
A Review of Weld Detailing and Recommendations for Seismic Moment-Resisting Frame Connections

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ABSTRACT

Moment-resisting frame (MRF) connections play a vital role in steel structures for transferring gravity and seismic loads, ensuring continuity between beams and columns and enhancing the overall stability and ductility of the frame. In New Zealand, both welded moment-resisting connections and moment endplate (MEP) connections are commonly adopted for buildings located in all regions of seismic demand. These connections require careful weld detailing to ensure their strength, ductility, and reliable performance under cyclic loading. Therefore, a wide range of experimental and numerical investigations has been conducted on such connections with various weld types, providing valuable insights into their structural behaviour, failure modes, and energy-dissipation capacity. This paper presents a comprehensive review of experimental and numerical studies on moment-resisting connections incorporating various weld types, including complete penetration butt weld (FPBW) and equivalent complete penetration butt welds (EFPBW) such as incomplete penetration butt weld (PPBW), and fillet weld (FW). The review examines the structural performance, weld detailing, and predominant failure mechanisms associated with each weld type. In addition, design recommendations and weld detailing practices are evaluated with reference to the Australian/New Zealand Standard (AS/NZS), the American Institute of Steel Construction (AISC) specifications, and the Federal Emergency Management Agency (FEMA) provisions for seismic moment-resisting connections. Finally, the study identifies the most suitable and cost-effective weld types for seismic MRF connections and provides recommendations to designers and fabricators in New Zealand's steel construction industry.

1 INTRODUCTION

In New Zealand, the Canterbury earthquakes (2010–2012) and the Kaikoura earthquake (2016) caused widespread damage across central and southern regions, subjecting buildings to moderate to severe ground

shaking and resulting in significant economic and social consequences (Clifton and MacRae, 2018). Under such extreme seismic demands, structures are vulnerable to progressive failure, potentially leading to partial or total collapse. Structural connections, particularly beam–column joints, are especially critical, as their failure can disrupt load transfer mechanisms and compromise overall structural stability (Wilkinson et al., 2005).

Steel structures are widely recognised for their favourable seismic performance due to their inherent ductility, relatively low weight, and versatility in structural configuration (Ribeiro et al., 2022). Recent developments, including replaceable fuse elements, self-centring systems, and the integration of damping and base isolation devices, have further enhanced structural resilience and reduced potential economic losses following major earthquakes (Macedo and Castro, 2019).

In seismic design, the primary steel lateral load–resisting systems include Moment-Resisting Frames (MRFs), concentrically and eccentrically braced frames, and shear wall systems. Rigid moment-resisting frame systems, where beam–column connections are designed to develop and sustain significant bending moments, are commonly classified as Ordinary Moment Frames (OMFs), Intermediate Moment Frames (IMFs), Special Moment Frames (SMFs), and Special Truss Moment Frames (STMFs), each providing progressively higher levels of ductility and energy dissipation capacity (Paul et al., 2000).

Steel MRFs are widely used in multi-storey buildings because they allow long spans and provide both local and global ductility in seismic regions. In these systems, beam–column connections are critical components, as they must transfer axial forces, shear forces, and bending moments between members while maintaining stiffness and strength under cyclic loading. MRF connections may be fabricated using welded, bolted, or riveted details. In New Zealand practice, welded moment (WM) connections and moment endplate (MEP) connections (see Figure 1) are commonly adopted in both multi-storey and single-storey steel buildings designed for seismic resistance.

This paper reviews recent research on WM and MEP connections and discusses relevant weld detailing provisions for seismic applications.

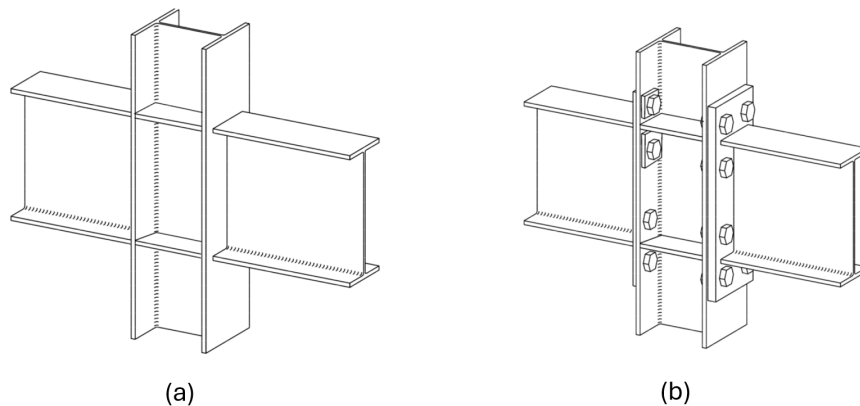


Figure 1: Schematic diagram of (a) welded moment connection (b) Moment endplate connection

2 RESEARCH STUDIES OF WELDED MOMENT AND MOMENT ENDPLATE CONNECTIONS

2.1 General

Simple welded moment connections incorporating web bolting and flange welding were widely used worldwide prior to the Northridge earthquake; Figure 2a illustrates a typical pre-Northridge welded moment-resisting connection. Post-earthquake inspections following the 1994 Northridge earthquake in the USA

revealed that welded flange–bolted web connections exhibited poor seismic performance, primarily due to premature connection failures. In many cases, fracture occurred at the weld between the beam bottom flange and the column flange (see Figure 2b). The primary cause of this failure mode was very high stress concentrations at the beam flange to column flange connection, exacerbated by weld defects at the root of the weld (associated with downward welding). Dubina et al., 2002 identified several contributing factors to this poor performance, including: (a) inadequate consideration of load-transfer mechanisms within the connection; (b) welding defects related to workmanship; (c) detailing that led to stress concentrations at weld roots and toes; (d) the use of low-toughness weld materials; and (e) unusually high seismic demands associated with high strain rates, although the latter was the least significant. Furthermore, most pre-Northridge connections could only develop beam plastic rotation capacities of less than 0.01 rad, indicating very limited ductility.

The Northridge earthquakes led to extensive research on improving steel moment frame design, resulting in deformation-based performance criteria for beam–column connections (FEMA 355, 2000). These were incorporated into the AISC Seismic Provisions, which require special moment frame connections to sustain inter-storey drift angles of ± 0.04 rad without significant strength loss.

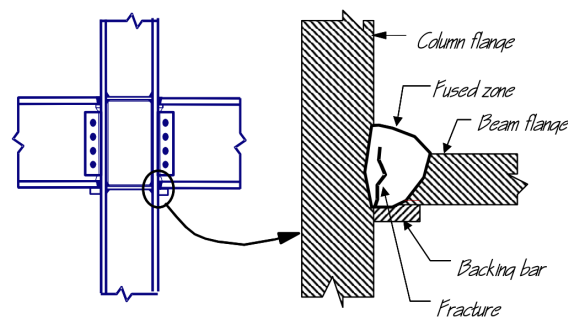


Figure 2: Pre-Northridge (a) typical welded connection (right) and (b) stress concentration location (left)

2.2 Welded moment (WM) connection

After the Northridge earthquake, welded moment (WM) connections were categorised into two types: (a) welded unreinforced flange–welded web (WUF-W) (see Figure 3a) and (b) welded unreinforced flange–bolted web (WUF-B) connections (see Figure 3b). These connections are prequalified for seismic applications and are widely adopted in practice.

For the WUF-W connection, Ricles et al., 2000 and Lee et al., 2005a,b carried out experimental investigations to assess its seismic performance. The prequalification of the WUF-W moment connection is primarily based on the outcomes of these two major research programs.

Mao et al., 2001 conducted three-dimensional finite element analyses (3D-FEA) to investigate the inelastic behaviour of WUF-W connections with various beam web–to–column flange attachment details. The study examined the influence of weld access hole geometry on connection fracture and identified the slope of the flat transition region as a critical parameter. A reduction in this slope decreased the plastic strain demand at the transition toe, thereby delaying the initiation of low-cycle fatigue cracks. Based on these findings, a modified weld access hole geometry with a 13° slope was proposed.

Ricles et al., 2002 experimentally evaluated WUF-W moment connections incorporating the revised weld access hole geometry (see Figure 4a) developed by Mao et al., 2001. All specimens were fabricated using high-toughness weld metal and exhibited stable cyclic behaviour, achieving inelastic rotation demands of not less than 0.03 rad prior to failure. Similarly, Lee et al., 2005a,b conducted experimental studies on WUF-W connections to investigate the effects of doubler plates, continuity plates, and panel zone flexibility, adopting a weld access hole slope of 15° (see Figure 4b). The results demonstrated that all specimens satisfied the rotational capacity requirements for SMF connections.

Han et al., 2014 examined WUF-W connections with a 21° access hole slope (see Figure 4c), which is close to the maximum limit specified by AWS D1.8. More recently, Skiadopoulos et al., 2022,2023 investigated WUF-W connections incorporating customised bevelled backing bars that were intentionally retained after completion of the beam flange-to-column face complete penetration butt welds (FPBW), using both numerical simulations and experimental testing.

In terms of welded unreinforced flange-bolted web (WUF-B), Early tests on WUF-B connections (Popov and Stephen, 1972) showed most specimens failed at or near welds. Later studies (Popov et al., 1985; Engelhardt and Hussain, 1993) reported similar abrupt failures with little inelastic deformation. Post-Northridge investigations (Popov et al., 1998) confirmed these connections fractured at very low drift angles.

Furthermore, post-Northridge studies (Stojadinović et al., 2000) showed that using notch-tough electrodes and improved details, such as removing the bottom flange backing bar, increased WUF-B connection ductility. However, SAC-post-Northridge did not meet the AISC Seismic Provisions of 0.03 rad total plastic rotation requirement for SMFs. The studies revealed a shift in failure mode from brittle and ductile overload weld fractures in pre-Northridge connections to ductile base-metal tearing near the access hole, followed by fast fracture. Similar fractures were observed in WUF-B connections with improved access hole geometry Han et al., 2007, indicating that this failure mechanism is largely insensitive to access hole design. These findings highlight that moment connections for SMFs should incorporate both weld-fracture mitigation and flange-overstress mitigation measures. Morrison et al., 2016 conducted a comprehensive numerical investigation and experimental testing of a modified WUF-B connection, incorporating a redesigned bolted web and a recently validated approach to promote plastic hinging away from the connection. The proposed connection demonstrated compliance with the SMF inter-story drift requirement of 4% without significant reduction in strength. Notably, no weld or near-weld fractures were observed, indicating enhanced ductility and improved seismic performance relative to conventional WUF-B connections.

Woerner et al., 2006 investigated WM connections using fillet welds (FWs), examining how weld geometry influences failure modes and performance. In their study, both the beam flange and web were fully welded to the column face with fillet welds. The results indicated that fillet welds designed using the stress component method provided sufficient deformation capacity for notch-tough weld metal within the tested specimen sizes. All connections achieved the required design ductility, exhibiting significant inelastic behaviour through extensive yielding before fracture. More recently, Taheri et al., 2018,2020 conducted six full-scale tests on WM connections using equivalent complete penetration butt welds (EFPBWs) such as incomplete penetration butt weld (PPBW), and FW. Both FW and PPBW types on MRF connections satisfied the 4% inter-story drift requirement, indicating their suitability for seismic applications.

2.3 Moment endplate (MEP) connection

During the 1994 Northridge earthquake in California, simple WM prequalified connections suffered beam flange to column flange fracture in the many steel structures. This led to extensive experimental and analytical studies on seismic moment connection performance, including the SAC Steel Project. Among the alternatives investigated, MEP connections were identified as a reliable seismic-resistant detail.

In North America, the design of MEP connection is governed by AISC 358 (AISC, 2010). Based on the research of Murray and Sumner, 2000,2002,2003 three bolted joint configurations are prequalified: four-bolt extended unstiffened, four-bolt extended stiffened, and eight-bolt extended stiffened. A stepwise design procedure grounded in yield line theory (Murray and Sumner, 1990) is provided for these connections. Figure 5 illustrate yield line mechanism of MEP connection.

In terms of unstiffened MEP connections (see Figure 3c), many studies have been investigated the seismic performance. Ghobarah et al., 1990a,b conducted two experimental series on MEP connections under cyclic

loading and concluded that properly designed MEP connections were suitable for moment-resisting frames in high-seismic regions. Bernuzzi et al., 1996 investigated flush and extended end-plate types and found that both performed satisfactorily in stiffness, strength, and rotational ductility. Morrison et al., 2017,2019,2022 proposed a new unstiffened MEP connection as an alternative to the eight-bolt stiffened MEP connection, featuring a non-symmetric octagonal bolt layout to optimize stress distribution.

For stiffened MEP connection (see Figure 3d), several studies, investigated the effect of stiffened MEP connections, showing that stiffener improve stress distribution, bearing capacity, and initial stiffness. However, they can also induce stress concentrations on the beam flange, negatively affecting connection behaviour. Eatherton and Murray, 2024 found that in built-up 24-bolt joints, stiffeners significantly reduced ductility due to flange fractures, whereas hot-rolled profiles achieved rotations up to 5% before failure. Recently, Tartaglia et al., 2018,2025 numerically studied the combined effect of this octagonal arrangement and stiffener on the MEP connection, highlighting its beneficial performance, and found that stiffeners on hot-rolled profile assemblies had negligible on joint ductility.

It is important to note that, in most cases of both unstiffened and stiffened MEP connections, FPBW welds were used to connect the beam flange to the endplate. Notably, some studies have also investigated the use of fillet welds (FW) and partial penetration butt welds (PPBW) welds for the beam flange in MEP connections. Tsai and Popov, 1990 investigated the cyclic behaviour of MEP connections by testing three specimens with FWs, without access holes or backing bars. It was found that the no beam weld failures were observed in any of the specimens. Moreover, Chen and Wang, 2009 investigated unstiffened MEP connections with FWs and PPJ welds. A total of 24 specimens were tested under monotonic loading and 6 under cyclic loading. The results revealed no failures due to weld cracking, and the cyclic behaviour remained unaffected, with local buckling being the predominant failure mode in all specimens.

To assess the effect of access holes on MEP connections, Adey et al., 2000 conducted test on both stiffened and unstiffened MEP connections, including specimens with and without access hole configuration. Observed failures included end-plate rupture, beam yielding, and lamellar tearing of the endplate, while no weld failures were recorded. The influence of the welding process, with or without coping holes, was found to be minimal.

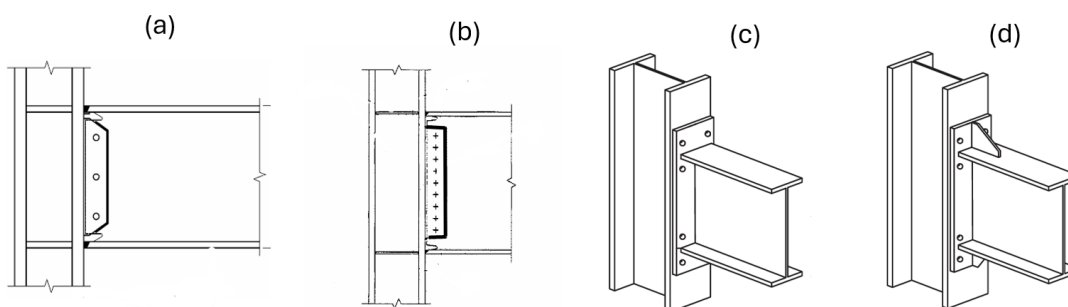


Figure 3: Schematic diagram of (a) WUF-W (b) WUF-B (c) unstiffened MEP and (d) stiffened MEP connection

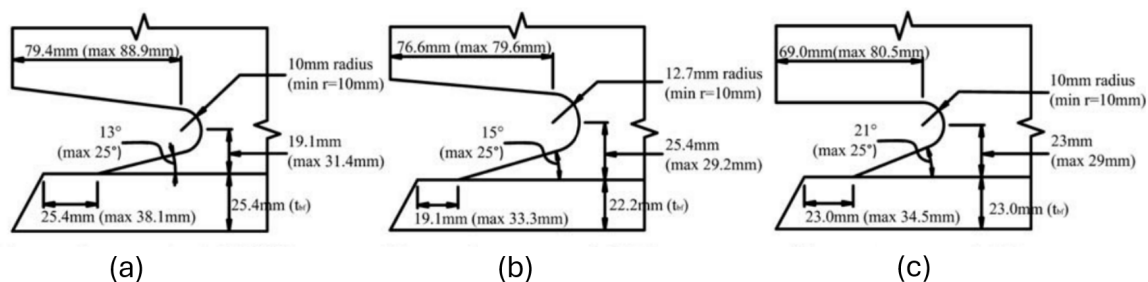


Figure 4: Different weld access hole shape

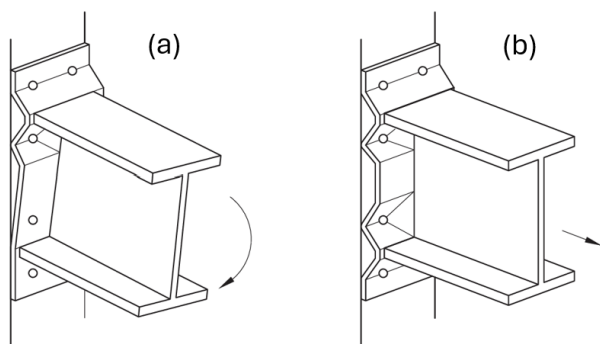


Figure 5: Yield line mechanism on MEP connection due to (a) moment and (b) axial tension force

3 CONNECTION DETAILS AND DESIGN PRACTICE

3.1 American recommendations

3.1.1 Welded moment (WM) connection

In welded moment (WM) connections, the FEMA guidelines recommend both WUF-W and WUF-B connections as prequalified for seismic applications. In which, The WUF-W connection is prequalified for use in OMFs, while the WUF-B connection is prequalified for use in both OMFs and SMFs. Moreover, the AISC (AISC 358, 2010) and FEMA guidelines (FEMA 350, 2000) provide detailed welding requirements, including ancillary weld details such as access hole geometry and backing plates, which are summarised below.

- *Beam flange-to-column weld details for connection*
 - FPBW groove welds shall be used for beam flange-to-column face connections in both WUF-W and WUF-B systems
- *Beam web-to-column weld details for connection*
 - A shear tab is used to connect the beam web to the column flange in both WUF-W and WUF-B systems. The shear tab is welded to the column face using full depth PPW to develop the full shear strength of the plate.
 - For the WUF-W connection, the beam web is connected to the shear tab using FW.
 - For the WUF-B connection, the beam web is connected to the shear tab using high-strength bolts, while the shear tab itself is welded to the column flange.
- *Weld access hole details*
 - Weld access hole geometry and quality requirements shall conform to the seismic design provisions of the American Welding Society Structural Welding Code – Seismic Supplement (AWS D1.8, 2016), illustrated in Figure 6a.
- *Backing plate details*
 - Top beam flange-to-column interface: Where steel backing is used for FPBW groove welds and is left in place, it shall be continuously welded to the column with fillet weld (min 8 mm) along the edge beneath the FPBW groove weld.
 - Bottom beam flange-to-column interface: Where steel backing is used for FPBW groove welds, the backing shall be removed. After removal, the root shall be back gouged to sound weld metal and back welded with a reinforcing fillet weld (min 8 mm).

- It should be noted that backing at beam flange-to-column flange joints shall not be welded to the underside of the beam flange, nor shall tack welds be permitted at this location.

3.1.2 Moment endplate (MEP) connection

According to the FEMA guidelines, both unstiffened MEP and stiffened MEP connections are prequalified for seismic applications and are suitable for use in both OMFs and SMFs. For these connections, AISC design guide 39 (AISC, 2023) and FEMA guidelines (FEMA 350, 2000) provides detailed guidelines weld details, ancillary weld details such as access hole geometry and backing plates, which are summarised below.

- *Beam flange-to-endplate weld details for connection*
 - FPBW shall be used for beam web-to-endplate connections for both the unstiffened and stiffened MEP connections.
- *Beam web-to-endplate weld details for connection*
 - FW shall be used for beam web-to-endplate connections for both the unstiffened and stiffened MEP connections.
- Guest plate weld details for connection
 - FPBW double-bevel groove weld shall be used for Guest plate to endplate or beam flange face connections for the stiffened MEP connections.
- *Weld access hole*
 - For high-seismic-ductility design, the FPBW weld of beam flange is made without using weld access hole and the backing plates are not used for both the unstiffened and stiffened MEP connections.
 - weld access holes are allowed to use in gravity, wind and low seismic -ductility application for stiffened MEP connection. Weld access hole geometry meets requirements of seismic design provisions of the American Welding Society standard (AWS D1.8, 2016).

3.2 New Zealand and Australian recommendations

In New Zealand, seismic structural systems are classified into four categories: (a) Category 1—fully ductile systems; (b) Category 2—limited ductility systems; (c) Category 3—nominally ductile systems; and (d) Category 4—elastic systems. Category 1 and Category 2 refers to high damage design or the conventional steel moment resisting frames. whereas Category 3 refers to low-damage designs. Most modern design seismic systems are Category 2 systems, as they are based on an estimate of the demand and capacity. However, Australian standards (AS 4100, 2020) are classified into three categories based on the structural ductility factors (μ), like the AISC such as (a) Special moment-resisting frames (fully ductile) (b) intermediate moment-resisting frames (moderate ductile) (c) ordinary moment-resisting frames (limited ductile) with structural ductility factors (μ) values of 4, 3 and 2 respectively. It is important to note that AS 4100 (AS 4100, 2020), recommended to use steel member and connections for fully ductile structures shall be designed in accordance with New Zealand steel structures standard (NZS 3404, 1997).

In New Zealand, seismic connections developed by Steel Construction New Zealand (SCNZ) (SCNZ 14.1, 2010), in accordance with NZS 3404 (NZS 3404, 1997) and AS/NZS 1554, recommend the use of welded moment (WM) and moment endplate (MEP) connections for seismic applications. Similarly, the Australian Steel Institute (ASI) (ASI, 2009a,b) has developed connection recommendations in accordance with AS 4100 (AS 4100, 2020) and AS/NZS 1554 (AS/NZS 1554, 2014) for these connections.

3.2.1 Welded moment (WM) connection

Unlike American standard, simple WM connections are recommended to use in New Zealand and Australia (see Figure 1a). SCNZ seismic connection guidelines (SCNZ 14.1, 2010) and the ASI connection design guide 11 (ASI, 2009a), provide detailed design procedures and welding details (see Figure 7a) for WM connection in accordance with AS/NZS 1554 (AS/NZS 1554, 2014), which are summarised below.

- *Beam flange-to-column weld details for connection*
 - FPBW or PPBW or FW shall be used for beam flange-to-column face connections.
- *Beam web-to-column weld details for connection*
 - FPBW or PPBW or FW shall be used for beam web-to-column face connections.

In New Zealand, these WM connections are shop welded, designed to develop the overstrength from the beams and used in conjunction with a moment resisting beam splice along the beams away from the column face.

In terms of ancillary weld details, there are no specific guidelines for access holes or backing plates in either New Zealand or Australian standards for seismic application. However, welding details and fabrication details must comply with AS/NZS 1554 (AS/NZS 1554, 2014) and AS/NZS 5131 (AS/NZS 5131, 2016), respectively. It is important to note that weld access holes are required only for FPBW welds and not for FW or PPBW welds. AS/NZS 5131 (AS/NZS 5131, 2016) provides general limitations for weld access hole geometry, including:

- The length of the weld access hole measured from the toe of the weld preparation shall be not less than 1.5 times the web thickness.
- The height of the weld access hole shall be not less than 1.5 times the web thickness and not less than 25 mm nor greater than 50 mm.
- The radius of the access hole shall be not less than 10 mm.

Regarding backing plates under seismic loading, AS/NZS 5131 (AS/NZS 5131, 2016) recommends two scenarios: (a) removable weld backing and (b) permanent steel backing. In the case of removable weld backing, a reinforcing FW with a minimum leg size of 6 mm should be applied to the column flange. For permanent steel backing, the backing bar shall be welded to the column using a continuous FW. It is important to note that welding between the backing plate and the beam flange is prohibited, as illustrated in Figure 6b.

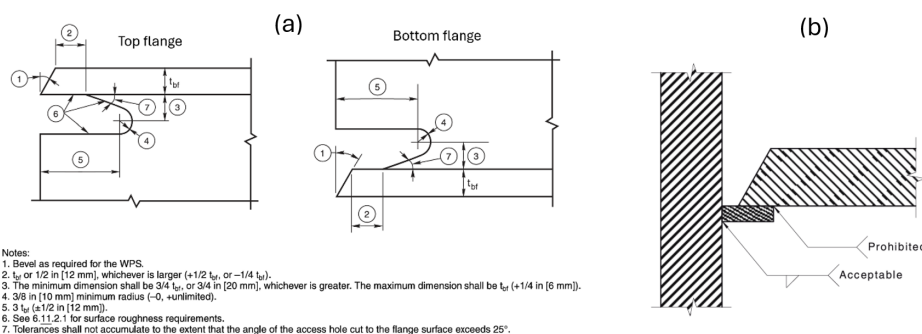


Figure 6: Ancillary weld details (a) access hole (AWS D1.8) (b) connection detail

3.2.2 Moment endplate (MEP) connection

Like the American design recommendations, both unstiffened and stiffened MEP connections are recommended for use in seismic applications. SCNZ seismic connection guidelines and the ASI connection design guide 12 (ASI, 2009b), provide detailed design procedures and welding requirements for MEP

connections in accordance with AS/NZS 1554 (AS/NZS 1554, 2014). A summary connection details (see Figure 7b,c) of these provisions is provided below.

- Beam flange-to-column weld details for connection
 - FW or FPBW or PPBW welds shall be used for beam flange-to-column face connections for both the unstiffened and stiffened MEP connections.
- Beam web-to-endplate weld details for connection
 - FW or FPBW or PPBW welds shall be used for beam web-to-column face connections for both the unstiffened and stiffened MEP connections.
- Guest plate weld details for connection
 - FW or FPBW double-bevel groove weld shall be used for Guest plate to endplate or beam flange face connections for the stiffened MEP connections.

It is important to note that weld access holes and backing plates are not required for moment endplate (MEP) connections. However, the SCNZ seismic connection guidelines (SCNZ 14.1, 2010) recommend providing a 25 mm hole radius on the gusset plate at the junction of the beam flange and the endplate to facilitate proper weld access and ensure full weld continuity between the beam flange and the endplate.

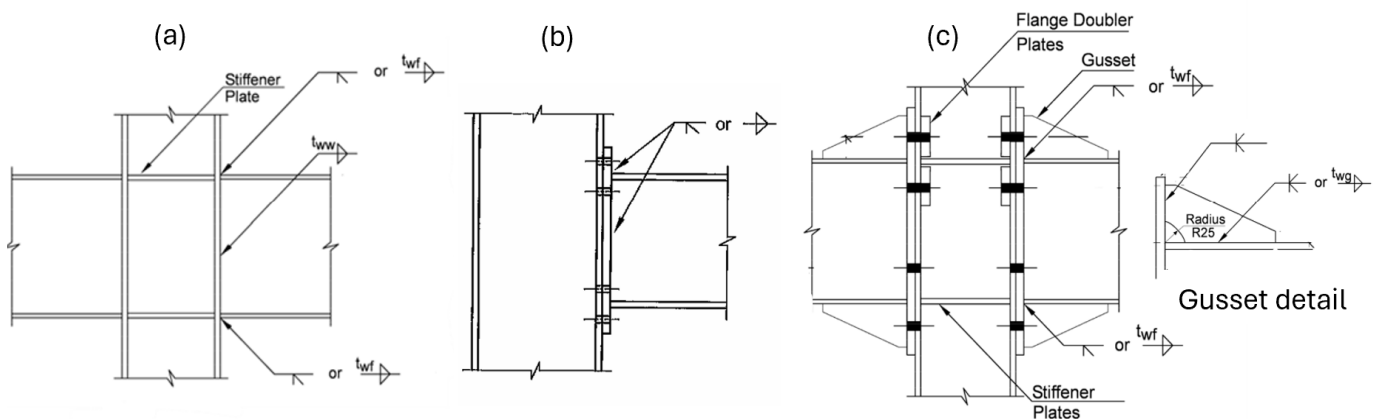


Figure 7: Weld details of (a) WM (b) unstiffened MEP (c) stiffened MEP connections

4 ECONOMICAL WELD CONSIDERATIONS

In both welded moment (WM) and moment endplate (MEP) connections, American design standards generally require FPBW welds between the beam flange and the column flange, and between the beam flange and the endplate, respectively. In contrast, the New Zealand standard NZS 3404 and Australian practice provide an exception among major international codes by permitting FW in welded beam-to-column connections in seismic regions. Karpenko et al., 2013 evaluated total welding costs, including material, preparation, and setup, for FW, PPBW, and FPBW welds with varying penetration ratios. The results showed that complete penetration FPBW welds were approximately 3.5–5 times more expensive than PPJ welds, while PPJ welds were the most economical, followed by FW. Consistent with these findings, ASI design Guides 11 and 12 (ASI, 2009a,b) recommend using FW instead of FPBW welds where feasible for cost efficiency, and SCNZ similarly advises FW for beam flanges for these connections. Moreover, the FW and PPBW welds are generally more cost-effective, depending on the welding and preparation techniques (Chandramohan et al., 2022; Taheri et al., 2023).

5 CONCLUSION AND FUTURE RESEARCH

This paper presents a review of research on welded moment-resisting (WM) and moment endplate (MEP) connections, focusing on their structural performance and weld detailing in seismic applications. The existing experimental studies reported that both WM and MEP pre-qualified connections compliance with the SMF inter-story drift requirement for seismic applications.

- Unlike American standards, Australian and New Zealand standards permit the use of fillet welds (FW) or incomplete penetration butt welds (PPBW), in place of complete penetration butt welds (FPBW), for WM and MEP connections at the beam flange-to-column flange and beam flange-to-endplate interfaces, respectively.
- The use of FW and PPBW welds eliminates the need for weld access holes and backing bars; Comprehensive testing of large MRF connections performed using standard New Zealand's fabrication practices confirmed adequate performance of connections with FWs, PPBW and BWs without coping holes (Tsai et al., 1990; Adey et al., 1999; Woerner et al., 2006; Chen and Wang, 2009; Taheri et al., 2018 and 2020).
- In terms of economical consideration, Australian and New Zealand standards recommend the use of FWs wherever possible in both WM and MEP connections. Furthermore, studies by Karpenko et al., 2013 and Taheri et al., 2023 reported that FW and PPBW welds are more cost-effective options than FPBW welds.
- It is recommended that New Zealand fabricators adopt these weld types for both WM and MEP connections as a cost-effective solution following the upcoming publication of revised standard NZS 3404, which will include requirements for EFPBWs.
- Although comprehensive literature exists on WM and MEP connections, studies on FW and PPBW welds for these connections are limited; therefore, further research is underway to simulate the behaviour of MRFs without coping holes and to validate this experimentally.
- With respect to ancillary weld details, such as weld access holes and backing plates, Australian and New Zealand standards provide only general guidance compared with American standards, indicating the need for clearer and more comprehensive provisions.

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