

## DESIGN OF REPLACEABLE-LINK ECCENTRIC BRACED FRAMES IN POST-EARTHQUAKE CHRISTCHURCH

J. J. G Ramsay<sup>1</sup>, A. Fussell<sup>2</sup>, and R. G. Wilkinson<sup>3</sup>

### ABSTRACT

The recent Canterbury Earthquake sequence has highlighted the need for minimizing downtime associated with post-earthquake building repair. In addition to maintaining a building which is safe and repairable after a large event, developers in Christchurch are increasingly interested in pursuing new seismic systems and technologies which limit damage and reduce the impact of repairs following earthquakes. For such systems to compete with conventional seismic systems in the commercial market, many factors including seismic performance, cost, erection time, materials, procurement, and design and construction challenges must be considered.

In this paper, the use of replaceable-link eccentric braced frames (EBF's) as a seismic structural system is discussed, based on a case-study of a recently designed commercial office building development in Christchurch. It is the first known use of replaceable-link EBF's in New Zealand.



Figure 1. Street level render of the Three35 project.  
Render courtesy of Jasmax.

### Introduction

In Christchurch and New Zealand, current seismic design philosophies have very much favored the capacity design approach developed in the mid 1960's by Professors Park and Paulay at the University of Canterbury which essentially is based around a well detailed "fuse" or weak link element. The advantages of this capacity design approach is firstly that there is a dependable ductile mechanism of inelastic deformation of the building to preserve life-safety, and secondly, that elements outside the yielding regions need only be designed for seismic "overstrength" of the weak link and are thus are designed for lower loads than had the building been designed for full seismic loading with no ductility. This generally results in a cost-effective structure. Currently, the New Zealand Building Act recognizes this approach, and is very much geared up solely towards life safety for Ultimate Limit State (ULS) events.

<sup>1</sup>Senior Structural Engineer, Ruamoko Solutions Limited, PO Box 12294, Christchurch, 8242

<sup>2</sup>Manager, Steel Construction New Zealand (SCNZ), PO Box 76403, Manukau City 2241

<sup>3</sup>Senior Structural Engineer, Ruamoko Solutions Limited, PO Box 12294, Christchurch, 8242

In addition to the immediate welfare of persons displaced by building damage, the recent Canterbury Earthquake sequence has demonstrated that for a small country like New Zealand with only three main commercial centres and a relatively spread population and resource, the temporary loss of building stock either for repair or rebuilding due to such damage has a large financial impact on the country as a whole. In addition this impact has significant flow on effects such as the loss of commercial production due to such disruption, and potentially the permanent loss of commercial activity in the region. Delays in rebuilding some years after the seismic sequence began also highlight the lack of local resource needed to carry out repairs.

Reserve Bank data from September 2012 (Parker, Steenkamp, 2012) estimates that the Canterbury Earthquake sequence has a repair bill of approximately 10 percent GDP which makes it a heavier burden on the national economy than Japan's 1995 Kobe earthquake (around 6.5 per cent of GDP) and the recent 2011 Japanese Tohoku earthquake and tsunami (around 3 to 4 percent of GDP).

It is no surprise therefore that insurance companies, governments, local bodies, building owners and developers are keen to look into systems and technologies that focus on damage avoidance and Repair Impact Minimization (RIM). One such technology is the use of conventional steel EBF's with replaceable shear-links that are bolted in place to adjacent overstrength collector beams. Whilst this system supports the premise of "damage avoidance", it is really in effect a RIM system. The intent is that the repair of the seismic system by introducing new shear-links is minimized by ease of replacement, and minimal damage to adjacent elements.

The application of this technology has been studied in 2010 research by Nabil Mansour at the University of Toronto in his PhD thesis titled "Development of the Design of Eccentrically Braced Frames with Replaceable Shear Links" (Mansour, 2010).

The use of EBF's with bolted shear-links has been selected on the Three35 project in Christchurch and is the first use of this technology in New Zealand.

### **Performance of Conventional EBF's in the Canterbury Earthquake Sequence**

For an earthquake series larger than the design level event, buildings on whole in Christchurch generally performed well in the recent Canterbury Earthquake sequence in terms of satisfying the NZ Building Act requirements which are focused primarily on life safety in a ULS event.

The number of steel structures in the Christchurch area is relatively low due primarily to the region's accessibility to cheap concrete aggregates, and ongoing steelworker labour disputes which began in the 1970's and lasted until the end of the recession in the 1990s. As such, most of the steel buildings in the area have been designed to modern seismic provisions.

Taking into account the above however, generally steel framed buildings in the Christchurch region have performed well and have exhibited a damage threshold higher than expected compared with concrete structures of similar ages. A surprising aspect has been the resilience of EBF's when cast integrally with concrete slabs. So unexpected was this high performance that the University of Auckland has proposed a research project into the beneficial effect of the concrete slab on the performance of EBF's, and the overall post-earthquake building displacements.

Conventional EBF's were first proposed in Japan by Fujimoto et al. (1972) and Tanabashi et al. (1974). In the United States, the EBF system was first studied by Roeder and Popov (1977) and the system rapidly gained acceptance by the design profession.

Conventional EBF's have generally performed better than expected, and have exhibited self-centering properties. One of the tallest examples in Christchurch is the twelve-storey HSBC Tower, which although designed for a lower ductility than is typical for an EBF system, suffered only minor seismic damage and has been passed fit for reoccupation without requiring structural repair to the seismic load resisting system. According to newspaper and television news items, this tower was the first central city high rise building to be reoccupied after the February 22<sup>nd</sup> event with tenants returning to work in the building in May 2011.

In contrast there have been many prominent examples of buildings with concrete structural systems that have been well designed to recent seismic provisions that have (generally with the exception of stairs) satisfied their design requirements but are simply unable to be feasibly repaired. An example of this is the Hotel Grand Chancellor on Cashel Street. As mentioned above, most of the stock of high-rise buildings in Christchurch were concrete structures designed for more ductility than their steel counterparts, however,

whether justified or not, there is a perception in Christchurch that earthquake damage from concrete structures is more difficult to fix following ULS events, and that steel buildings have generally performed “better”. To the general public, this perception is exacerbated by the fact that the two buildings which caused the most fatalities and attracted the most media attention in the February 22<sup>nd</sup> event in Christchurch were both concrete shear wall buildings.

The performance of conventional EBF technology has been such that no significant changes have been made to current design practice for conventional systems and the current design provisions are readily adaptable to low damage solutions.

### University of Toronto Research into Replaceable-Link EBF's

Research from the University of Toronto entitled “Development of the Design of Eccentrically Braced Frames with Replaceable Shear Links” carried out by PhD candidate Nabil Mansour (Mansour, 2010) involved the experimental verification of the removable link with extended endplates. Full scale testing and finite element work was undertaken on six frames with removable links, of these two were with extended endplates similar to those used on the Three35 project.

This research showed the performance of the removable links was similar to that of conventional eccentrically braced frames with the links exhibiting very good ductile behavior, developing stable repeatable yielding.

The testing undertaken showed that the replaceable-links achieve and exceeded the ductility acceptance criteria of 0.08 radian inelastic link rotation and in the testing undertaken, the on-site replaceability of the link sections was confirmed even in the presence of residual deformations of 0.5% drift.

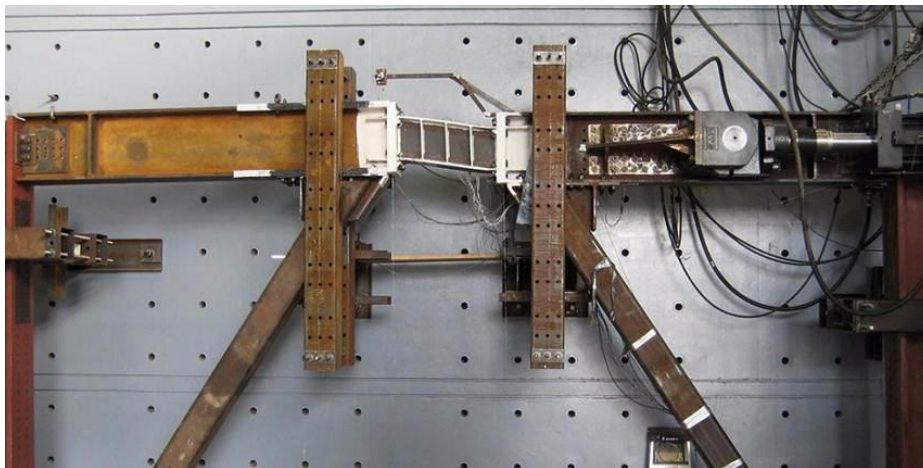


Figure 2. Test setup – frame at 1.7% drift, and total plastic link rotation of 0.113 radians. Photo courtesy of University of Toronto.

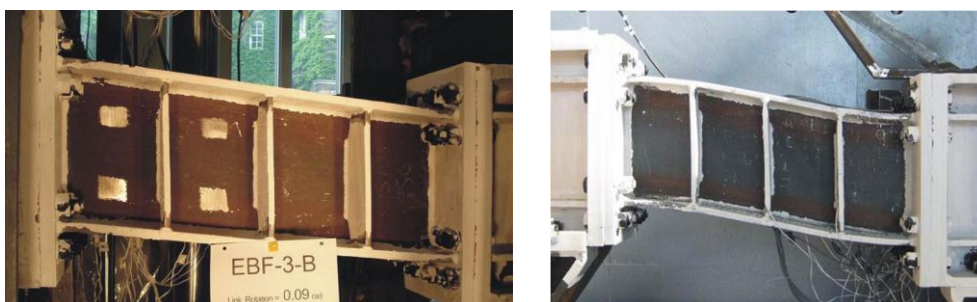


Figure 3. Test links at 0.09 radian rotation, and 0.11 radian rotation. Photos courtesy of University of Toronto.

## Three35 Project Description

The Three35 development is located at 335 Lincoln Road in Christchurch, and comprises of three stand-alone buildings. These include two three-storey office buildings, and a three-storey carpark building. The replaceable-link EBF's were used only in the two office buildings, and the carpark is conventional concrete shearwall design. Each office building has a footprint of approximately 1,100m<sup>2</sup>, and are around 50m x 22m in plan dimension. The office buildings only are discussed in this paper.



Figure 4. Building location (not to scale).  
Image courtesy of Jasmax.

The site is located at the western corner of the intersections of Lincoln Road and Bernard Street in Addington, Christchurch as shown in Fig. 4 above.

The vertical load carrying structure for the office buildings generally consists of roof and floor members spanning in the longitudinal direction of the buildings between primary structural members running in the transverse direction. At roof level, DHS steel purlins span between steel UB rafters/portal frames. At levels one and two, the floor system consists generally of a precast concrete rib and timber infill system with an insitu concrete topping, spanning between composite steel beams supported by steel columns.

The lateral load resisting system for these buildings comprises steel EBF's. Specifically two "D-braced" and one "K-braced" frames in the longitudinal direction, and two "D-braced" and two "K-braced" frames in the transverse direction. The EBF's are designed for a structural ductility factor of  $\mu=3.0$ .

The precast perimeter walls at one end of the building are designed to be butt-jointed with limited stiffness, and are not intended to resist lateral loads and are deliberately detailed as such.

Seismic inertial loads are transferred to these EBF elements through steel roof cross-bracing in the roof plane and through in-plane diaphragm action at floor levels.

For both office buildings, the geotechnical conditions varied, and with unmodified ground, differential liquefaction induced settlements were predicted in the order of 100mm to 190mm under SLS and ULS loads respectively. Both buildings are supported on steel compression-only screw-piles founded in the "Riccarton" gravel layer at approximately 23m depth below existing ground to mitigate expected settlement and provide assurance to the insurers. Reliance on base-friction and passive resistance to resist lateral loads was considered appropriate following the slab subgrade improvement and ground beams installed to interconnect the pile caps.

In accordance with damage avoidance principles it was decided that a suspended ground floor slab supported on concrete finger beams and piles would be required to avoid issues with settlement due to liquefaction. The ground floor slab is 150mm thick and doubly reinforced (and not sawcut), to allow it to span between beams supported on piles. The intent is that following a ULS event, any void between the ground beneath the slab due to settlement be grouted again to provide friction resistance to take out some seismic base-shear.

## **Reasons for Selection of EBF's with Replaceable-Links as a Structural System**

Early on in the project, the client was looking for innovative approaches to the design of the structure. The main reasons for the selection of the replaceable-link EBF's include:

- Proven track record of existing EBF's in Christchurch and perceived performance of steel as a "better" performer compared to concrete.
- Improved insurance and bank finance approval for a system which is easier to repair.
- Potential marketing opportunities for tenants by utilizing new seismic technology in current climate.
- Reduced downtime associated with repairs following large seismic events.
- Bolted detailing offering ease of constructability and time of erection. Also, large demand for office space in post-earthquake Christchurch due to diminished building stock means erection time is critical for optimizing rents and capitalizing on demand. Longer lead-time for concrete means steel offered faster erection times.
- Steel construction meant lower overall mass of structure and slightly decreased seismic load.

The concept of a replaceable-link EBF system was discussed between engineer, architect and client very early on in concept design.

### **Design Parameters and Methods**

The main design guidance for the replaceable-link EBF system was a combination of existing design guidance for conventional EBF's and other publications and advice. Specifically:

- NZS3404:1997.
- HERA report R4-76 Seismic Design Procedures for Steel Structures (Feeney and Clifton, 1995).

For design of the link elements and for baseplate design, there was limited design guidance available, and the design philosophies were undertaken in conjunction with Steel Construction New Zealand (SCNZ) and the existing research from the University of Toronto.

SCNZ in conjunction with HERA and Associate professor Charles Clifton from the University of Auckland are currently updating HERA report R4-76 Seismic Design of Steel Structures (Feeney and Clifton, 1995) to include replaceable-links, partially based on experience with this project.

SCNZ advised that baseplates should include shear-keys to transfer shear from EBF's into foundations instead of hold-down bolts. The design guidance for this was from the AISC publication "Base-Plate and Anchor Rod Design" (Fisher and Kloiber, 2010).

In addition, connections were designed to NZS3404:1997 (New Zealand Standard, 1997), SCNZ "Steel Advisor" publications (Cowie, 2008-2009), and the British Steel Construction Institute 1995 Publication SCI Publication Joints in Steel Construction - Moment Connections (The Steel Construction Institute, 1995).

Associate Professor Charles Clifton of the University of Auckland Civil Engineering School has also had input into the design support provided through SCNZ.

Following the Canterbury Earthquake sequence, much information has been available and put out in rapid succession. In accordance with best post-earthquake design practice the design of Three35 project includes the design engineers experience based on post-earthquake reconnaissance in detailing connections of secondary elements to primary, and recommendations from the Canterbury Earthquakes Royal Commission of Enquiry Interim Report (Canterbury Earthquakes Royal Commission, 2011).

## Seismic Analysis

Due to the relatively regular plan layout and low rise nature of the building, the building was analyzed through hand calculations and static elastic design software. An equivalent static analysis was conducted using the NZS1170.5 Soil Category D spectrum, and due to the lightweight nature of the roof, the additional 8 percent of the base shear was shared between roof and level 2 (i.e. an additional 4 percent base shear at roof and at level 2).

The seismic force distribution was done using a simple Microstran model of one level with a rigid truss model for floor diaphragms, and actual concept sized steel EBF members for lateral load resistance. The model considered accidental eccentricity of +/-0.1B as prescribed in NZS1170. Allowing for a small amount of redistribution, it was seen that the demand on each frame was approximately 33 percent of the total seismic shear.

The EBF shear-links were designed for a displacement ductility of 3.0 (category 2 EBF's), and all other lateral load resisting structure and associated connections were designed for the over-strength of the yielding shear-links.

Table 1. Design information for typical Three35 office building.

Limit State	ULS	SLS
Building Weight, $W_t$	14,363 kN	
Period (1 <sup>st</sup> Translational. Mode), T	Assumed to be approximately 0.40 seconds	
Seismic Load Coefficient, $C_D$	0.29 ( $\mu=3.0$ )	0.18
Building Base Shear Force, $V_b$	4,223kN ( $\mu=3.0$ )	2,585kN

Static linear design of building components was carried out using Microstran Version 8. Overall approximate building deflections under seismic loads, as governed by the steel EBF's were calculated as follows:

Table 2. Overall building displacements.

	Total expected SLS displacement (mm)	Total expected ULS displacement (mm)
L1	15	35 (1%)
L2	30	70 (1%)
L3	45	105 (1%)

### Replaceable-Link EBF Design Details and Challenges

The challenges involved with the design of the replaceable-link EBF's were similar to those faced in regular EBF design, but with the following additional challenges:

- The damage avoidance premise offered by the replaceable-link system must flow into other items. This meant that the ground floor and foundations were designed as suspended elements on piles with the effect that larger pile loads were involved, and crack control of the ground floor slab was difficult due to not saw-cutting to maximize and control slab spans.
- Also included in the damage avoidance premise was allowing the precast concrete wall panels (required for fire-rating at boundaries) to undergo similar drifts to the frames. This required careful detailing of the base and floor connections to allow for rotation. In a similar fashion, stair connections were carefully thought out to accommodate the drifts as per the Interim Report from the Royal Commission into the Canterbury Earthquakes (Canterbury Earthquakes Royal Commission, 2011).
- The Architect initially wanted to use circular hollow sections (CHS) for the EBF columns. This was briefly looked into using HERA CON1002 (Cowie, 2008), but given the rapid design and construction timeframe on the project, the introduction of another level of complication proved difficult without adequate testing and design guidance, especially for the D-Braced EBF configuration, and the idea was shelved.
- The Architect's layout was uncompromising when it came to building set-out and configuration of the steel frames. In the end four different frame configurations including two types of "D-braced" configured frames and two types of chevron or "K braced" configured frames had to be modeled, resulting in a large spreadsheet for analysis.

- The use of standard HERA connections (Hyland et al., 2010) was not applicable to the link endplate connections due to the large link sizes and configurations required. Similarly, the endplate design in the SCI publication Joints in Steel Construction – Moment Connections (the derivative document for the HERA connections), (The Steel Construction Institute, 1995) were only applicable for a limited selection of bolt configurations. To avoid an unsatisfactory yield mechanism, this meant that a model of the specific endplate configuration had to be set up using a grillage in the static analysis software Microstran. Gap elements were then introduced to allow for plates butting up to each other which could also move apart as required. This modeling was carried out using input from SCNZ. Refer to Fig. 5. With three different link endplate configurations and four different EBF types, this resulted in a significant amount of modeling of individual elements. Bolt prying loads from this analysis from this model matched those recommended by SCNZ. The designer must ensure all load conditions are taken account of and must include any axial tension from the analysis into the shear-link endplate bolt design.

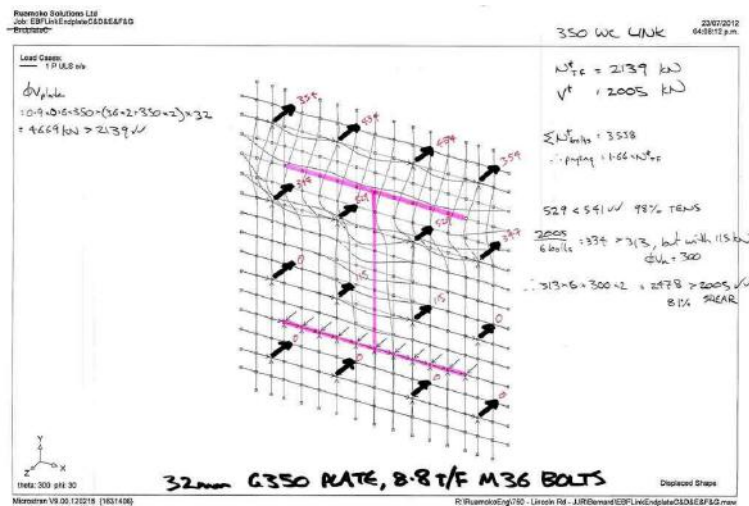


Figure 5. Link endplate modeling using Microstran.

- During the construction monitoring phase of the project, it was suggested By Charles Clifton from the University of Auckland that hold-down bolts for EBF baseplates be designed for tension only and that that a shear key be provided to retain comparable lateral stiffness of the tension column and compression column bases throughout an earthquake. There is a tendency for more shear to be transferred through the compression column side and potentially overload the column and brace and potentially disrupt the assumed transfer of design actions through the EBF system. The shear key was designed using a modification to the AISC publication, “Base Plate and Anchor Rod Design” (Fisher and Kloiber, 2010). This required significant design effort, as the loads in the Three35 project were significantly larger than those used in the case studies in that publication. The introduction of the shear key following issue of construction drawings had a significant effect on the foundation design, especially as the foundations were already on the critical path and being constructed.

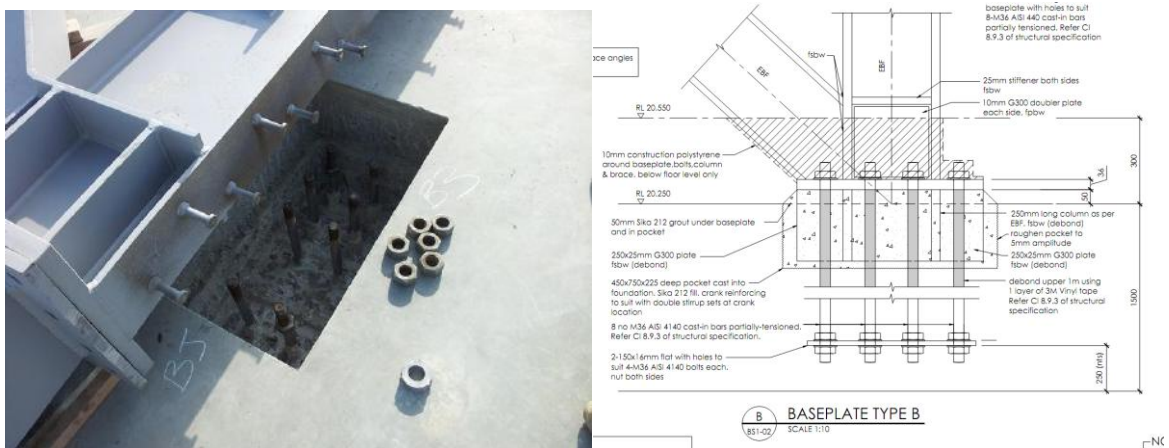


Figure 6. Shear key and details of hold-down bolts.





- Due to the replaceable-links being a new concept in New Zealand, for financial purposes it was necessary to meet with the insurance company to explain the technology. This is likely to be common practice for damage avoidance design in the region in the future.
- To satisfy the territorial authority that the design was appropriate for building consent it was necessary for the design to be peer-reviewed. This was carried out by BCD group in Hamilton, New Zealand. With new seismic resisting technology it is important that designs be peer-reviewed by another independent engineer. Engineers must allow for the extra time required for discussions and peer-review queries. Also, with territorial authorities now demanding a Producer Statement – Design Review (PS2) at the time of consent application, it is imperative that the construction programme allows for this.

### **Construction Feedback and Challenges**

As with all projects, collaboration between engineer, contractor and project team is essential to the success or otherwise of a project. This is especially true with emerging technologies such as replaceable-link EBF's. As new challenges emerge for designers, some of the challenges of new technology are even greater for contractors. Some of the challenges encountered on the Three35 project are outlined below:

- The tight design programme on the project meant that procurement of materials was on the critical path. The replaceable-link system called for some materials that are non-standard to Christchurch steel fabricators (such as thick endplates that satisfy the material requirements of NZS3404:1997, AISI4140 bars, large grade 8.8 bolts and members with large section sizes). In retrospect, it would have been more efficient to have known what the materials the steel fabricator could source before completing the design. Columns for D-braced EBF's were non-standard and were originally specified as 310UC198, however despite initial discussion with steel fabricator saying they could source this item, they had to import some members from Japan which required some lead time and a process of inspection of various mill certificates to ensure compliance.
- Limited overseas steel stocks meant that some column sections had to be fabricated by Steltech in Auckland. For this project it was not economic for Steltech to fabricate sections with butt-welded web to flanges, and as such some fillet welds needed to be ground and butt welded locally where required (for example adjacent to active link areas).
- The steel fabricator advised that there were no grade 8.8 hold-down rods in New Zealand with the length specified on the Three35 project, and there was significant lead-time associated with procurement of enough grade 8.8 bolts for the replaceable-link endplates. As such, AISI 4140 bolts were specifically milled, threaded, galvanized (where applicable) and tested for the hold-down rods. Erection bolts were used in place of grade 8.8 bolts to replaceable-link endplates until the grade 8.8 bolts were on site. This resulted in significant double-handling.
- In all projects the location of the hold-down bolts is critical. This is further complicated on this project by shear-key requirements. Despite the contractors best efforts to cast bolts in the correct position some were out of alignment and holes had to be carefully enlarged in accordance with requirements of NZS3404:1997. In hindsight use of a steel template or a detail which allows for some bolt movement may have been prudent. In one instance the bolts were a phase out meaning that two were completely off the endplate. In this situation it was fortunate in that circumstance that the endplate could be bridged to a return EBF.
- The design and layout of some elements changed once the steel fabricator was appointed due to that fabricators preferred weld methodologies, maximum workshop and trucking sizes etc. This is to be expected to some degree in most structural steelwork projects, but is even more applicable where significant sizes and welds are required such as in the projects incorporating replaceable-link EBFs.
- Due to the large volume of steelwork on this job, the local steel fabricator subcontracted some welding to a North Island fabricator. The steel inspected from that fabricator contained a number of unacceptable welding defects which resulted in significant grinding and re-welding re-work. Sections of the structure had already been erected and the re-work of these sections had to be carried out on site. In part these issues were due to different quality assurance procedures in the standards. AS/NZS1554.1:2011 calls up ultrasonic examination to comply with AS2207 (the standard that specifies the ultrasonic examination requirements). Appendix C of AS2207 is informative, and gives guidance on factors that influence probe selection. AS/NZS1554.1 however also states ultrasonic

inspection requirements, but allows for variations to the equipment requirements providing such variations are in agreement with the principal. To meet and resolve the varying requirements, these issues must be discussed with the fabricators and compliance inspector prior to any fabrication to ensure there is agreement between all parties involved.

- Experience on this project found that extensive non-destructive weld testing (NDT) and quality assurance measures are required for a greater proportion of critical welds compared to a conventional EBF. It is imperative that an independent compliance inspector be engaged on a project involving replaceable-link EBF's.
- Despite deliberately including a specification item precluding the use of leveling nuts under EBF baseplates, on several frames leveling nuts were installed immediately below the baseplate which therefore meant the part-turn method of pre-tensioning the hold-down bolts to NZS3404:1997 could not be achieved. This was discussed with SCNZ, and it was decided that the remedial for this would have been worse than the status quo. While not a safety issue, this had to be explained to the client, who accepted more SLS movement than could have been achieved with pre-tensioning.

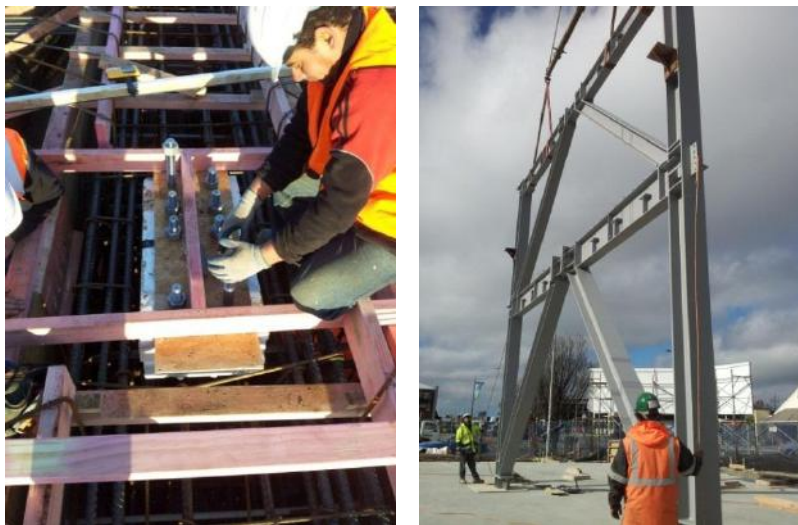


Figure 8. Construction photos of EBF baseplate installation and erection of a K-braced EBF.

### **Recommendations/Conclusions**

Experience on the Three35 project shows that the replaceable-link EBF system is a viable option that is attractive to developers and able to be constructed in the local market using damage avoidance principles, but is not without its challenges. Although the replaceable-link concept is simple in theory, experience shows that there are some “teething” difficulties involved with introducing this type of new system into the market.

The conventional EBF system has been a proven performer during the recent Canterbury Earthquake sequence. Based on the testing and real experience with the replaceable-link EBF system on the Three35 project, with some more established design guidelines and industry awareness, this system has the potential to become a leading seismic structural system for future projects. There is no reason that that this replaceable-link EBF systems can not only compete but also exceed that of conventional EBF systems on a performance and cost basis.

The following are recommendations and conclusions based on the experience on the Three35 project:

- Designers should be aware that until the system becomes more commonplace, there is likely to be extra detailed design effort for a bolted-link EBF system compared to that for a conventional EBF design. Allowances should be made by the designer to discuss peer-review items required by territorial authorities. A collaborative relationship between engineer, contractor, and steel fabricator is likely to be required more intensively until the technology becomes more mainstream. Designers should take account of these items in their project fees.

- As exemplified by this project the importance of independent non-destructive weld testing and sound quality assurance procedures for structural steelwork compliance cannot be overstated. There also needs to be sound quality assurance procedures for bolted connections that are tensioned to the /TB condition. It appears that the NZ standard for weld testing NZS1554.1:2011 needs to be aligned with the Australia standard it cites (AS2207).
- The nature of the bolted shear-links is suited to full height erection of half frames for buildings up to five stories. K-braced EBF's are likely easier to transport than D-braced EBF's as they are about half as high during transportation.
- Standardization of frame sizes and members is beneficial for both designers and contractors. In addition, replacement of bolted links after a ULS event is likely to be more efficient if EBF's incorporate more standardized section sizes and link lengths.
- Having standardized connections to link endplates would be hugely beneficial for designers. This would likely require standard link and collector beam sizes which could be standard solutions which may become familiar to designers and contractors alike in the same way the HERA connections have (Hyland, Cowie, and Bird, 2010).
- Currently there are no formal requirements to check global flexural continuity of EBF columns in a building. Designers should be aware that a pin (such as in a machined bearing splice) in an EBF column may in theory lead to a potential soft-storey mechanism. This may need to be addressed in future EBF design guidelines, especially for taller buildings where the stiffness varies with building height and there are no "strongback" elements such as concrete panels.
- The industry needs more guidance on the design of shear keys for EBF baseplates. AISC guidance (Fisher et al., 2010) is generally suitable only for small loads. In addition, it would be useful to have standardized hold-down bolt solutions that provide some tolerance for bolts cast slightly out of position. Some of these options are described in ERC1004 (Cowie, 2009) but solutions involving solvents and polystyrene may be hard to control in practice and if not carried out correctly by contractors could be risky.
- For the RIM/damage avoidance principles to be valid, some base rotation is required. More design guidance on achieving base rotation to baseplates and shear keys would be useful to designers.
- The Three35 project demonstrates that the replaceable-link EBF systems result in specification of the upper limit of hot-rolled section sizes. Designers should attempt to ensure fabricators can either source these sections or can fabricate the sections before specifying. This saves design time and requires close collaboration between fabricator and engineer. Care should be taken when specifying fabricated sections that plate can be sourced, that appropriate factors as per table 6.3.3(1) NZS3404:1997 are used to obtain the  $\alpha_c$  factor for compression depending on whether sections are fabricated from flame cut or as-rolled plate. Designers should be aware that for fabricated sections that fillet welding of webs to flanges is generally standard practice, Should butt-welding be required, it should be checked that such sections can be fabricated in order to avoid delays due to rechecking etc.
- Where possible, a K-braced configuration is generally more efficient than a D-braced configuration to reduce column sizing and to reduce collector beam combined actions.
- When considering the use of replaceable-links, designers should make sure the premise of RIM is tracked through the whole job. This may increase the detailing of movement tolerances between secondary and primary elements to ensure that minimal repair work is required for re-occupancy. For replaceable-link EBF systems, this may result in a cost premium greater than just the increased cost of detailing of the links themselves. For example, there is little point in incorporating replaceable-link detailing when the building suffers too much ground settlement due to seismic liquefaction at ULS to be economically repaired and in such circumstances designers may consider using a suspended ground floor. Designing in this fashion requires increased collaboration with architects, ceiling designers etc. Designers also need to be careful to reduce yielding in all overstrength elements, for example collector beam endplates to allow new link to be fitted.

- If at all possible it is most efficient for the client/contractor to appoint the steel fabricator as early as possible in the project (preferably in the design phase) to reduce design time involved with changing material and section sizes. Member sizing of EBF's is in some ways an iterative process and fundamental decisions are made with regards to overstrength factors early on in the design phase. The sooner these decisions can be resolved the quicker and more efficient the design phase will be.
- Experience on the Three35 project has demonstrated that it is likely that until standardized solutions infiltrate the industry that there will be some cost premiums associated with using the replaceable-link EBF system compared with existing conventional EBF systems. Some of this cost is due to RIM detailing around the project. It is likely however, that in time these costs will offset the dis-benefits offered by conventional EBF systems, and prove a more economical system.

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