Eurocode 3 and AISC 341. Although the moment frames are costly, they are still popular in buildings among architects and building owners because they provide building systems with minimum obstruction to building functionality on any given storey. Full penetration but weld is commonly used to connect the beam flanges to the column flange. Butt welds are more expensive than fillet welds. To provide economical fabrication of steel structures, the New Zealand standard also allows fillet welds in moment resisting connections (MRCs). However, the cost of fillet welds escalates with the increase of the weld size. Partial penetration butt welds can be cheaper to fabricate than large fillet welds for thick flanges. EN 1993-1-8 includes a definition of an "effective full penetration of T-butt welds" implying that it can be used to replace butt welds leading to cost savings. This study evaluates the seismic performance of welded beam-to-column connections using effective full penetration of T-butt welds. Three full-scale T-shaped specimens were tested under the cyclic loading based on the SAC protocol. The results demonstrated that the effective full penetration butt welds work very well under seismic loading, forcing final failure into the beam and enabling considerable energy to be dissipated within the beam plastic hinge, with minor yielding in the connection panel zone and no failure in the beam to column welds.

Keywords: Effective full penetration T-butt welds, Seismic test, Moment resisting connections, Cyclic loading, Energy dissipation.

INTRODUCTION

Steel moment frames are used widely as a structural system for multi-storey buildings providing a long-span plan and high local and global ductility for structures in seismic areas [1, 2]. Fully welded moment resisting connections using in moment frames have been qualified by many standards and codes such as NZS 3404 [3], Eurocode 3 [4] and AISC 341 [5]. The web and both flanges of the beam are welded to the column using fillet or butt welds. Dubina and Stratan [2] reported that the details of welds or welding defects could be a possible cause for premature fracture in welded connections. Woerner, et al. [6] addressed the effects of weld geometry on the failure mode and performance of the fully welded connections. However, there are no reports of experimental testing to determine the performance of MRCs with "effective full penetration T-butt welds" in accordance with the EN 1993-1-8[4] (Figure 1) under simulated seismic loading conditions.

According to EN 1993-1-8 [4], effective full penetration of T-butt welds must meet two conditions. Firstly, the total throat thickness of welds on both sides should be equal to or larger than the thickness of the stem plate (t) on the T joint. Secondly, the un-welded gap between the roots of the welds should be the smaller of 3 mm or t/5. See Figure 1.The EN 1993-1-8 Standard [4] considers the strength of the effective full penetration T-butt weld providing these two conditions to be equal to that of a full penetration butt weld. However, many studies have shown that the porosity or notch formed due to discontinuities at the root of the welds make these welds susceptible to low cycle fatigue failure in seismic regions [7, 8, 9]. It should be noted that for marine structures, it has been well established that an effective full penetration T-butt weld can be achieved as long as fillet weld is sufficient sized to prevent weld throat cracking in static [10], cyclic loading [11,12] and low-cyclic fatigue conditions [13].



$$a_{nom,1} + a_{nom,2} \geq t$$

$$c_{nom} \text{ should be the smaller of } {}^{t}/_{5} \text{ and } 3mm$$

Figure 1. Effective full penetration of T-butt welds after EN 1993-1-8.

After revealing poor performance of welded moment resisting connections in US and Japan, a long-term program has been directed by Heavy Engineering Research Association (HERA) to evaluate the suitability of connection under the earthquake loads in New Zealand. The program has investigated both economic and technical aspects of MRCs to make them safe and cost-effective connections [14, 15]. Based on Woerner, et al. [6] the New Zealand steel design code NZS 3404 [3] is an exception that allows using fillet welds for the welded beam-to-column connection in seismic regions among the standards. The study reported fillet welds are economical with the maximum throat thickness of 11 mm, and the butt welds substitute for the bigger sizes. In the latter study, Karpenko, et al. [16] analysed the total price considering the weld material and fabrication cost for full penetration and partial penetration butt welds with different penetration ratio. The study concluded that the so-called "effective full penetration T-but welds" were most economical among the other types.

This study assessed the seismic performance of welded connections using effective full penetration T-butt welds by conducting large-scale tests in continuation of the HERA program to make the welded moment resisting connections more cost-effective in New Zealand without loss of their high reliability.

SPECIMENS AND TEST PROCEDURE

The testing was conducted in accordance to the SAC testing protocol [17]. Three T-shaped large-scale specimens representing the exterior beam-to-column moment frame connections were tested under cyclic loading (Figure 2). Pinned supports were embedded at the end of each side of the column (horizontal member), and the actuator applied the force at the tip of the beam. A Shore Western 913D Series hydraulic actuator with maximum ∓330 KN rated load, and stroke length of ∓ 150 mm was employed in the tests. Two lateral supports at different heights of the specimen were intended to prevent out-of-plane movement of the beam, thus ensuring the required in-plane response was achieved. The beam and column section sizes for each specimen are given in Table 1. All beams had similar plastic moment capacity but with very different depths, flanges and web thicknesses. The tensile coupon tests were conducted on the web and flange of all specimens. Table 2 displays the tensile strength results and the section properties for bending. The column sizes were chosen considering the overstrength factor according to the NZS 3404 Standard [3]. Consequently, the ratio of moment capacity of the column to beam should be more than 1.15 representing the overstrength factor for category 2 members (Table 2). The beams in specimens 2 and 3 were welded sections with fillet weld size 10 mm and effective full penetration butt weld size of 6.5 mm respectively.

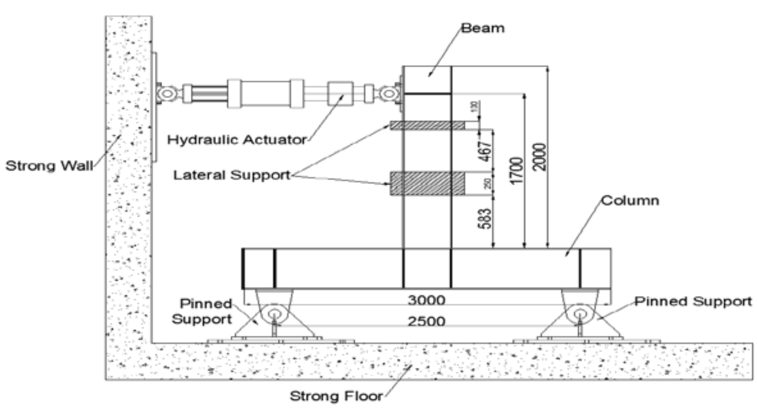


Figure 2. Test set-up based on the SAC protocol. All numbers in millimeter [mm].

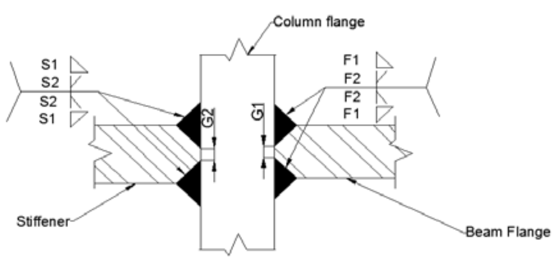
Table 1. Dimensions of beams and columns for experimental tests.

Steel Type	Specimen	Member	Designation	Depth of section (d) [mm]	Flange Width (b_f) [mm]	Flange Thickness (t_f) [mm]	Web Thickness (t_w) [mm]	Flange Slenderness Ratio
Carbon Steel (Grade 300 S0) AS/NZS 3679.1	1	Beam	410 UB 53.7	402.6	178	10.9	7.6	9.33
		Column	460 UB 74.6	457.4	190	14.5	9.1	7.24
	2	Beam	ST 20	400	105	20	8	2.78
		Column	460 UB 74.6	457.4	190	14.5	9.1	7.24
	3	Beam	ST 32	200	175	32	16	2.87
		Column	460 UB 74.6	457.4	190	14.5	9.1	7.24

Table 2. Tensile coupon test results and section properties.

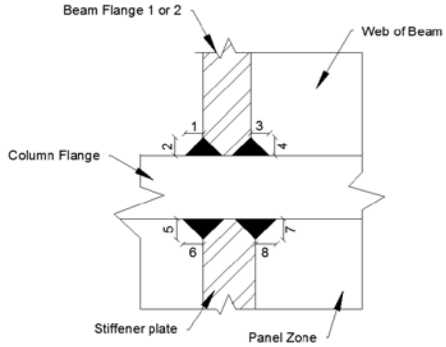
Specimen	Member	Designation	f_y $[N/mm^2]$	f_u $[N/mm^2]$	$Z_e/10^3$ $[mm^3]$	M_{sx} $[KNm]$	M_{sx} (Column) / M_{sx} (Beam)
1	Beam	410 UB 53.7	356	497	1035.49	368.64	1.50
	Column	460 UB 74.6	337	492	1637.71	551.91	
2	Beam	ST 20	328	491	1057.2	346.76	1.59
	Column	460 UB 74.6	337	492	1637.71	551.91	
3	Beam	ST 32	334	490	1014.78	338.94	1.63
	Column	460 UB 74.6	337	492	1637.71	551.91	

All the welded connections designed based on the NZS 3404 Standard [3]. The doubler plates were welded into the panel zone of all specimens. The thickness of the stiffener plates was identical to the thickness of beam flanges in all tests except test 1 with stiffener thickness of 12 mm. All welds were performed according to AS/NZS 1554.1 Standard [18] with weld category SP and the E49 electrodes. Figure 3 and associated table illustrates the drawing weld details and sizes designed for welding the beam flange and stiffener to the top column flange in all specimens. In addition, the weld leg sizes were measured after fabrications with the weld gauge, and Figure 4 demonstrates the measured weld sizes.



Specimens	Drawing weld size [mm]					
	F1	F2	S1	S2	G1	G2
1	4.4	4.4	4.8	4.8	2.1	2.3
2	8.5	8.5	8.5	8.5	2.9	2.9
3	14.5	14.5	14.5	14.5	2.9	2.9

Figure 3. Dimensions of effective full penetration of T-butt welds based on the drawings.



Specimens	Beam flange	Average measured weld sizes [mm]							
		1	**2**	**3**	**4**	**5**	**6**	**7**	**8**
1	Flange 1	8.06	9.25	8.37	10.5	8	8	10.5	10.75
	Flange 2	8.25	10	9.12	9.25	7.37	8.87	9.37	10.75
2	Flange 1	11.5	8.87	10.25	10.62	9.12	12.12	9.5	13.62
	Flange 2	11.12	9.37	11.62	11.87	9.62	11.75	9.62	12.87
3	Flange 1	19.37	17.12	19.62	18.87	19	19	16.62	18.62
	Flange 2	18.56	16.06	18.12	19	16.37	18.75	17.25	18.125

Figure 4. Measured leg size of welds.

The cyclic loading history was based on the SAC protocol [17], with quasi-static loading to enable thorough visual observation to be made during the testing. The loading rate increased as the peak displacement increased to keep the frequency of each set of cycles similar, once again to assist in getting high quality visual data. Approximately, the loading rate was intended to generate the same frequency for each load step. After load step 10 the loading history continued with the constant rotation angle of 0.06 because of limitations in the stroke of the actuator. Table 3 gives the loading history.

Table 3. Derivation of loading history for the tests.

Load Step #	Peak deformation ϴ [rad]	Peak Displacement [mm]	Loading rate [mm/sec]	Frequency [Hz]
1	0.00375	7.23	1	0.035
2	0.005	9.64	1	0.026
3	0.0075	14.47	2	0.035
4	0.01	19.29	2	0.026
5	0.015	28.93	4	0.035
6	0.02	38.57	4	0.026
7	0.03	57.86	8	0.035
8	0.04	77.15	8	0.026
9	0.05	96.44	10	0.026
10	0.06	115.72	16	0.035

RESULTS

Mode of failure and weld performance

Moment frames with a weak beam, strong column strength hierarchy enables considerable absorption of the incoming earthquake energy through plastic hinge formation. This is beneficial for overall building response. The weld's strength must be sufficient to allow the desired beam plastic hinging to occur. Otherwise, weld rupture before developing beam plastic hinge reduces energy absorption as well as potentially detrimentally affecting the overall structural performance [2]. Figures 5, 6 and 7 illustrate the failure of all specimens after applying the cyclic loads for tests 1, 2 and 3, respectively.

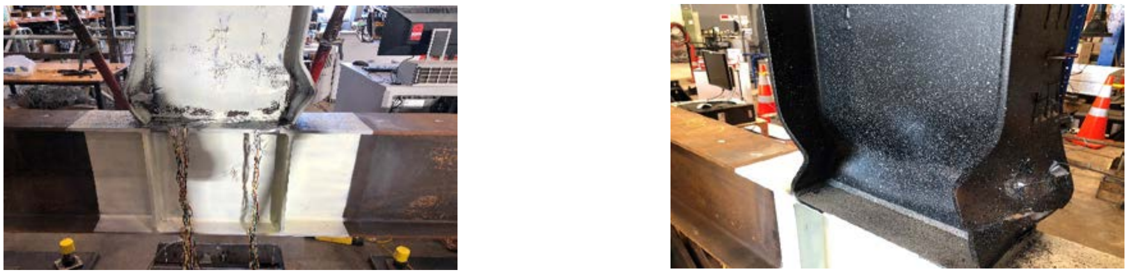


Figure 5. Specimen 1 after failure.

Figure 6. Specimen 2 after failure.

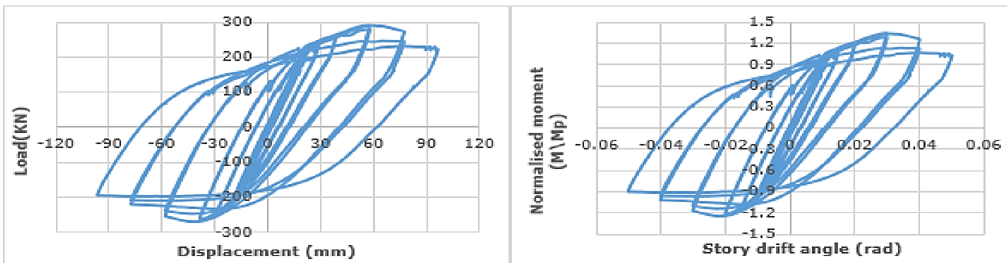
Figure 7. Specimen 3 after failure.

Plastic hinges were created in the specimen 1 under cyclic load test with clear local flexural buckling. The crack initiated at the plastic hinge zone on the beam flange when the basic load history reached to step 9 (Cycle 31). The lateral supports did not work ideally during the test, and there was slight lateral movement in the beam and column members. In specimens 2, as it is evident from Figure 6, both torsional and flexural buckling developed in the specimen. The fracture happened after applying 38 cycles of load. The beam flange crack initiated after significantly unplanned torsional movement in the beam and minor axis bending in the column due to the inefficient lateral restraining system. The extensive paint peeling indicated the large plastic deformation of beam flange and web. Contrary to the specimens 1 and 2, the lateral movement of the specimen 3 was fully prevented by the restraining system. A considerable amount of in-plane plasticity was observed on the beam flanges during the test. The crack initiated after a high number of cycles (94 Cycles) at the toe of the fillet weld and propagated toward the web of the beam. Some other minor cracks around the fillet weld toe were observed as well. Local buckling of the flange due to the plastic flexural demand was not observed in specimen 3, due to the very low slenderness ratio of the beam flange. The visual inspection results did not show any cracks or damages on the welds. The measured weld leg sizes (Figure 4) were approximately double that of the specified weld leg sizes for test 1, and 3~4 mm bigger than specified for test 2 and 3.

Strength and plastic rotation capacity

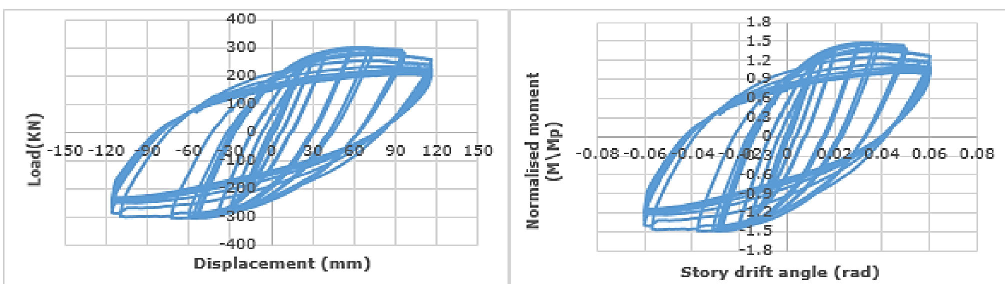
Since the section (plastic) moment capacity of all beams was similar, it was expected to obtain the same ultimate load in all connections. Figures 8-a, 9-a and 10-a show the load versus displacement curves. The results implicated approximately same ultimate strength of 300 KN for all specimens. The tests were stopped after 20% of load drop in the connection at which point the specimen was considered failed. The ultimate load occurred at load step 7 (Cycle 28) and load step 9 (Cycle 30) for tests 1 and 2, respectively. The graphs 8-a and 9-a and data analysis revealed that ultimate load did not correspond with maximum displacement in the cycles with high amplitudes in test 1 and 2. The lateral movement and local buckling of the specimens could explain this mismatching between ultimate load and maximum displacement during the test. The maximum load for test 3 happened at load step 10, and the load gradually dropped after increasing the number of cycles.

The maximum allowable plastic hinge rotation for the members with negligible axial force considering the earthquake loads or effects is 0.04 rad according to Table 4.7.1 of NZS 3404 [3]. Furthermore, based on the ANSI/AISC 341-10 Standard [5], the beam-to-column connections used in the seismic force resisting system shall reach to at least rotation angle of 0.04 rad with providing measured flexural resistance equal or greater to 80% of moment plastic capacity of beam section at this drift story angle. Figures 8-b, 9-b and 10-b illustrate the normalised moment verses story drift angle for each specimen. As it is obvious from the graphs, all connections provide sufficient story drift angle and flexural resistance according to NZ and US standards.



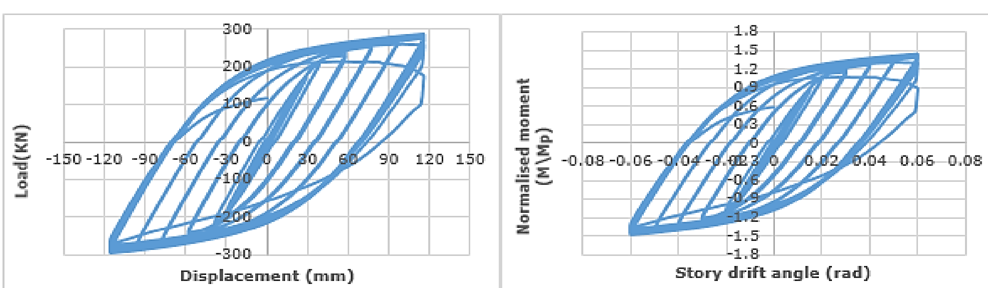
(a) (b)

Figure 8. Hysteresis behaviour of connection in specimen 1: (a) load-displacement curve, (b) Normalised moment versus story drift angle.



(a) (b)

Figure 9. Hysteresis behaviour of connection in specimen 2: (a) load-displacement curve, (b) Normalised moment versus story drift angle.



(a) (b)

Figure 10. Hysteresis behaviour of connection in specimen 3: (a) load-displacement curve, (b) Normalised moment versus story drift angle.

Stiffness and energy dissipation

The initial stiffness of the specimens was calculated by dividing the ultimate load to the maximum displacement at load step 1 and the results are presented in Table 4. The results showed the initial stiffness of specimens 1 and 2 were close, while the initial stiffness of specimen 3 decreased by 42% from specimens 1 and 2, due to the much more flexible beam cross section in test 3 compared with that in tests 1 and 2. (Beam depth of 200mm for specimen 3 compared with 400 mm for specimens 1 and 2).

Table 4. Initial stiffness calculation.

Test no.	Initial stiffness [KN/mm]
1	10.9
2	11
3	6.4

Moreover, the amount of energy dissipation of the connection was obtained by adding the area of each hysteresis loop. Figure 11 shows accumulated energy dissipation for each specimen. The amount of energy absorption in test 3 was considerable by taking account of the high number of cycles before failure.

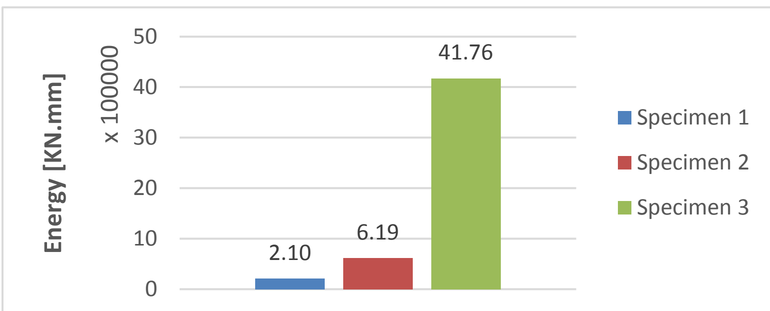


Figure 11. Accumulated energy dissipation.

CONCLUSIONS

This study assessed the seismic performance of full effective penetration of T-butt welds in welded moment connections. The summary of key results are as follows:

1. The visual inspection of the effective full penetration T-butt welds at the end of the testing did not show any damage or fracture in the weld metal under cyclic load proving the suitability of using this weld type in seismic regions.
2. The final failure mode of test 1 was local flexural buckling with substantial plastic deformation in plastic hinge zone. In specimen 3, although there was a great amount of in-plane plasticity on the beam flange, local flexural buckling formation was not observed in the specimen. Specimen 2 failed under flexural-torsional buckling with developed plastic hinge. It seems the crack formed due to the torsional movement of the section in specimen 2. The crack at specimen 3 initiated at the weld toe lead to low cycle fatigue failure of the specimen.
3. Test 3 showed a very high number of cycles to failure, however this is in large part due to the very low flange slenderness (lower than would be used in actual beams in practice) preventing flexural induced local buckling of the flanges.
4. The predicted overstrength factor based on NZS 3404 provides sufficient ductility for the connection.
5. All welded moment connections complied with the criteria of NZS 3404.1 and ANSI/AISC 341-10 Standards and were qualified for use in seismic areas.
6. Considerable energy dissipation developed, particularly for test 3 with the high number of cycles.
7. The study recommends suitability of effective full penetration T-butt welds in seismic areas. However, further testing is needed to develop recommendations for a welding procedure required to reliably achieve the weld penetration that is required. Corresponding quality and inspection requirements to ensure compliance of the proposed weld type in the fabrication workshop environment need to be developed.

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