Development and Research of Eccentrically Braced Frames with Replaceable Active Links

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Introduction
Ductile eccentrically braced frames designed in accordance with the New Zealand Steel Structures Standard, NZS 3404, provide life safety during a design level or greater earthquake; however, the eccentrically braced frame active link may sustain significant damage through repeated inelastic deformation. This may necessitate post-earthquake replacement of the active link. A bolted replaceable active link can be used to facilitate replacement after a strong earthquake, which reduces repair costs.

New Zealand design guidance for the seismic design of steel eccentrically braced frames was first published in 1995 by the New Zealand Heavy Engineering Research Association within HERA Report R4-76 and has been widely used in practice. This guidance has been recently updated and now includes seismic design procedures for eccentrically braced frames with replaceable links. This article covers the development and research of eccentrically braced frames with replaceable links. This includes discussions of the comprehensive research programme recently completed in Canada investigating the performance of eccentrically braced frames with replaceable links and finite element analysis undertaken by the New Zealand Heavy Engineering Research Association, to verify the design procedure for eccentrically braced frames with replaceable links.

Performance Requirements of EBF's with Removable Links
The key performance requirements for removable link elements are:
- They must be designed and detailed to achieve ≥ 0.08 radian plastic rotation (shear mode) under the design level or greater earthquake.
- Inelastic demand must be limited to the link element
- Ease of removal and replacement post earthquake

Early Development
The concept of active link beam segments welded to end-plates has been tested extensively in previous cyclic tests programs in the U.S., in which link specimens were fabricated with endplate connections for ease of installation in test setups. However, the end plate connections were often over designed, as the main intent of the experimental research was to evaluate the hysteretic response and performance of the link beam, which in an eccentrically braced frame (EBF) is integral with the floor collector beam. Figure 1 shows a typical setup.

Figure 1: Experimental Testing of Shear Links (AISC, 2007)

Specific investigations of the removable link concept for eccentrically braced frames were first investigated by Stratan and Dubina (2004) at the Politehnica University of Timisoara in Romania. Stratan and Dubina performed tests on I-section links with bolted endplate connections. However, the endplates were flush with the I-section top and bottom flanges, which resulted in observed failure in the connections. This highlights the need for appropriate design and detailing of the endplate connection to suppress non-ductile overload failure modes and the need for robust and relatively simple load paths through the connections. Statan and Dubina conducted two experimental programs. The first experimental program focussed on the active link element (refer to figure 2) and the second program focussed on full scale structures (refer to figure 3). At the frame level, the experimental tests showed that the removable link solution is feasible. Inelastic deformations were constrained to the removable links alone with all other frame members and connections remaining in the elastic range.

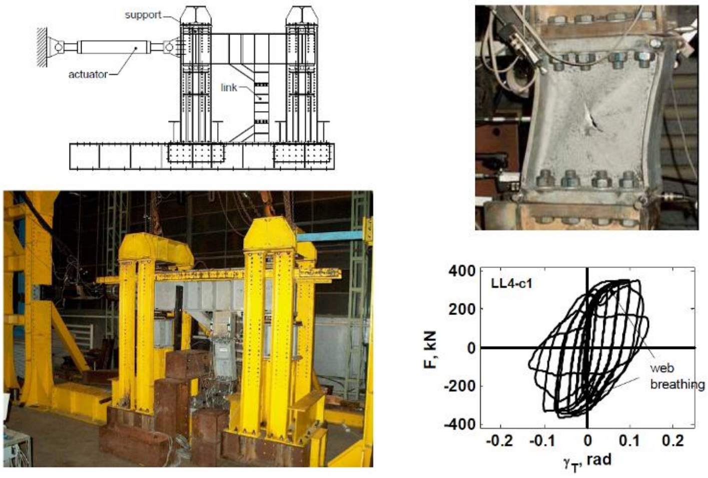

Figure 2: Active Link Element Experiments at the Politehnica University of Timisoara (Stratan et al, 2009)

Figure 3: Full Scale Testing at the Politehnica University of Timisoara (Stratan et al, 2009)

Recent Canadian Research

Experimental Program

Extensive Canadian research and experimental testing on eccentrically braced frames (EBF) with replaceable shear links has been recently completed. This research conducted by Nabil Mansour is presented in detail in his doctorate thesis. Two replaceable link types (see figure 4) with different bolted connections were evaluated and tested:

i) W-sections (I sections) end plate connections, and
ii) Back-to-back channels with eccentrically loaded web connections.

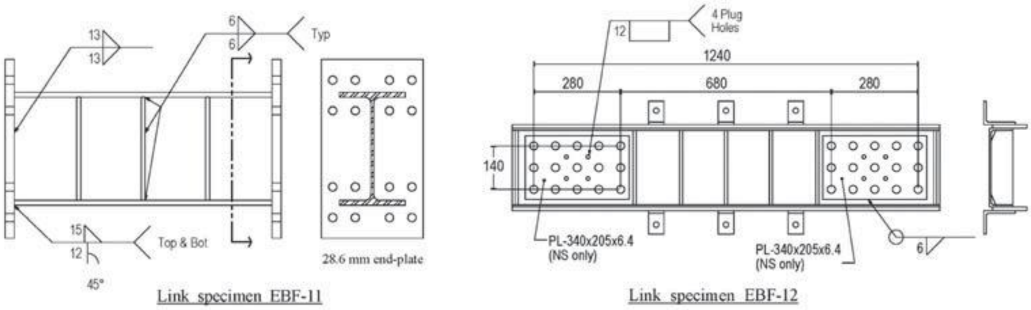

Figure 4: Examples of link specimens tested: a) I section end plate; b) back-to-back channels

The experimental program involved the testing of 13 full-scale shear links: the first 6 specimens at the University of Toronto characterized the local response of the replaceable links; and the remaining 7 links tested at École Polytechnique de Montreal, where a full-scale frame, 7.5 m wide by 4.5 m high, was setup to confirm the local response of the replaceable links and characterize the global frame behaviour.

The replaceable shear link specimens qualify or meet the performance requirements of the steel structures standard if they achieve 0.08 rad plastic link rotation without fracture or severe strength degradation during a standard cyclical load test. The loading protocol specified in appendix S6 .3 of the 2005 AISC seismic provisions for structural steel buildings was used for the test program (Mansour et. al. 2008).

University of Toronto

Test Setup

The subassemblage chosen for the experimental program models a portion of a typical chevron bracing EBF system. The test setup was devised to reproduce the force distribution and deformation experienced by the link in an EBF frame. The replaceable link elements are connected at both ends to floor beam sections. The links are subjected to a constant shear force along their length, equal reverse curvature bending moment at the ends, and no axial force. A schematic representation of the testing arrangement is shown in Figure 5. The hydraulic top ram of the MTS test machine applies forces cyclically to the test rig subassemblage, which is transmitted through the connections to the links as a shear force.

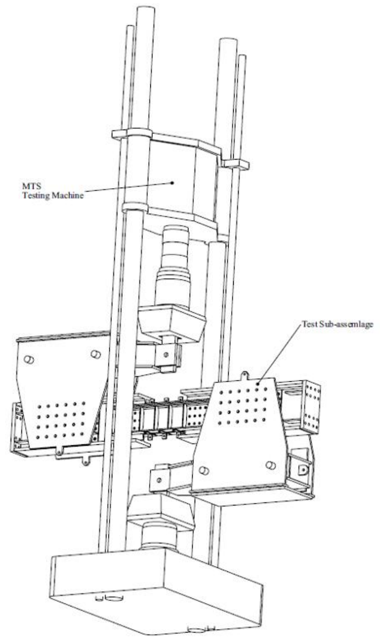

Figure 5: EBF-2 Shear Link Test Setup Schematic (Mansour, 2010)

Table 1 provide a list of the replaceable link test specimens. The dimensions of the specimens were designed to represent the first storey links of a 5-storey EBF designed according to the Canadian Building Code and representative of a high seismic area. Sets of two identical specimens were tested for each link type, to assess the replaceability of the links and the performance of the replaced link in comparison to the original. A total of 6 EBF link specimens with comparable shear yield strengths were fabricated. They comprised of two sets of the double channel links with reinforced top and bottom flanges (EBF-1 and 2), and one set of the welded end-plate W-section (EBF-3).

Table 1: Test Specimens

Group	Specimen	Section	Link Length e (mm)	Intermediate stiffeners	Web connection reinforcement plate	Faying surface condition
End Plate Connection	3A	W360x101	900	3 at 200mm	none	Mill scale (A)
	3B	W360x101	900	3 at 200mm	none	Blast Cleaned (B)
Bolted Web Connection	1A	C 310x31	680	4 at 200mm	6.35mm welded on 4 sides	Mill scale (A)
	1B	C 310x31	680	4 at 200mm	6.35mm welded on 4 sides	Blast Cleaned (B)
	2A	Cut W310x39	680	4 at 200mm	none	Mill scale (A)
	2B	Cut W310x39	680	4 at 200mm	6.35mm welded on 4 sides	Blast Cleaned (B)

Results

A summary of test results is provided in table 2. The behaviour of the end plate links closely resembled that of traditional chevron EBFs with shear link beam sections. Full hysteretic curves were observed. Both specimens had an almost identical loading history response. At the 0.09 radian cycle, cracks were observed at the corners of the link web, initiating from the weld access holes. Crack propagation in the link web extended from the access hole parallel to the flanges then vertically up at the connection interface, until fracture of the web panel at completion of the 0.11 radian cycle.

The double channel shear links experienced bolt slippage at the link connections. As a result, pinching was observed in the hysteretic curves. Connection rotation was responsible for 35% of link rotation in EBF-1A, and increased to 54% in EBF-1B. The bolt holes in the floor beam were already ovalized from the original link specimen. This indicated that thicker web reinforcement plates for the floor beam are required to prevent bolt hole elongation.

Table 2: Test results

Specimen	Peak link rotation (rad)		Maximum link shear force (kN)	Failure mode
	γ	γ_p		
UT-1A	0.130	0.125	1438	Cracks in link web at intermediate stiffener. Test terminated due to problems in setup before link failure
UT-1B	0.170	0.166	1478	Fracture of link web at intermediate stiffener
UT-2A	0.073	0.066	1132	Fracture at web connection
UT-2B	0.110	0.102	1311	Exceeded test setup deformation limits
UT-3A	0.110	0.104	1274	Fracture of web initiating at weld access hole
UT-3B	0.110	0.105	1279	Fracture of web initiating at weld access hole

The hysteretic energy dissipated by the replaceable shear link as it undergoes inelastic behavior was compared to that dissipated by an elastic perfectly-plastic (EPP) link. The double channel links dissipated 70% of the EPP link. This is due to the observed pinching in the hysteresis due to bolt slippage in the connection. Nonetheless, as the bolts were pretensioned, energy was also dissipated as the webs of the link and the floor beam slipped with respect to each other. However, the W-section links with the end plate dissipated hysteretic energy slightly less than the EPP link. The end plate connection remained rigid throughout the test, and all the plastic link rotation was due to the inelastic link shear deformation.

École Polytechnique de Montreal
Test Set-up

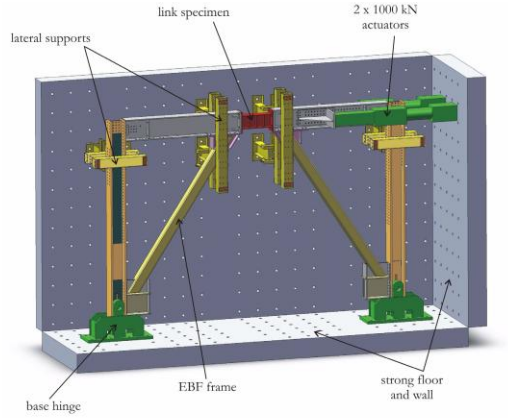

Figure 6: EBF frame and test setup (Mansour, 2010)

The full scale sub-assembly EBF frame test set up is shown in figure 8. The test set up was designed to represent the first storey of a 5 storey EBF frame designed in accordance with the appropriate Canadian codes for a high seismic area.

The lateral load was applied onto the collector beam reacting into the strong wall. The actuators push and pull on the EBF frame, imposing predetermined cyclic shear deformations on the link. The loading system subjected the link to constant shear along its length, and reverse curvature bending. The actuators were connected to the frame in such a manner that half of the applied actuator load is transferred axially through the link. This represents the worst case loading scenario that can be imposed on an EBF. Lateral bracing of the floor beams was provided at the two ends of the link and also for the column sections, as shown above. The setup was instrumented to record both global structural response and local element responses including the link specimens and their connections.

The specification of each specimen, including the primary features of the link-to-beam connection, link length, and link stiffener spacing are summarized in table 3. The first four specimens, 11A, 11B, 12 and 13 were tested as part of the bare steel frame. Specimen 15 was tested to evaluate the response of a shear critical double channel link with an eccentrically welded web connection. Specimens 14 and 16 were tested as part of the steel frame with a composite steel deck on top of the floor beams.

Table 3: Test Specimens

Group	Specimen	Section	Link Length e (mm)	Intermediate stiffeners	Web connection reinforcement plate
End Plate Connection	11A	W360x72	800	3 at 200mm	N/A
	11B	W360x72	800	3 at 200mm	N/A
Bolted Web Connection	12	Cut W 250x24	680	5 at 150mm	6.35mm welded all around
	13	Cut W 250x24	680	7 at 99mm	6.35mm welded on 4 sides
	14	Cut W 250x24	680	7 at 99mm	6.35mm welded all around
	16	Cut W 250x24	680	5 at 150mm	4.76mm welded all around
Welded Web	15	Cut W 250x24	590	5 at 150mm	6.35mm

Test Results
The links tested exhibited very good ductile behaviour, developing stable and repeatable yielding. The end-plate links exhibited a full-hysteretic response that was very similar to that of traditional eccentric braced frames with a continuous beam. The web connected links exhibited a slightly pinched hysteresis because of the localized nonlinear response of the bolted connection; however, the web connected links developed larger total deformation capacity than the end plate connections. All the specimens exceeded the inelastic link rotation angle of 0.08 radians. See table 4.

Table 4: Test Results

Specimen	Peak link rotation (rad)		Maximum link shear force (kN)	Total frame drift (%)		Failure mode
	γ	γ_p		θ	θ_p	
EPM-11A	0.101	0.095	1120	1.52	1.12	Fracture of top flange near weld
EPM-11B	0.119	0.112	1170	1.71	1.40	Fracture of link web at intermediate stiffener
EPM-12	0.111	0.106	916	1.32	1.12	Fracture of link web at intermediate stiffener
EPM-13	0.111	0.104	976	1.18	0.85	Fracture at web connection
EPM-14	0.111	0.103	1108*	1.37	1.06	Fracture of link web at intermediate stiffener
EPM-15	0.130	0.125	890	1.54	1.31	Fracture of link web at intermediate stiffener
EPM-16	0.090	0.087**	855	0.95	0.80	Fracture of link web at intermediate stiffener

*Link shear includes the resistance provided by the composite slab
**Load decreased to 50% of peak load only at the end of second half of the 0.09 radian cycle

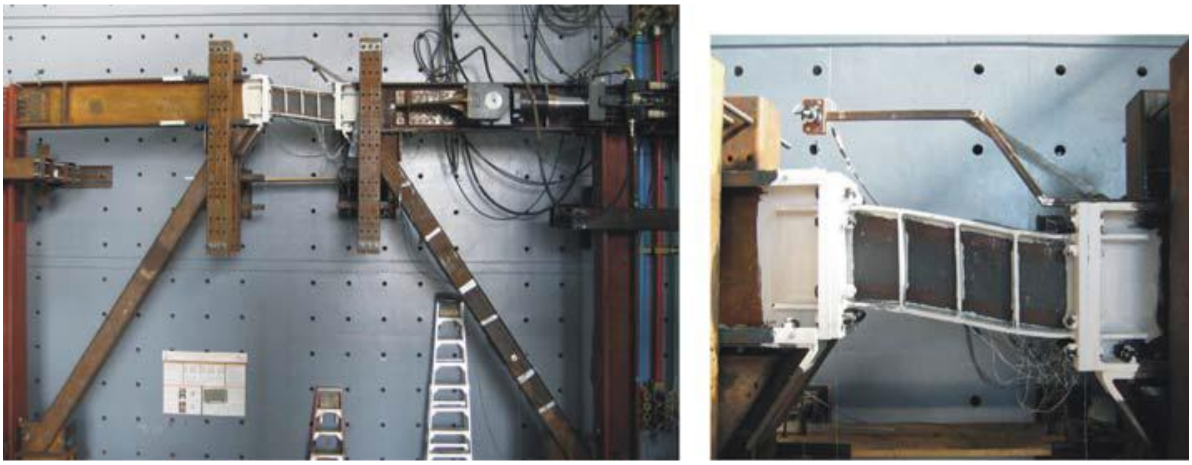

Figure 7: Full scale testing of Links with End Plates (Mansour, 2010)

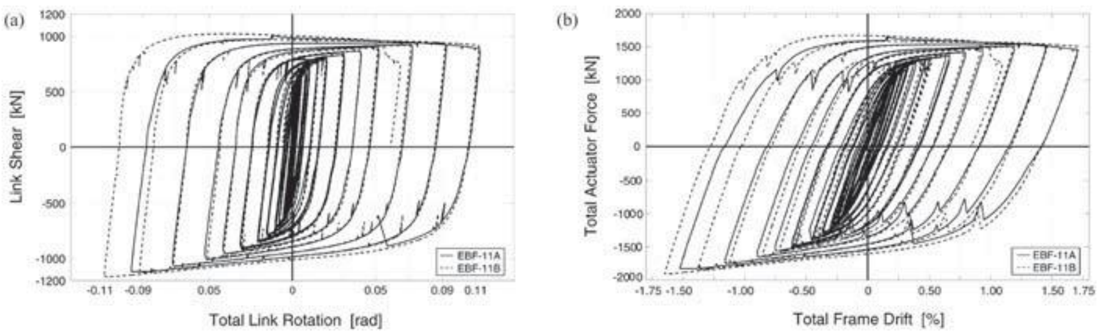

Figure 8. Response of Replaceable Links with End Plates (Mansour, 2010)

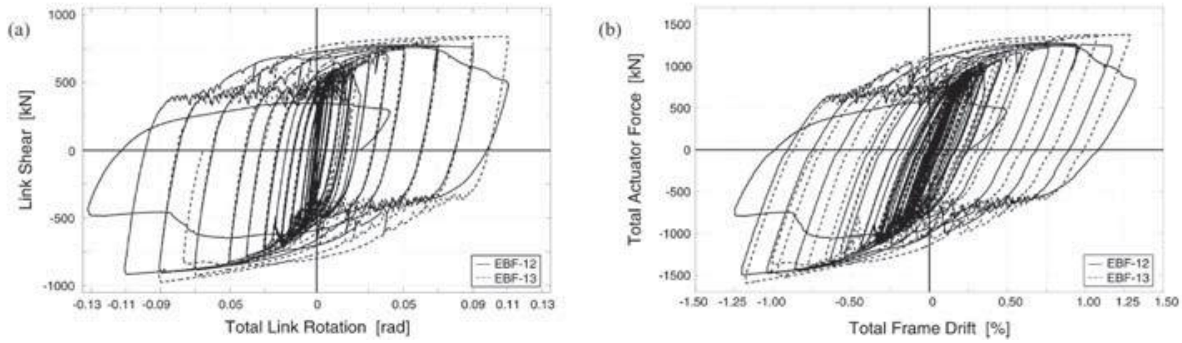

Figure 9: Response of Replaceable Links with Back-to-back channels (Mansour, 2010)

Investigation into Replaceability of Links

Mansour investigated the on-site replaceability of the link sections. The back-to-back channels active link type could be replaced even in the presence of residual deformations of 0.5% drift, although such a level of drift would render the building unusable. Both the bolted and welded connections were investigated. While both procedures proved feasible, the welded technique provided greater flexibility and ease for the erection of the replacement links. It was also experimentally demonstrated that the repair of the damaged concrete slab can be done on-site with the usual existing concrete slab repair procedures and products.

Mansour also investigated the replacement of the end plate connected links. It was observed that the replacement of the end-plate connected links required realignment of the test frame to its plumb position before the link specimens could be fitted in-between the floor beams. This could prove more difficult to achieve in a real building. Detailing the end-plate connected links short by a few millimetres and filling the gap between the link end-plates and floor beam end-plates with shims is possible. Test conducted by Sumner (2003) using beams fabricated 5mm short with the gaps filled with finger shims were done, where no adverse consequences or differences in connection behaviour were observed.

Canadian Research - Summary of Key Findings

All link specimens (with exception to UT-2A, a bolted web connected link) tested at University of Toronto and École Polytechnique de Montreal exhibited stable hysteretic shear yielding. These specimens met and exceeded the ductility acceptance criteria of 0.08 radian inelastic link rotation prescribed by both the Canadian Standards and 2005 AISC Seismic Provisions.

End-plate connected shear replaceable links displayed a consistent full hysteretic response up to γ_p=0.104 radians, that is very similar to that in traditional EBF construction, where the yielding link beam is an integral component of the floor beam.

Replaceable links with web-bolted connections exhibit a more pinched response, but because of the connection rotations, are able to sustain larger inelastic link rotations, on average γ_p=0.119 radians and a maximum of γ_p=0.166 radians.

Repair of Pacific Tower, Christchurch

The repair of the 22 storey structural steel framed Pacific Tower in Christchurch following the 2010/2011 Christchurch earthquake series featured the world's first building application of the bolted replaceable links for eccentrically braced frames. The Pacific Tower had combined conventional EBF and moment resisting frame seismic resisting systems and was completed in 2010. The Pacific Tower performed well during the earthquake series, but required 42 EBF links to be replaced. These links were successfully replaced with bolted replaceable active links (mostly) but also some fully welded ones. The performance, damage assessment and repair of the Pacific Tower eccentrically braced frames are described in more detail in (Gardiner et al, 2013).

HERA Finite Element Analysis

Advanced finite element analysis (FEA) has been undertaken by the New Zealand Heavy Engineering Research Association (HERA) to verify the performance of eccentrically braced frames with removable link (Mago, 2013). The FEA is based on the SCNZ developed design example (Cowie & Fussell, 2013). The principle objective of the FEA is to assess if the methodology supressed inelastic demand in secondary elements.

Description of Model

The model was built in ABAQUS/CAE. The solver used is ABAQUS Standard version 6.12-001.

The FEA model represents one storey of an eccentrically braced frame with detailed modelling of the active link region and a 1m wide slab included. See figure 10.

The collector beam is 610 UB101 with a 32mm thick Grade 350 end plate. Eight M30 (category 8.8/TB) bolts are connecting the collector beam to the removable link on each side. The removable link is 310 UC118 Grade 300 member and its end plates are 25mm thick Grade 350 steel. There is one 12mm thick Grade 250 web stiffener on one side of the removable link, which is "butt welded" to the flanges and web. The columns are 350 WC280, while the braces are 250 UC90. Both the columns and braces are rigidly connected to the collector beam.

The FEA simulation was performed in two dynamic implicit quasi static steps.

Step-1: Time 1 represents the tightening of all M30 bolts.

Step-2: Cyclic enforced displacements have been applied on the left and right collector beam to column connections as shown in Figure 10 and Figure 11. The duration of Step-2 is 74 units of time, with each unit of time representing one peak displacement. This loading regime was developed as follows:

1. It is based on the active link undergoing the plastic rotation testing regime developed by the AISC protocol for active links specified in the attached document. See Fig 4.1(a) of (Richards and Uang, 2003).
2. The entire frame was modelled as shown in HERA Report R4-145 (Mago, 2013) and the displacements onto the frame were determined that are necessary to develop the link rotation angle from the AISC protocol.
3. These displacements were then enforced onto the frame to generate the results
4. The concrete slab was modelled elastically as this makes running time much shorter. However it means that if the slab is modelled with the actual elastic stiffness of the concrete it will over represent the slab contribution to the subassemblage strength when the slab operates in the elastic range. To overcome this, two slab elastic stiffnesses were modelled, with the larger one applicable up to the end of the slab elastic range and the smaller one applicable for larger cycles of loading. This is described in more in HERA Report R4-145.

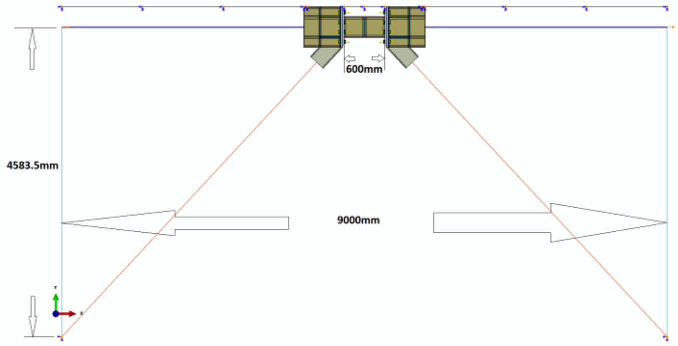

Figure 10. FE model of the 1-storey EBF with removable link (Mago, 2013)

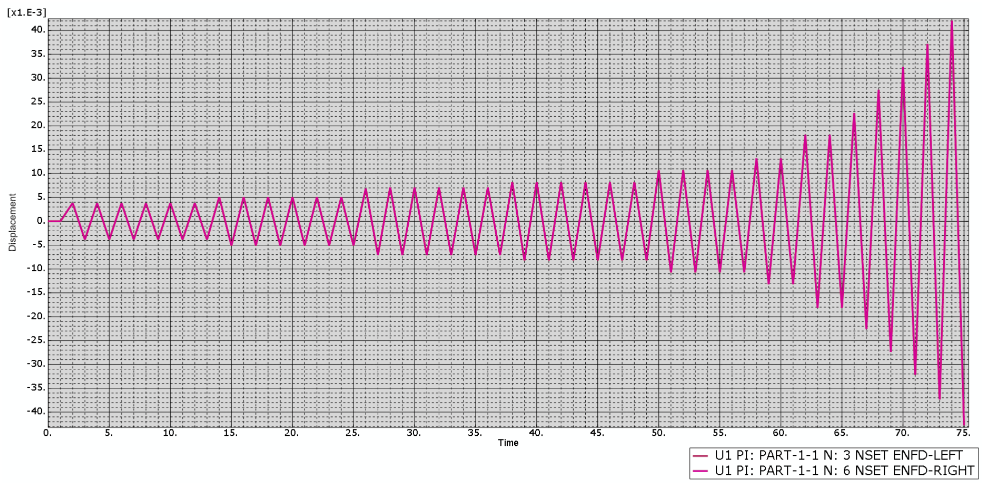

Figure 11: Cyclic enforced horizontal displacement applied at the left and right beam to column connections simultaneously (Mago, 2013)

Results of Finite Element Analysis
The results of the FEA show that the bolted design procedure is satisfactory in suppressing inelastic demand in all components except the active link. Very limited plastic strains were observed in the collector beam adjacent to the endplate under the highest cycles of loading, these plastic strains are minor and would not be visible or detrimental to structural performance in practice. Refer to figures 12 and 13.
The initial clamping force between the active link endplate and the collector beam endplate induced by the bolt full tensioning is reduced but not eliminated by active link stretch due to geometrical deformations. Because compression contact is maintained slip on that plane will not develop during the earthquake excitation.

The bolt tension forces do not increase which means bolt failure will not happen during the earthquake excitation.

The welded web stiffener attracts significant local plastic strain and demonstrates the potential benefit of a bearing sandwich stiffener which allows the web to deform plastically independent of the stiffener.
The results from the finite element analysis confirm that the design procedure developed provides a satisfactory solution for a bolted replaceable active link.

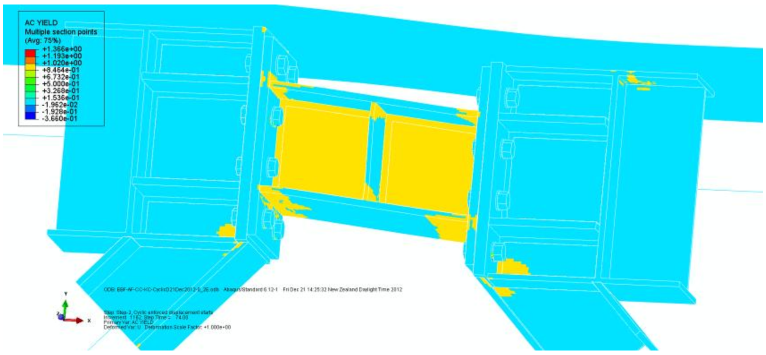

Figure 12: Active yield flag telling if the material is yielding or not following the last cycle (Mago, 2013)

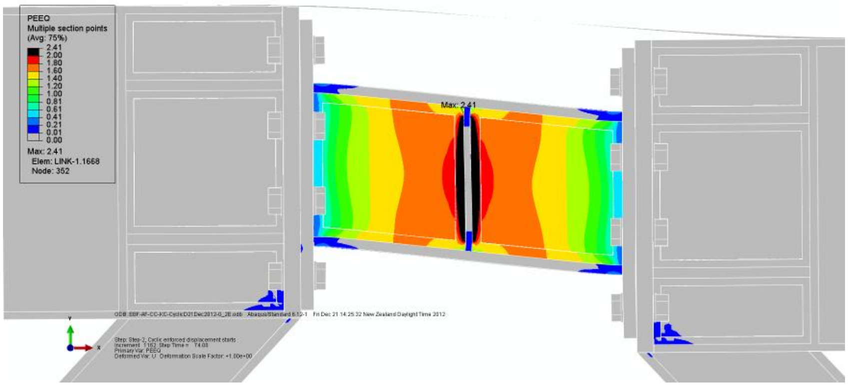

Figure 13: Equivalent plastic strain contour for the last cycle (Mago, 2013)

Conclusion
This paper provides an overview of the research, experimental testing and finite element analysis of eccentrically braced frames with replaceable shear links. The results of the comprehensive Canadian full scale testing demonstrated that the replaceable links achieved the inelastic link rotation code requirements. The New Zealand Heavy Engineering Research Association finite element analysis verified the design procedure developed for New Zealand (Clifton, Cowie, 2013) achieved the objectives of suppressing inelastic demand away from the active link, allowing the rest of the structural system to remain elastic under earthquake excitation.

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