PERFORMANCE OF RIGID WELDED BEAM TO COLUMN CONNECTIONS UNDER SEVERE SEISMIC CONDITIONS

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SUMMARY

This paper presents details of the current design model used for rigid welded beam to column connections in New Zealand moment-resisting steel framed (MRSF) seismic-resisting systems. This design model was revised following the January 1994 Northridge earthquake, in light of the damage to the standard US rigid welded MRSF connection in that severe event.

The efficacy of the model was tested in two series of tests; the first under pseudo-static rates of loading and the second at seismic-dynamic rates of loading. A brief overview of the scope, aim and key results from these tests is given. The paper concludes with references to sources of more detailed information on the design model and test series.

INTRODUCTION

Background

During the 1994 Northridge (Los Angeles) earthquake, many large-scale welded beam to column connections in moment-resisting steel frames performed poorly, undergoing connection failure prior to development of the expected inelastic action in the beams. The predominant failure mode involved sudden fracture of the beam flange to column flange connection, turning the as-designed rigid joints into actual semi-rigid joints.

Although the pre-1994 site-welded, shop-bolted detail used in Los Angeles is seldom used in New Zealand, the exercise of reasonable prudence led to the New Zealand rigid welded connection design model being reassessed. A revised design model for this type of joint was subsequently put through two sets of experimental tests; a large-scale, pseudo-static test series on seven specimens [Butterworth, 1997] and a large-scale, seismic-dynamic test series on five specimens [Scholz et.al., 1997]. (The latter tests were undertaken at earthquake rates of loading, the former tests at 10^{-2} to 10^{-3} times earthquake rates of loading).

These tests demonstrated the validity of the revised design model and the provisions incorporated into the 1997 revision of the Steel Structures Standard [NZS 3404, 1997].

Scope of Paper

First, this paper presents the general philosophy behind the seismic design of connections in New Zealand.

It then presents, in as much detail as space permits, the New Zealand design model for rigid welded connections. This is followed by brief details of the two experimental testing programmes used to verify important aspects of this design model. The paper concludes with comments on the validity of the design model and key results from the research programme. This is followed by the references, tables and figures.

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PHILOSOPHY BEHIND THE SEISMIC DESIGN OF CONNECTIONS

The general philosophy behind the seismic design of connections, as summarised from [Clifton, 1999], are as follows:

(1) Select a load path through the connection that is as simple and direct as practicable. In making this selection:

(1.1) Determine realistically the internal actions being generated in the members being connected, when these members are responding inelastically. For example, in an I-section subject to inelastic bending, the flange and at least half the web will develop in excess of their horizontal yield capacity.

(1.2) Determine actions on members based on the anticipated inelastic response of the overall structural system. This is especially applicable to determining column axial loads in seismic-resisting systems.

(1.3) Make provisions for transfer of internal forces from elements of the supported member into elements of the supporting member that lie parallel to the incoming force. For example, in a rigid beam to column strong axis connection, the incoming beam flange axial force must pass through the column flange and into the column web.

(1.4) Make provision for component forces introduced when an applied force changes direction.

(1.5) Design to suppress connector-only failure modes (ie. bolt or weld failure alone) by making the design capacity of connectors sufficiently greater than either the design capacity of connection components or the design capacity of combined connector/connection component modes of failure.

(1.6) Ensure that all load paths from a supported member element to a supporting member element are effectively equally stiff, unless the effect of differential movement within the connection is known to be not detrimental. This means, for example, that if the flanges of a beam are directly welded to a column, the beam web must also be directly welded to that column and not connected via a welded or bolted cleat. Similarly, all elements of a beam must be directly welded to the endplate in a moment-resisting endplate connection and the endplate is then bolted to the column.

(1.7) Welds transferring force between two elements that are perpendicular (eg. from a beam flange into a column flange) must allow for a balanced, symmetrical transfer of force. This means either double-sided fillet welds of equal leg length, double-sided incomplete penetration butt welds of equal size or complete penetration butt welds.

(2) Design all connection components and connectors to resist the design actions associated with the given load path, including overstrength actions where required (NZS 3404 provides appropriate guidance).

(3) Avoid, wherever possible, weld details where the skill of the welder determines the strength and performance of the weldment. Where this cannot be avoided, pay particular attention to meeting the specified weld quality for these welds.

(4) Avoid putting welds into direct bending.

(5) Detail the connection to retain its dependable load-carrying capacity under anticipated inelastic rotation of the connected members. Specific instances of this are given in NZS 3404.

(6) Ensure that weld consumables, bolts and steel are suitably notch-tough (NZS 3404 gives suitable guidance regarding material selection).

DESIGN MODEL FOR RIGID WELDED CONNECTIONS

Scope of Coverage

Design model details presented herein relate to the connection between the beam and the column in a conventional, rigid, shop-welded connection.

Design aspects relating to the connection interface between the beam member and the column face are presented in detail, while design of the column stiffeners and panel zone are covered in general philosophy terms only, with reference to the relevant provisions of NZS 3404.

Welds Between Beam Flanges and Column Flange

The location of these welds is shown in Figure 1. They are required to transmit the tension capacity of the flange into the column. The specific requirements, from Clause 12.9.1.2.2(1) of NZS 3404, are as follows:

(1) For complete penetration butt welds, the weld must be stronger than the beam flange, in accordance with AS 2205.2.1. Also, for welds connecting members that will be subject to inelastic demand under seismic action, $f_{uw} \geq f_u$ is required, where f_{uw} is the nominal tensile strength of the weld metal and f_u is the nominal tensile strength of the parent metal.

(2) For double-sided fillet welds, the required beam flange design action, S_f^*, is given by:

$$S_f^* = \phi_{oms} R_u = \phi_{oms} b_f t_f f_{yb} \tag{1}$$

where:

ϕ_{oms} = overstrength factor from Table 12.2.8(1) of NZS 3404 [5]

$R_u = b_f t_f f_{yb}$ = nominal yielding capacity of full flange section (b_f = flange width; t_f = flange thickness); f_{yb} = nominal yield stress)

The overstrength action in equation 1 reflects the fact that the beam flange to column flange weld is the primary load-carrying path from the beam to the column and that the nominal capacity of the flange will be exceeded, in practice, due to material variation and strain hardening. Both these factors are accounted for in the overstrength factor.

The fillet weld design capacity, ϕV_w, to resist S_f^*, is determined from Clause 9.7.3.10 of NZS 3404 by:

$$\phi V_w = \phi 0.6 f_{uw} t_t L_w \tag{2}$$

where:

ϕ = strength reduction factor = 0.8 for fillet weld category SP, which is the weld category required by [NZS 3404] for seismic applications

t_t = design throat thickness = $t_w / \sqrt{2}$ for an equal leg length fillet weld

t_w = leg length of fillet weld

L_w = length of weld = $2_{bf} - t_{wb}$

b_f = width of the beam flange

t_{wb} = thickness of the beam web

Equation 2 is based on the principle that the nominal capacity of the weld is given by shear failure (at $0.6 f_{uw}$) across a failure plane of the weld defined by ($t_t L_w$).

Welds Between Beam Web and Column Flange

The location of this weld is shown in Figure 1. A beam web undergoing yielding due to inelastic moment or axial tension will develop longitudinal action equal to the tension yield capacity of the web, at least over the region of the web adjacent to the beam tension flange. For dependable joint performance, the beam web to column flange weld must be able to transfer this action directly through to the column flange, via a load path of comparable stiffness to that through the beam flange into the column flange.

This weld is typically a double-sided fillet weld and the above requirements lead to a weld design action, s_w^*, given by equation 3 from [Feeney and Clifton, 1995].

$$s_w^* = 0.5 \phi t_{wb} f_{yw} \tag{3}$$

where:

s_w^* = design action/unit length on each weld of the double-sided fillet weld

t_{wb}	=	thickness of the beam web
f_{yw}	=	yield stress of the beam web
ϕ	=	0.9 (ϕ for the beam web, as given by [NZS 3404] for a butt weld of SP category)

For beams in inelastic bending under seismic action, sizing the weld on this basis over the full depth of the web will provide adequate capacity to resist the associated vertical shear force [Feeney and Clifton, 1995].

The fillet weld design capacity/unit length is again determined, from Clause 9.7.3.10 of [NZS 3404], by:

$$\phi v_w = \phi 0.6 f_{uw} t_t \tag{4}$$

Transfer of Internal Actions from the Beam into the Column

This requirement is made to suppress net section fracture at the interface between the beam flange weld and the column flange (along the line AA in Figure 1). The need for this comes from a review of the USA joint failures in the Northridge, 1994 earthquake and the application of this to New Zealand connections [Clifton et.al, 1998]. The requirement is expressed as equation (5):

$$\frac{A_{fct}}{A_{fb}} \geq \frac{\phi_{oms} f_{yb}}{0.9 f_{uc} \phi_t \eta} \tag{5}$$

where:

A_{fct}	=	area, in elevation, of the column flange over which the incoming beam tension flange force (N^*_{fbt} in Figure 1) is transmitted into the column flange.
A_{fb}	=	area of the beam (tension) flange
f_{yb}	=	nominal yield stress of the beam flange
f_{uc}	=	nominal tensile strength of the column (flange)
ϕ_t	=	through thickness to longitudinal tensile strength factor = 1.0 for steels of Australasian origin and 0.85 for hot-rolled wide flange jumbo column sections >40mm thick
η	=	column flange force reduction factor, which = 1.0 except for high values of (axial load & moment), as given by [Clifton et.al., 1998; Feeney & Clifton, 1995].

Details regarding the use of equations 5 and 6 are given in section 8.1.2 of [Feeney & Clifton, 1995], including the use of fillet weld reinforcing of a complete penetration butt weld, if equation 5 is slightly undermet.

Use of double-sided fillet welds between the beam flanges and the column flange, rather than a complete penetration butt weld, is advantageous in meeting equation 5, as A_{fct} is more than doubled.

Column Stiffeners and Panel Zone

The design requirements for these are presented in Clause 12.9.5 of NZS 3404.

The tension and compression stiffeners are designed to transmit the internal axial forces from overstrength beam flange action (tension/compression), into the column, via the flanges and web. These stiffeners are sized for the overstrength action, so that inelastic action in them is minimised.

The panel zone design is based on preventing significant inelastic action developing in this region until the incoming beam(s) have commenced to yield, then allowing panel zone yielding to develop as an additional means of absorbing the overstrength beam actions.

EXPERIMENTAL TESTING PROGRAMMES

Aim and Scope of Test Programmes

As previously stated, the aims of the large-scale, pseudo-static test programme were to determine the efficacy of the design model presented above (especially equation 5) under pseudo-static rates of loading and to determine differences between the performance of flange butt welded and flange fillet welded connections. Table 1 gives details of the seven specimens, Figure 2 shows the test rig set-up and Figure 3 shows one specimen ready for testing.

The seismic-dynamic test programme investigated the same items, but under earthquake rates of loading. Because of equipment constraints, the test member sizes were smaller; each of the five specimens tested used a 410UB54 Grade 300 beam and a 460UB67 Grade 300 column. Design of the panel zone, tension and compression stiffeners was in accordance with NZS 3404 Clauses 12.9.5.2 and 12.9.5.3. Details of the weld sizes, strengths are given in Table 5 of [Clifton et.al., 1998] and are presented in companion paper by Scholz & Clifton.

(1) Specimen MRC1 used a double-sided fillet weld between the beam flanges and column flange, sized below the NZS 3404 requirements through using ϕ_{oms} = 1.0 in equation 1 herein, ϕ = 1.0 in equation 2 and sizing the weld leg length to within 1 mm of that required. The double-sided fillet weld between the beam web and column flange was sized, to the nearest 1 mm, using ϕ = 1.0 in equation 3 and ϕ = 1.0 in equation 4.

(2) Specimens MRC2 and MRC3 used double-sided fillet welds between all beam and column elements, sized in full compliance with NZS 3404 (equations 1-4 herein).

(3) Specimen MRC4 used a complete penetration butt weld between the beam flanges and the column flange and double-sided fillet welds between the beam web and the column flange; all complying with NZS 3404.

(4) Specimen MRC5 used double-sided fillet welds between all beam and column elements, with the total required weld strength sized in accordance with equations 1-4 herein (but with these provisions applying to both welds). However, the beam flanges to column flange welds were unevenly sized, in order to minimise the amount of overhead welding that would be needed if this joint were to be site welded. The result was a 6mm leg length fillet weld on the underside of each flange and an 18mm leg length fillet weld on the top side of each flange. Within the context of the current design model [Feeney & Clifton, 1995] and provisions of NZS 3404, this specimen had adequate weld strength, but did not meet the general principle from [Clifton, 1999], that the load path should be as balanced as possible.

Loading Regimes

(1.1) The loading regime for the large-scale, pseudo-static tests is described in [Butterworth, 1997] and can be summarised as follows:

(i) 2 force controlled cycles to 60% of the lateral load calculated to develop the nominal beam section moment capacity, M_s. The corresponding deflection, $\Delta_{0.6Ms}$, was measured and multiplied by (1/0.6) to give the displacement corresponding to μ = 1.0. All subsequent cycles were displacement controlled.

(ii) 2 cycles to ductility μ = 2

(iii) 2 cycles to ductility μ = 4

(iv) 2 cycles to ductility μ = 6

(v) 2 cycles to ductility μ = 8, etc.

(1.2) The applied loading rate was typically 10^{-2} to 10^{-3} of that associated with earthquake loading.

(2.1) The loading regime for the large-scale, seismic-dynamic tests is described in [Clifton et.al., 1998; Scholz et.al., 1999] and can be summarised as follows:

(i) 2 force controlled cycles to 60% of the lateral load calculated to develop M_s , in order to obtain $\Delta_{0.6Ms}$ and hence ductility Δ_{μ} = 1.0. All subsequent cycles were displacement controlled.

(ii) 3 cycles to μ = 1

(iii) 3 cycles to μ = 2

(iv) up to 20 cycles to μ = 3; if achievable, in which case

(v) cycles to μ = 4 until failure.

(2.2) Each cycle of loading from μ > 1.0 was applied at a constant loading rate of 150mm/sec, so as to give 1 cycle/second for steps (iii) above.

(2.3) The loading was representative of that associated with inelastic response of a conection in a medium-rise, moment-resisting frame.

Key Results

Space limitations herein permit only presentation of the key results from the two test series, these being:

(1) In all cases where the welds were designed in accordance with the NZS 3404 strength requirements, failure occurred in the beam yielding region. Some results from specimen D2, large-scale, pseudo-static tests, are shown herein in Figures 4, 5 and 6. In all these cases, plastic rotation levels in the compact beam sections were comfortably in excess of those typically required in practice. Both fillet welds and butt welds performed similarly and satisfactorily.

(2) Beam flanges welded to columns with either undersized fillet welds (ie. specimen MRC1) or unbalanced fillet welds (ie. specimen MRC5) suffered flange weld failure prior to development of the desired beam plastic rotation.

(3) Panel zone shear strains of up to 1.6% were measured, however visible signs of yielding showed up only when the panel zone was painted to highlight commencement of yielding.

(4) The presence of an axial tension force in the column had no observable influence on the failure modes or observed behaviour of the joint.

(5) There was a strength increase of around 1.25 due to the dynamic rate of loading in the seismic-dynamic tests, however this strength increase did not change the strength hierarchy within the test specimen or the observed modes of failure and joint behaviour.

CONCLUSIONS

These are as follows:

(1) The design model presented in section 8.1.2 of [Feeney and Clifton, 1995] and the design provisions of NZS 3404 Clause 12.9.5 are adequate to ensure dependable seismic behaviour in a rigid, welded MRSF with shop welded beam to column connections, at least within the member sizes tested. These sizes are representative of medium-rise to high-rise construction as used in New Zealand.

(2) The presence of imperfections that were borderline/slightly in excess of those specified by AS/NZS 1554.1 [AS/NZS 1554.1, 1995] (the relevant welding standard) for weld category SP did not adversely affect the performance.

(3) The all fillet welded specimens performed as well as the specimens with complete penetration butt welds between the beam flanges and the column flange, provided that the beam flange to column flange fillet welds are balanced, two-sided welds designed for beam flange overstrength action to NZS 3404.

REFERENCES

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Butterworth, J.W. (1997), *Cyclic Testing of Large Scale, Fully Welded and Semi-Rigid Steel Beam Column Joints,* Department of Civil & Resource Engineering, University of Auckland, School of Engineering Report No. 577.

Clifton, G.C. et.al. (1998), "Performance of rigid welded beam to column joints under inelastic cyclic loading," Conference Proceedings, *Welded Construction in Seismic Areas*, American Welding Society, Miami, USA, pp. 161-174.

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Table 1. Member and weld details for large-scale, pseudo-static tests

Speciman Designation	Beam Size/Flange Thickness	Beam Grade	Column Size	Column Grade	Column Axial Load	Beam Flanges to Column Flange Welds(1,4)	Beam Web to Column Flange Welds(4)
A1 and A2	610UB 101 14.8mm	300	350WC 258	300	$0.2N_t$	CPBW +5mm FW reinforcement	10mm FW double-sided
B1 and B2	610 UB 101 14.8mm	300	350 WC 258	300	0	CPBW	10mm FW double-sided
C1 and C2	610UB 101 14.8mm	300	475PB 147	350	$0.2N_t$	CPBW(2)	10mm FW double-sided
D2(5)	610UB 101 14.8mm	300	350WC 258	300	0	18mm FW double-sided (see note (3))	10mm FW double-sided

Notes to Table 1:

1. CPBW ≡ complete penetration butt weld; FW ≡ fillet weld
2. No repair was made to 1.5mm undercut around the flange tips of C1.
3. The 18mm leg length fillet weld was run around the root radius on the inside face of the beam flange.
4. Welds used the FCAW process, E7/T-G weld metal, f_{uw} = 496 MPa.
5. Specimen D1 was a semi-rigid joint, details of which are given in [Butterworth, 1997].

Table 2 Key performance parameters from the large-scale, pseudo-static tests

Speciman Designation	$M_{sx,beam}$ (kNm)	$M_{rx,column}$ (kNm)	$M_{pz,column}$ (kNm)	$M_{max,beam}$ (kNm)	$\frac{M_{max,beam}}{M_{sx,beam}}$	Maximum Beam Plastic Rotation (milliradians)	$\frac{M_{max,beam}}{M_{pz,\ column}}$	Maximum Panel Zone Shear Strain (percent)
Relevant Notes	1	1	1, 2	3		3		3
A1	920	1350	1565	1420	1.54	55	0.91	1.0
A2				1340	1.46	75	0.86	0.75
B1	920	1430	1565	1301	1.41	60	0.83	0.60
B2				1375	1.49	90	0.88	0.90
C1	920	1140	1325	1315	1.43	55	0.99	1.65
C2				1328	1.44	90	1.00	1.45
D2	920	1205	1565	1360	1.48	60	0.87	0.80

Notes to Table 2:

1. Calculated from measured mean mechanical properties and cross section dimensions.
2. Panel zone shear capacity converted to equivalent moment at beam end.
3. As measured, ignoring sign (sense of quantity).

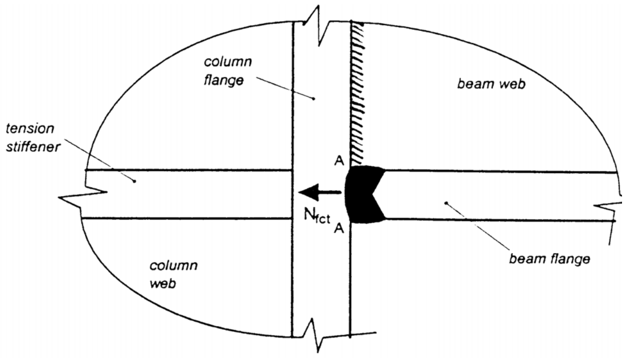

Figure 1 Beam flange to column flange detail (from [3]).
Note: The butt weld detail shown is for a shop-welded connection.

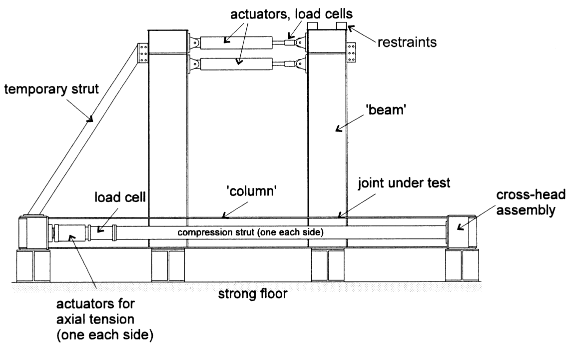

Figure 2 Test rig setup for large-scale, pseudo-static tests (from[4])

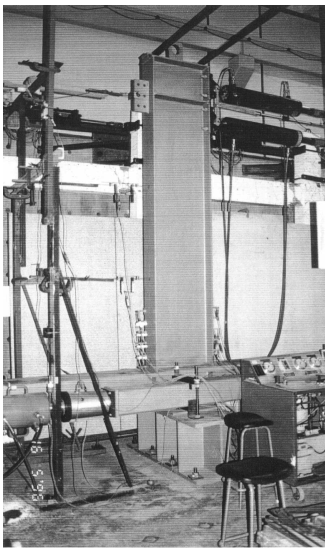

Figure 3 Large-scale, pseudo-static specimen in test rig, prior to loading

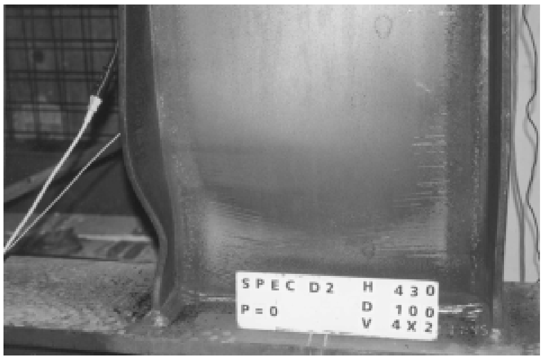

Figure 4 Specimen D2 at cycle 2 to displacement ductility 4 (μ = 4)

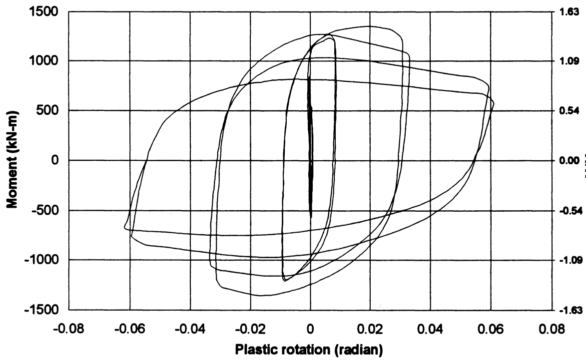

Figure 5 Specimen D2; moment versus plastic rotation

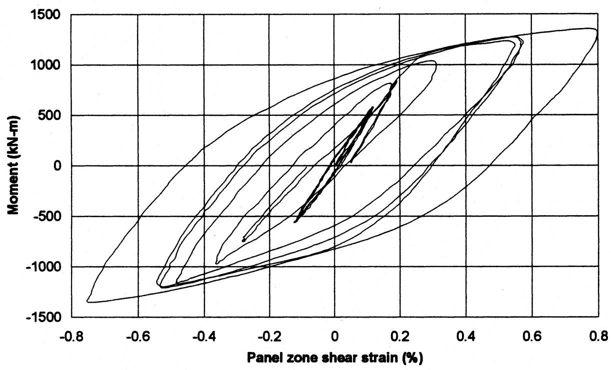

Figure 6 Specimen D2; beam moment versus mean panel zone shear strain