

Eccentrically Braced Frames with Removable Links – Design Methodology

Author: Kevin Cowie^a, Alistair Fussell^a, Martin Wong^a, Charles Clifton^b, Dmitry Volynkir^b

Affiliation: a. Steel Construction New Zealand Inc., b. The University of Auckland

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Eccentrically Braced Frame, EBF, Removable Links

Introduction

Steel eccentrically braced frames are expected to sustain damage during a design level earthquake through repeated inelastic deformation of the active link. Repair is therefore expected to be costly and disruptive, even if the structure has met its goal of providing life safety during an earthquake. The replaceable or removable active link concept addresses these drawbacks as it allows for quick inspection and replacement of damaged links following a major earthquake, significantly minimising the time to reoccupy the building.

The development and research of eccentrically braced frames with removable links is covered in Steel Advisor article *EQK1005* (in preparation at the time of writing this article). This article presents the design methodology for eccentrically braced frames with removable links. It focuses on the design and detailing of the link, and where necessary, modifications to the design procedure for conventional EBF's. Readers not familiar with the global seismic behaviour and design of eccentrically braced frames should consult HERA report R4-76. Steel Advisor article *EQK1007* provides a worked design example to illustrate the application of the proposed eccentrically braced frame with removable link design procedure.

Anatomy of a Removable Link

The configuration of an eccentrically braced frame with removable links is shown in figure 1. The clear active length is ideally selected such that the EBF behaves in a shear mode, as this makes design of the bolted connection more feasible due to the lower moment developed within the active link. Replaceable links would typically be fabricated from hot rolled universal column (UC) or custom welded I sections. Collector beams have a depth greater than the removable link section. The removable link is connected with an extended end plate bolted connection to a flush end plate on the collector beams. This requires the collector beam to be a deeper section than the active link. Braces may consist of universal columns (UC) or SHS sections birdsmouthed over a gusset plate connection (figure 1).

The active link end plate connection is commonly connected to the collector beam end plate with an eight bolt arrangement using high strength property class 8.8 bolts. Where the links are long and the capacity derived active link flange tension forces are large a 16 bolt arrangement may be required. Alternatively, consideration should be given to using property class 10.9 bolts.

A flush collector beam end plate is used to allow for the placement of the floor deck on top of the collector beam.

The gap between the top of the active link and the underside of the slab should be at least 50mm and sufficient for the following:

- To allow a timber packer to be placed between the top of active link and underside of deck to support the decking during concrete placement when the decking ribs run parallel to the EBF frame
- To allow a flat jack to be inserted to push the link out following a severe earthquake if replacement is needed

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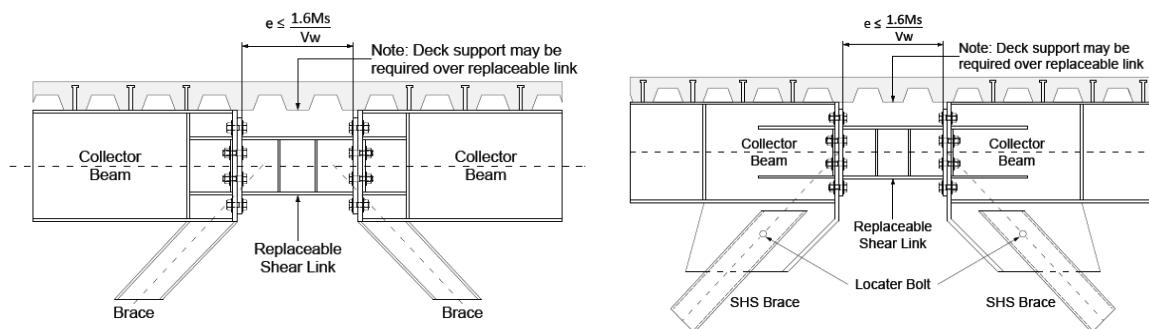


Figure 1: Replaceable Link Configuration, 1a: UC brace, 1b: SHS brace

Design Actions on a Removable Link

The typical force distribution in active links and collector beams of eccentrically braced frames under lateral load is shown in figure 2. The lateral force is assumed in figure 2 to be applied to both sides of the frame, and thus no axial load is transferred through the active link. There will be axial load present in the active link where diaphragm forces cannot be transferred to the collector beams on each side of the active link. Where this is the case then the active link must be designed for this axial load in accordance with NZS 3404 Clause 12.11.3.5. The active link beam is chosen such that the shear capacity is not significantly greater than the shear demand. Design actions are then factored up using the overstrength shear capacity of the active link. The designer should determine the number of different active link sizes to be used in accordance with the R4-76 recommendations and then average the design seismic actions across the levels where the active links sizes are the same and size the active link as closely as possible to this average.

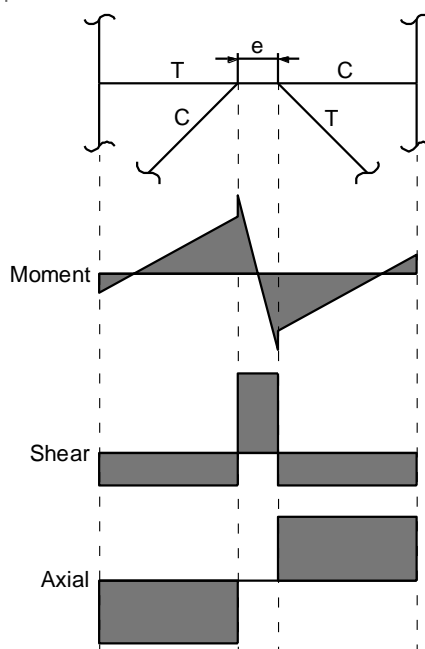


Figure 2: Design Actions in Collector Beams and Active Links under Lateral Load

Removable link end plate connections are subject to shear, moment and possible axial forces. Where there is no axial forces the link end connection is designed for the following capacity design derived actions:

1. Overstrength shear demand

$$V_{link}^c = \phi_{oms} V_w$$

where

ϕ_{oms}

Overstrength factor from NZS3404

V_w

Replaceable link nominal shear capacity

2. Capacity derived moment demand

$$M_{link}^c = V_{link}^c \frac{e}{2}$$

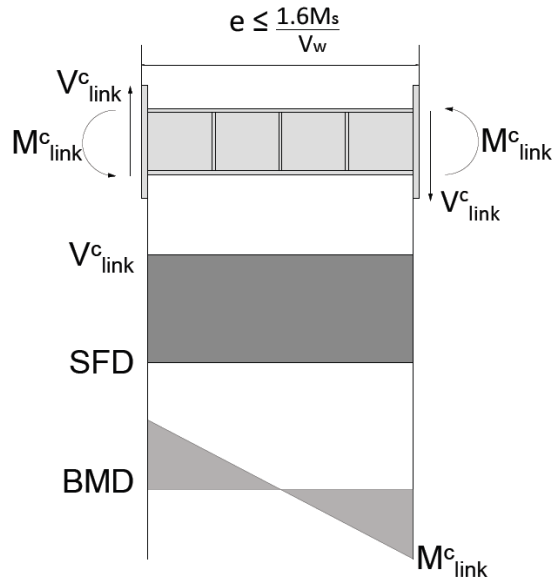


Figure 3: Link End Connection Forces

Geometric considerations

To minimise endplate capacity derived moment design actions the link length must be kept short. For these short link lengths the allowable interstorey drifts may be limited by the plastic rotation limits. The maximum rotation angle between the collector beam and active link is given by the following equation and is illustrated in figure 4.

$$\gamma_p = \frac{L}{e} \theta_p$$

where

θ_p

rotation angle of frame inelastic drift interstorey drift for an EBF

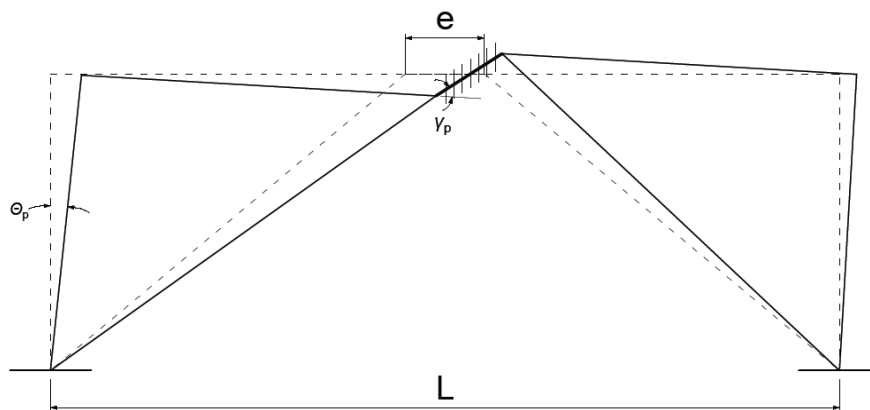


Figure 4: Plastic Rotation of Link

Note that the plastic rotation of the active link is what is checked against the plastic rotation limits of NZS 3404 Clause 12.11.3.3. To avoid overestimating this, the elastic drift of the EBF must be accurately determined. This can be obtained from equations 3.7 to 3.11 from (Sullivan, 2012).

A second geometric consideration is that the link beam depth must be sufficiently smaller than the collector beam depth to allow an extended end plate on the removable link. As a guide the collector beam should be 240mm deeper than the active link.

System ductility and member categories

System ductility

In keeping with the rapid return to occupancy performance criteria for the system, the appropriate structural ductility displacement ductility factor for EBF's with removable links is $\mu=3$ (limited ductility). This has been shown from the Christchurch 2010/2011 MCE intensity earthquake series to result in buildings that effectively self centre and with demand on the EBF active link that is unlikely to require replacement. Note that this will not guarantee no structural repair is required, however this outcome is very likely.

Member categories

For conventional eccentrically braced frames the active link and the collector beam are the same member. The *Steel Structures Standard* NZS 3404 permits some inelastic action in the collector beam. Because of this the collector beam in NZS3404 is classified as a primary seismic member for material and section geometry requirements. In the design procedure for the EBF with removable links the collector beam is designed for the full overstrength design actions i.e. no inelastic action in the collector beam. Because of this the collector beam can be classified as a secondary seismic member for material, section geometry and lateral restraint requirements.

Design considerations

Ductile connection design

The principle source of design guidance for ductile moment endplate plate connections is SCNZ report 14.1 *Steel Connect* (Hyland, Cowie, Clifton, 2008) and HERA Report R4-142 (Clifton, Mago, El Sarraf, 2007). This publication only considers 8 bolt MEP connections with property class 8.8 bolts. The bolted end plate connection of the active link to the collector beam is designed to achieve a balanced design joint capacity, with no significant ductility demand being forced into any one component of the connection and with all potential brittle failure modes protected. The concepts in those documents could be extended to 8 bolt connections with grade 10.9 structural bolts provided these are appropriately fully tensioned (contact SCNZ for bolt properties and methods of full tensioning)

Connections between the active link and the collector beam must be designed and detailed to resist the active link overstrength shear capacity and the capacity design derived bending moment (and capacity derived axial forces if present). This is necessary to ensure that dependable shear yielding occurs in the active link before the capacity of the connection is exceeded.

Welds to end plates

Welds between the active link and the end plate are sized to develop the design tensile capacity of the web and flanges so as to ensure ductile behaviour. Active link flange and web welds may be either complete penetration butt welds or symmetrical fillets welds placed either side of the flanges or web. The active link web is usually welded to the end plate with balanced double-sided continuous fillet welds.

The same approach applies for the welds between the collector beam and the collector beam end plate.

Avoid excessive deformation of the collector beam end plate

It is important to avoid excessive deformation of the collector beam end plate to facilitate ease of removal of the active link. This is achieved by ensuring the tension capacity of the collector beam end plate exceeds that of the active link end plate. This is achieved by:

- 1) Making the collector beam end plate thicker than the active link end plate or
- 2) Adding a backing plate to the second row of bolts (between the flanges) – see HERA Report R4-142 for the increased design capacity using this backing plate

Design actions in bolts

The design actions in bolts must make an allowance for the increase in the bolt tensile loads due to prying action on the bolts. The design procedure in SCNZ *Steel Connect* and HERA Report R4-142 incorporates the prying action when evaluating the various modes of plate behaviour. However, this does not account for the influence of prying on the actual tension force in the bolt. For mode 1 behaviour, the bolt tension for this check can be taken conservatively as the installed bolt tension, as there will be no separation at the bolt line. For mode 2, some separation is possible, however finite element analyses have shown that for fully tensioned bolts this increase is very minor. So for mode 2 also the installed bolt tension can be used to account for bolt prying.

The installed bolt tension for a property class 8.8 bolt is 90% of the design tension capacity so this can be used directly in the interaction equation to account for bolt prying effects in mode 1 and 2 connections. If mode 3 governs for any bolts (which can apply in 16 bolt connections as noted later in this paper) no shear should be assigned to these tension side bolts.

The design procedure for moment end plate connections typically assume the bolts around the tension flange of the beam take the moment and bolts around the compression flange take the shear. This is appropriate for moment resisting frames where shear demand is relatively small. However for EBF active links shear demand is large, relying solely on shear transfer on bolts around the compression flange may not be sufficient. When this is the case, consideration should be given to additional shear transfer via the bolts around the tension flange of the active link beam. The shear capacity of the bolts is downrated due to the presence of axial load using a square root interaction equation, in accordance with NZS 3404 Clause 9.3.2.3.

Collector beam/brace panel zone

The panel is subject to axial load and shear, figure 5. The design actions on the panel are:

1. Force couple due to the link moment
2. Axial force in collector beam
3. Shear force from vertical component of brace force

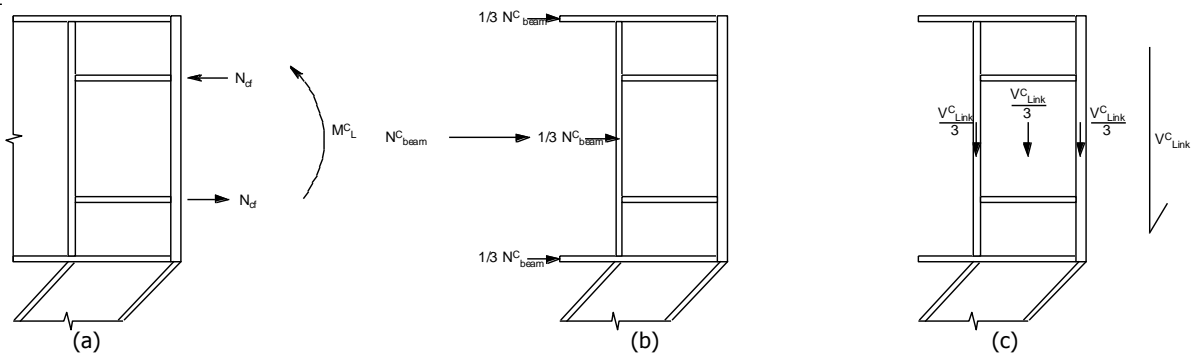


Figure 5: Design actions on collector beam/brace panel zone, 5a: Link moment, 5b: Collector beam axial, 5c: Shear from brace force

These actions contribute to a complex distribution of stresses in the panels zone. A simplified quadratic shear-axial load interaction equation is proposed for the collector beam-brace-active link panel zone (see Figure 5 for location of this panel zone).

$$\left(\frac{V_p^*}{\phi V_p} \right)^2 + \left(\frac{N_p^*}{\phi N_p} \right)^2 \leq 1.0$$

To utilise this interaction equation simplifying assumptions need to be made about the distribution of load carried by the panel zone web, flanges and horizontal stiffeners.

The first simplifying assumption is to consider the panel zone as an upper and lower stiffened tee section (figure 6). The tee section flange and horizontal stiffeners will only be effective in resisting axial loads.

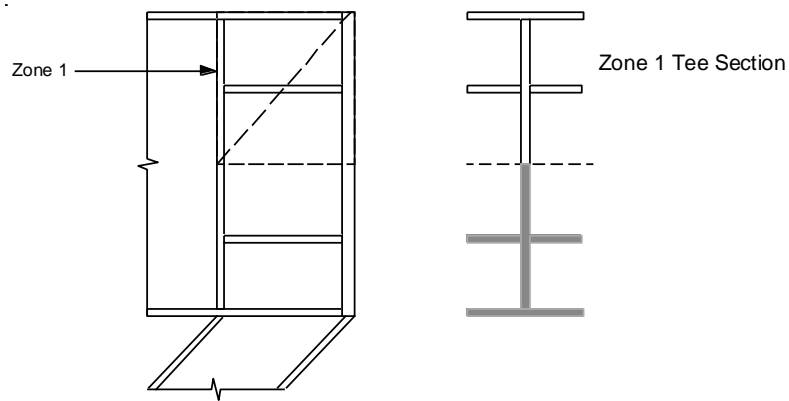


Figure 6: Tee section

The principle of load superposition will be used to determine the axial design actions in each panel zone tee section. The worst load condition for a tee section will occur when the axial force in the collector beam is additive with the force couple generated by the link moment. For example in figure 5 the maximum panel zone compressive actions will occur in the upper tee section.

The second simplifying assumption is that the collector beam and brace loads are carried equally by each flange and web (figure 5b, 5c).

An example of this panel zone check methodology is found in Steel Advisor article EQK 1007.

This simplified check is currently being verified by advanced finite element investigation by the New Zealand Heavy Engineering Research Association (HERA). The results of the investigation will be reported in HERA Report R4-145.

Endplate thickness

The tension capacity of a moment endplate connection is found by considering three failure modes (figure 7), the capacity of the connection will correspond to the mode with lowest capacity. These modes are:

- Mode 1 – plate yielding
- Mode 2 – plate yielding and bolt extension
- Mode 3 – bolt failure

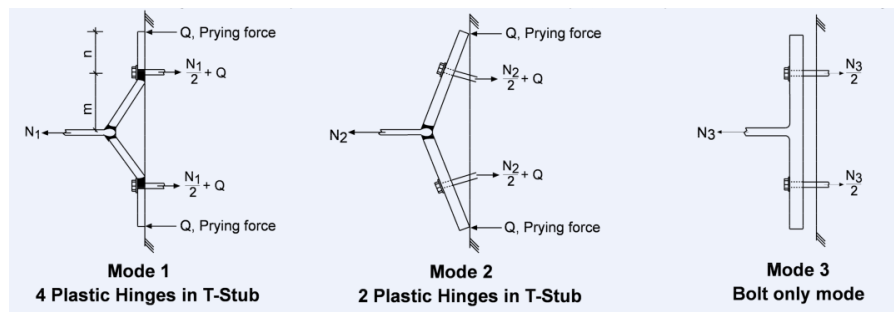


Figure 7: Three modes of end plate behaviour

The endplate is sized to ensure mode 3 behaviour has the highest capacity of the three modes. Design equations for the tensile capacity of each end plate mode of failure are presented in SCNZ report Steel Connect while those using backing plates are given in HERA Report R4-142.

To ensure the bolts do not fail in shear, there is an additional requirement to limit the plate thickness.

$$T_p < 0.9d_f$$

Where

T_p is the plate thickness

d_f is the bolt diameter

Design Actions in Collector Beam

For EBFs with removable links, the collector beam overstrength design actions may not be reduced to 80% as per clause 12.11.7.2 in NZS 3404 to avoid inelastic demand in the collector beam.

Detailing Issues

Stiffener Arrangements

Vertical stiffeners/end plate on the collector beam must align with brace flanges. Similar horizontal stiffeners on the collector beam must also align with the active link flanges. Figure 8 shows a fracture in a carpark building in Christchurch where the collector beam flange fractured in the Christchurch earthquake due to the offset of the vertical stiffener to the brace flange.



Figure 8: Fracture due to Offset of Vertical Stiffener to Brace Flange

Slab support over link

The replaceable link section depth is less than the collector beam depth. This means that the floor slab is not supported across the link length, which could range from 0.6 - 0.9 m. Where the floor slab is a composite metal deck slab, the slab support depends on whether the decking is spanning parallel or perpendicular to the active link. In all cases during construction the decking will be supported off the top of the active link by a timber packer which is removed after the concrete has set.

When the steel decking is spanning parallel to the active link, it is effectively supported off the secondary beams on either side of the active link.

When the decking is spanning perpendicular to the active link (ie as shown in Figure 9), then additional reinforcement will be needed to span across the gap and develop positive moment capacity in the cover slab above the gap. This should be designed to carry the concentrated load specified by NZS 1170.5 for the occupancy of the floor with this load placed on the slab over the centreline of the active link. If the EBF is at the edge of the slab, then there will be a full depth stop-end and the bar can be placed near the bottom of this stop-end. If the EBF is not near the edge of the building the decking will be spanning over the link as shown in Figure 9 and the reinforcement should be placed directly onto the top of the decking and span a development length plus the slab depth past the active link on each side.

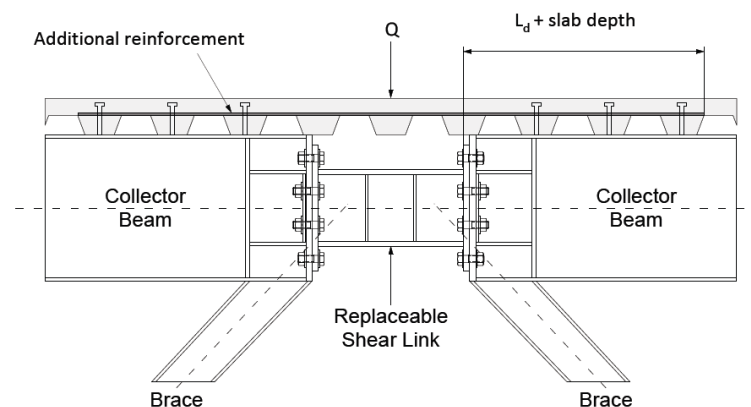


Figure 9: Additional Reinforcement for Floor Slab across Link

Column baseplates

Columns of seismic loading resisting systems are most vulnerable to plastic hinging at their bases if the base connections possess a high degree of fixity and the inter-storey drifts are high. To ensure the low damage performance objectives of the EBF with removable links system are realised, column bases should either be pinned or semi-rigid with flexural yielding of the column base suppressed by base rotation that occurs at a lower moment than the column section moment capacity.

Even nominally pinned base details, such as that shown in figure 10 will provide base fixity that should be modelled.

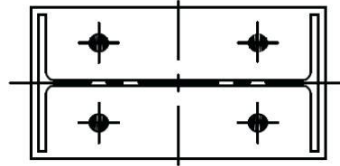


Figure 10: A Typical Nominally Pinned Base Connection

Column base stiffness is best represented using rotational springs. The formulae for rotational springs are as follows (from NZS 3404 Clause 4.8.3.4.1 (a) and (b) and rearranged):

$$k_o = 1.67 \frac{EI_c}{L_c} \text{ (Fixed)}$$

$$k_o = 0.1 \frac{EI_c}{L_c} \text{ (Pin)}$$

Semi-rigid base connections have been the subject of research in North America. There is a current research programme in Taiwan testing pinned and semi rigid base connections. New Zealand research into semi rigid base connections is also planned. In practice, a detail such as that shown in Figure 9 will provide rotational stiffness closer to that of the fixed base so the rotational stiffness for the fixed base should be used. If considering a semi-rigid base which will allow rotation within the connection rather than the column itself, provisionally use a stiffness of

$$k_o = 1.0 \frac{EI_c}{L_c}$$

Design Aids

SCNZ Steel Connect

Steel Construction New Zealand report *Steel Connect* contains pre-engineered moment end plate connections for a range of sections. These can be used as a starting point in designing the removable link end plate connections. The 250UC and 310UC range of standard connections use an 8 bolt arrangement (2 bolts in each row). *Steel Connect* adapts the design method set out in (SCI P207, 1995). For applications not covered by these pre-engineered solutions, specific design can be undertaken using the design methodology in this document.

Final Considerations

Not Suitable for Every Application

The removable link concept presented in this article is not suitable for every EBF application. This will be the case when the connections actions are very high making it difficult to achieve practical connection designs. When this occurs consideration should be given to:

Use of Property Class 10.9 Bolts

The actions on the bolts for the removable link connections are large. For some configurations it is difficult to 'fit' the number of bolts required in the connection if using property class 8.8 bolts. If this is the case property class 10.9 bolts may be used. Property class 10.9 bolts are available by indent in New Zealand with 3 -5 months lead time.

Or

Use of 16 bolt connections

An example of a 16 bolt MEP connection is shown in figure 11. There is limited design guidance for a 16 bolt MEP connection.

Note that a 16 bolt arrangement requires wide endplates and hence wide collector beam and active link flanges. For a short active link operating in the shear mode, the flanges are not prone to local buckling hence their slenderness ratio can be taken as that for category 3 members if subject to design axial load across the active link and up to the yield limit from NZS 3404 Table 5.2 if the active link is not subject to design axial load across the active link.

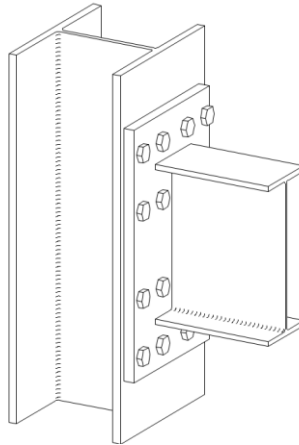


Figure 11: 16 Bolt Moment End Plate Connection

Custom Welded Links

Using custom welded links allows the strength of the link be tailored to the demand. This reduces the capacity derived actions on the rest of the EBF members. Custom welded links are more expensive than a hot rolled section of equivalent weight. However the savings in cost of secondary elements may more than offset the increase in cost. Full strength butt welds are required between the web and flange of welded links.

More efficient transportation of frame elements

The removable link concept allows the collector beams and braces to be welded to the EBF column. This can be done over 3 – 4 stories and transported as one element to site.

Alternatively the active link can be bolted into the collector beams in the fabrication shop and the collector beam/link/brace subassembly transported to site in the same manner as for a welded active link/collector beam/brace system.

A variation on this with the SHS brace is that the collector beam/active link is shop assembled and brought to site, with the SHS being birdsmouthed, fitted over the gusset plate on site, pinned in place with a locator bolt as shown in Figure 1 and site welded. That system has been successfully used on several multi-storey EBF and CBF braced frames in New Zealand (see eg HERA Steel Design and Construction Bulletin, Issue No 46, pp 16-17).

Conclusion

A method for designing eccentrically braced frames with replaceable links has been presented in this article. The replaceable link uses moment end plate connections to connect the link member to the collector beam. Careful detailing of the connection is required to ensure good performance. The overall design of EBF members using the replaceable link concept is similar to conventional EBF frames. The EBF with replaceable links has several advantages, the most important being that it allows for quick inspection and replacement of damaged links following a major earthquake, significantly minimising the disruption to the structure and allowing the building to be rapidly returned to service with any link replacement being undertaken during subsequent scheduled maintenance periods.

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