PROPOSED DEVELOPMENT OF A DAMAGE-RESISTING ECCENTRICALLY BRACED FRAME WITH ROTATIONAL ACTIVE LINKS.

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SUMMARY

Eccentrically Braced Frames (EBFs) are widely used seismic-resisting systems, as they allow both strength and stiffness to be optimised while providing good ductility capacity. However, in theory they have a low damage threshold in severe earthquakes and post-earthquake repair of conventional EBFs will be difficult and expensive.

This paper presents the Numerical Integration Time-History (NITH) analysis of two ten storey EBF buildings; one with a conventional active link and the other with a new form of low damage active link based on rotational sliding bolted plates. The low damage active link can be designed to allow rotation only, or to allow both rotation and axial extension. The conventional active link response in terms of displacement, rotation and inelastic demand was well within the range of the rotational active links under the records considered. The analysis shows that average maximum displacement of the building and rotation of the link for both the rotational and the rotational+extension active links was almost identical. The extension of the rotational active link permitting axial extension was less than 1.5 mm. Axial load demands on the collector beams and braces were similar in the case all three active links.

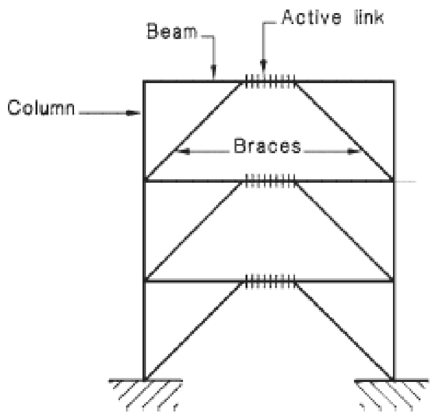
It can be concluded from the analysis that the rotational active link with extension is not required, as the lateral extensions can be accommodated within the rotational plates with nominal clearances in the bolt holes to accommodate the lateral extension

INTRODUCTION

The EBF is a steel-framed earthquake resisting system for multi-storey buildings which are designed for the highest level of ductile demand.

It is one of the most widely used multi-storey steel seismic-resisting systems in New Zealand and many other first world seismically active countries, as it allows for both strength and stiffness to be optimised in a system that delivers dependable inelastic response and takes up little building useable space in plan or elevation. The seismic-resisting system is used in both new building construction of steel and concrete frames and in retrofit of existing concrete and steel framed buildings.

The principal components of an EBF are shown in Figure 1a. These comprise the active links, beams (also known as collector beams), braces and columns. The EBF is designed for ductile response to severe earthquakes, i.e. where the frame will undergo the maximum expected extent of controlled damage in a severe earthquake, which is permitted by the Loading standard, NZS1170.5.



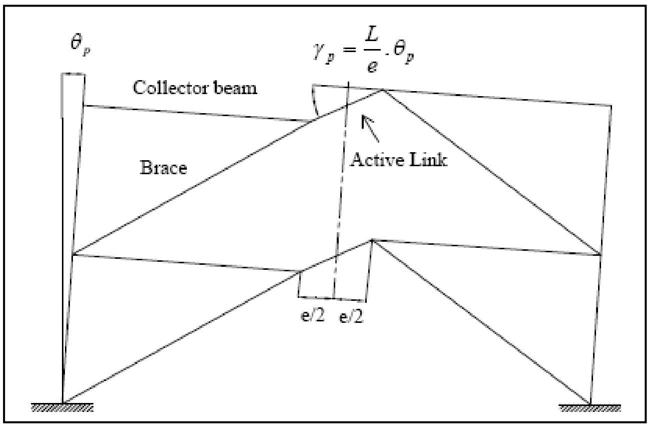


Figure 1: ***Member terminology for: (a) a V-braced eccentrically braced frame and (b) inelastic behaviour of a V braced system (NZS 3404 1997/2001/2007).***

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This controlled damage is designed and detailed to be contained principally in the active links, which will undergo considerable plastic deformation in a severe earthquake. The idealised pattern of inelastic demand on the EBF system is shown in Figure 1b. Until recently, the design philosophy for building response to severe earthquakes focussed only on ensuring that the building did not collapse so that occupants could escape after the earthquake. The building would then be assessed for repair or replacement

However, due to the high time and financial cost of business interruption and building repair, the emphasis is now shifting to development of seismic-resisting systems that can be rapidly returned to service following a severe earthquake. This poses potential challenges to the EBF because of the very large local inelastic demand expected in the active links in a severe earthquake and the low damage threshold at which this damage will commence.

A research programme is therefore being undertaken to develop a damage-resisting eccentrically braced frame with rotational sliding active links. This is a significant new concept which is intended to address the "Achilles' heel" of traditional eccentrically braced frame (EBF) systems, i.e. their low damage threshold and difficulty of repair following a severe earthquake. The work is being conducted in two parts, the first is an analytical study, and the second is an experimental study. This paper presents the results of the analytical study.

PERFORMANCE OF EXISTING EBF SYSTEMS

According to the NZ code, and many other worldwide codes, EBFs are designed for the highest level of ductility demand, or the lowest level of design force, of any structural steel seismic-resisting systems, because of their desirable combination of elastic stiffness and good inelastic performance. The inelastic demand is concentrated into the active links, which dissipate energy by shear yielding and bending. The length of an active link dominates the behaviour of the energy dissipation. The active links are categorised by (NZS3404 1997/2001/2007) into three different lengths. Short links designed and details in accordance with NZS 3404 undergo principally shear yielding with limited flexural action. Long links undergo principally flexural yielding at the ends of the link, with this inelastic demand concentrated into a plastic hinge zone at the link ends, similar to beam end yielding in a rigid moment-resisting frame. Intermediate length links vary in behaviour between these two regimes depending on the link length.

Because short links yield in shear and this demand is reasonably evenly spread over the length of the link, they are allowed the highest inelastic rotation angle by NZS 3404, being 0.08 radians. Long links are allowed only 0.03 radians, being the same as for plastic hinges in beams, and their behaviour is similar to that of a beam in a moment-resisting frame with a flexural plastic hinge forming at each end of the active link and minimal inelastic demand in the middle of the long link.

The failure mode of active links is dominated by web fracture following extensive web buckling due to the shear or flexural yielding as explained in previous paragraph. Long links can also undergo flange fracture due to high flexural demand at the ends of the links, as shown in Figure 2. Most of the recent research work on EBFs has focussed on achieving higher dependable inelastic capacity from the active link, for example (Okazaki & Engelhardt 2007). However, one of the prominent disadvantages of this current active link design is the need for post earthquake replacement of distorted active links. Because of the concentrated inelastic demand in them, replacement is in principle required at a relatively low threshold of ground motion and replacement is very costly and physically difficult to do. Mansour et al (2009) conducted experimental investigations on replaceable active links; the design of the replaceable active link includes welding the link to the end plate which is then bolted to the collector beam, and trimming one side of the link's flange which is also connected to the collector beam eccentrically. The test results demonstrate that the performance of replaceable active link is identical to the conventional active link, however replacement of the link is still required.

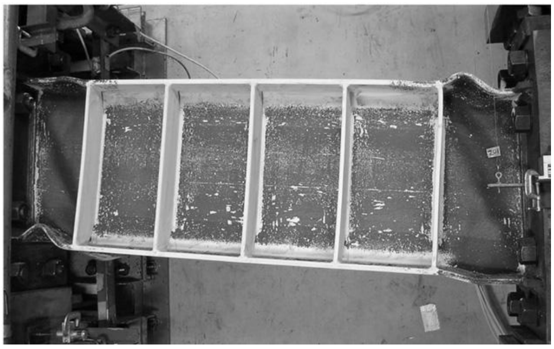

Figure 2: Web fracture and buckling of a long active link (Okazaki & Engelhardt 2007).

This proposed research aims to produce a damage resistant EBF system, which can be easily and quickly restored to service following a severe earthquake, without the need to replace the active link component.

PROPOSED ROTATIONAL ACTIVE LINKS OF THE NEW EBF CONCEPT

The key to this damage-resisting system is an unique form of active link, as shown in Figure 3. The behaviour is fundamentally different from the conventional active links. It does not involve inelastic shear or bending action in the active link. Instead, the "inelastic" demand on the active link is resisted by stable rotational sliding of a circular group of bolts on each side of the active link. The concept of the rotational active link is derived from combination of sliding hinge joint and slotted holes energy dissipater developed by (Clifton 2005), (MacRae *et al* 2010) and (Lai 2000) . The components and seismic energy dissipation of rotational active link are described as follows:

1. The proposed rotational active link comes in a form of two active link stubs which extend across the active link region, from a conventional brace and collector beam intersection. There is a clear gap between the active link stubs, this clear gap provides accommodation for the inelastic displacements of the active link stubs.

2. The web plates and wear resistant steel shims (which might be combined into one component each side) extend across the active link region on both sides of the active link stubs, as shown in Figure 3. The web of each active link stub has slotted holes in a circular arrangement in conjunction with a nominal hole in middle of the circular pattern of slotted holes. The active link steel plates will have slotted holes in a horizontal arrangement that align with the slotted holes in web and in which the bolts are installed in the centre of the slotted holes. The steel plates in parallel are connected by cap plates with fully tensioned HSFG 8.8 bolts, with a larger size bolt for the centre of slotted hole. This connection generates clamping force between the web and the steel plate, generating the shear resistance required in the active link through development of rotational moment resistance in each set of circular slotted holes. Note that Figure 3 shows brass shims, based on the work of (Clifton 2005).

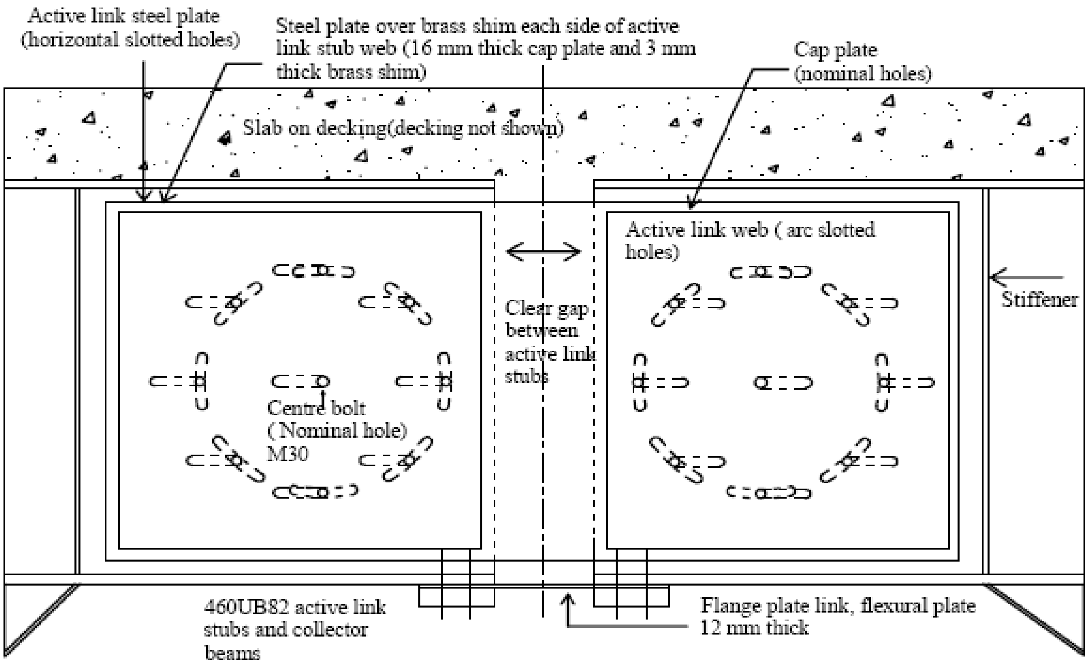

Figure 3: The rotational active link concept for an EBF system.

Note: The active link plates and cap plates are *not* shown in this figure.

However, more recent testing on the sliding hinge joint by Hsen Han at University of Auckland (Khoo *et al* 2011), has established that wear resisting steel shims produce the same performance as brass shims and these will be used in this programme, as they will make construction easier given that they can be tack welded into position before erection.

3. The "inelastic response" mechanism of this rotational active link is relatively simple. As the EBF system moves laterally when subjected to an earthquake motion, the cap plates are forced into rotational sliding relative to the active link stubs about the central bolts. The clamped resistance to this rotational sliding generates the earthquake resistance across the active link of the EBF system.

4. Assessment of the feasibility of this concept and the rotational demands on the circular slotted holes were made by rigid body rotational studies of the EBF sub assemblages over a typical storey. This was undertaken using AUTOCAD. The results are shown in Figure 4, in which the two sub assemblages have been rotated to the maximum NZS 1170.5 allowed interstorey drift angle of 0.025 radians, with this rotation centred on the intersection of the brace and column at the storey below the active link, as would be the case in practice. They show that, in addition to the rotation, the active link must undergo an axial extension from at the initial at-rest position to the maximum rotated position, in this instance from an initial distance of 992 mm to a fully extended distance of 1029 mm.

- None of the previously reported experimental tests have accounted for this axial extension as they have been on statically determinate subassemblages incorporating the active link, in which the horizontal distance between the two supporting columns has been able to reduce as the active link deforms. In practice, however, the two columns remain a fixed distance apart at the active link level due to the continuous floor slab and surrounding structure, as shown in Figure 4. An allowance for this longitudinal extension across the active link is made through horizontal slotted holes in the active link plates; as shown in Figure 3.

- As with the Sliding Hinge Joint (Clifton 2005) a cap plate with nominal sized holes sits over the active link plates on each side. If brass shims are to be used, they will be sandwiched between each of the cap plate, active link plate and active link web stub to facilitate stable sliding. If steel sliding surfaces are shown to be suitable then the shims can be omitted and the active link plate becomes the effective shim.

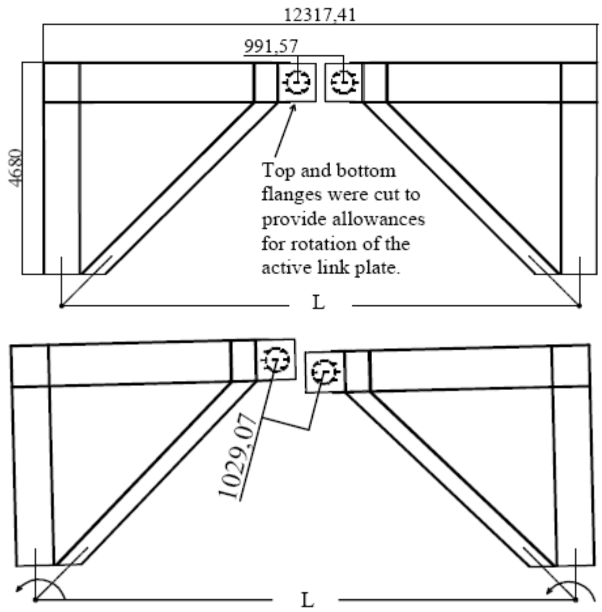

Figure 4: Rigid body rotational study of EBF subassemblage over a typical storey; (a) at 0 rad inter-storey drift and (b) after inter-storey drift limit of 0.025 rad.

5. The two bottom flanges of the active link stubs are connected with a flange plate link which will hinge at each end of the active link stub across the clear gap and will if necessary be provided with slotted holes to accommodate the longitudinal movement between the two active link stubs. Similarly, the top flanges of the stubs are connected with a deck support plate which provides a support surface and continuity for decking spanning onto the EBF beam/link. Under the earthquake motion, the deck support plate is forced into hinging at each end of the clear gap and ensures that the active link stubs do not punch through the slab.

This concept offers an active link which is easily able to develop greater than current design rotation limit of 0.08 radians with no components becoming inelastic other than the bolts, which are forced into the inelastic range when fully tensioned on installation and are engineered to perform in this state for the lifetime of the building, and the flange plate link flexural plates that can be replaced at leisure. Preliminary calculations on a 10 storey frame designed for compliance with NZS 3404 ((Patel, C and Lal, M 2010) have shown that the required shear capacity can be developed by realistic combinations of active link plates and bolts. However, the system as shown in Figure 3 is not an actively self centering system as there is no active restoring force to bring the active link back into a level position at the end of the earthquake, but it is more self-centring than conventional active link because of the change in hysteresis loop.

FEASABILITY STUDY OF ROTATIONAL ACTIVE LINK AND CONVENTIONAL ACTIVE LINK

In order to investigate the inelastic behaviour of proposed rotational active links and conventional active links in a building, non linear time history analysis of a V-braced system of ten storey frame was performed using the specialised earthquake analysis program RUAUMOKO. The design of conventional EBF and rotational EBF are based on an EBF developed by (Patel, C. and Lal, M.2010) and shown in Figure 5a and Table 2. The design of the building was in accordance with the HERA Seismic Design Procedure for Steel Structures, HERA Report R4-76 (Feeney and Clifton, 1995/2000), Steel Structures Standard (NZS 3404) and Loadings Standard (AS/NZS 1170), and based on the drift requirement.

Table 1: The permanent loads and imposed loads applied in design of ten storey buildings.

Load	Pressure(kPa)
Structural Dead load	0.5
Permanent load	3.15
Cladding	1
Imposed Load	3
Imposed Load (roof)	0.25
Self Weight	0.6

(a)

(b)

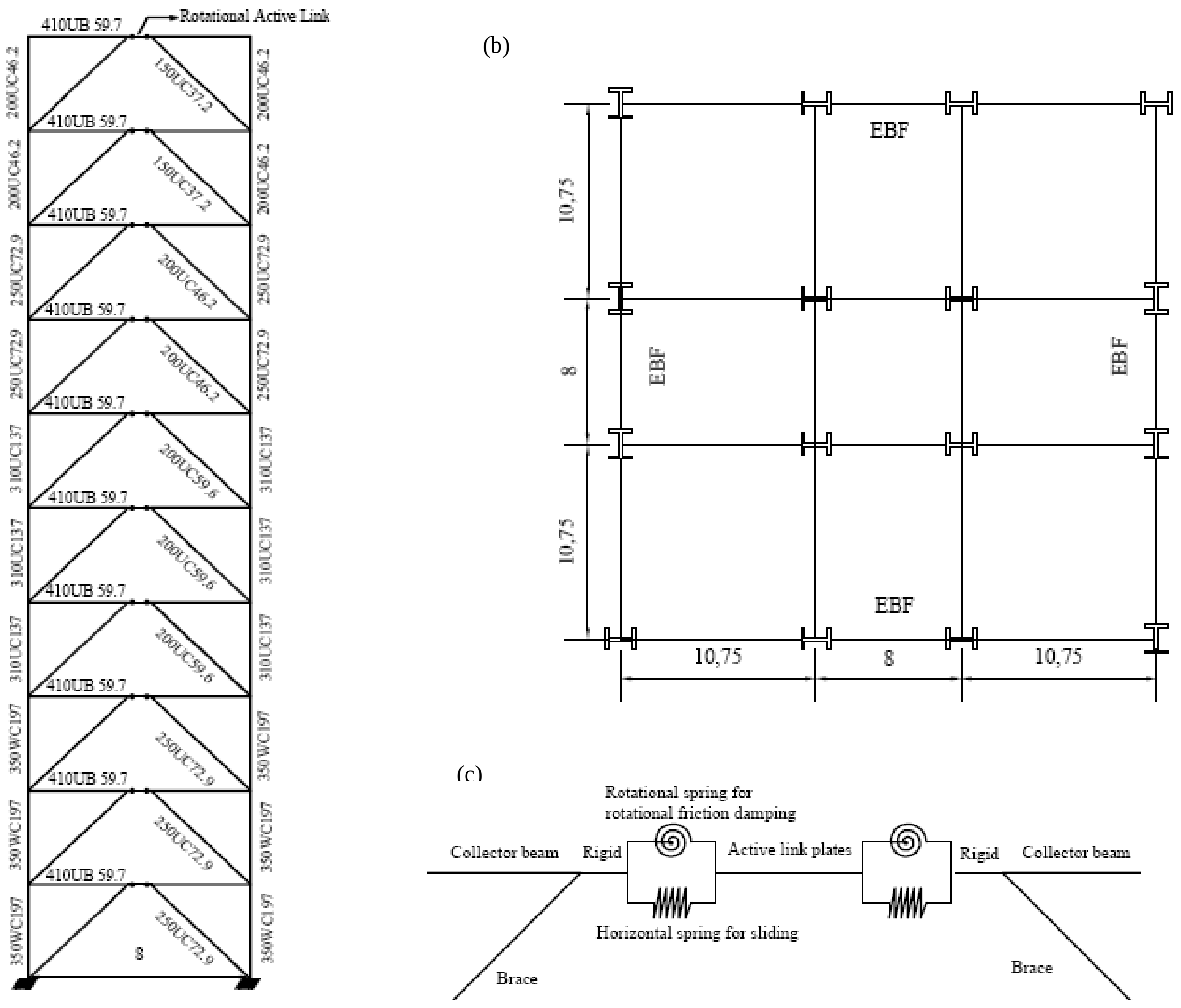

Figure 5: The design of rotational active link with ten storey building: (a) Elevation of the EBF, (b) Plan view of the ten storey building, (c) Model of the rotational active link with extension.

The principal EBF design parameters are presented below:

1. The ten storey building is located in Wellington, the ground soil is Type C, natural period is 1.96 sec, which considers foundation effect approximately, based on NZ3404 upper limit stiffness recommended for fixed based columns, and annual probability of exceedence is 1/500.

2. The loads used in preliminary design are shown in Table 1.

3. The dimensions and the member sizes are demonstrated in Figure 5a and Table 2.

4. The plan layout of the ten storey building is shown in Figure 5b.

Table 2: The design of ten storey buildings with conventional EBF.

<table>
<tr><th>Storey Level</th><th>Active Link</th><th>Collector beam</th><th>Storey Level</th><th>Brace</th></tr>
<tr><td rowspan="2">8--10</td><td rowspan="2">200UC60</td><td rowspan="2">200UC60</td><td>9--10</td><td>200UC46.2</td></tr>
<tr><td rowspan="2">7--8</td><td rowspan="2">200UC60</td></tr>
<tr><td rowspan="2">5---7</td><td rowspan="2">250UC73</td><td rowspan="2">250UC73</td></tr>
<tr><td>4--6</td><td>250UC73</td></tr>
<tr><td>1--4</td><td>250UC89.5</td><td>250UC89.5</td><td>1--3</td><td>310UC96.8</td></tr>
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Table 3: Moment and shear capacity of the rotational active link.

No of storey	V^*_{Link} (kN)	M^*_{Link} (kNm)	No of Bolts	ϕVss (kN)	ϕMn (kNm)
1--3	423	105.8	4 x M36	256	153.6
4--6	389	96.9	5 x M30	175	87
7--8	69	69	4 x M30	175	70
9--10	41.5	25	4 x M24	68	43.8

THE DESIGN OF THE CONVENTIONAL AND ROTATIONAL ACTIVE LINK

The design of the conventional active link was in accordance with NZS 3404 Section 12-11 and HERA Report R4-76 section 11.

In order to estimate the clamping force of the rotational active link, the sliding shear capacity of the bolts from (Clifton, 2005) was used to determine design capacities. Given the symmetric sliding on the bolt in this application, the bolt sliding shear capacity should be greater. The rotational active links designed to accommodate the rotation and horizontal extension of the link through slotted holes. The capacity of the active link is expressed as rotational moment which develops in the active link region when subjected to an earthquake load and hence generates the shear resistance. The design procedure for the rotational active link is briefly explained as follows:

The rotational moment action and the rotational moment capacity are expressed as:
$M^* = V_{link}e/2$ and $\phi M_n = \phi V_{ss}$* no of bolts*e

where:

e = length between the two centre bolts of the rotational active link
ϕV_{ss} = shear sliding capacity of the bolts.

The designed bolts and their capacities are shown in Table 3.

Allowance for the sliding shear capacity of the bolts in the longitudinal slotted holes was made in a similar manner

ANALYSIS OF THE ROTATIONAL ACTIVE LINK

Using the ten storey building, as shown in Figure 5, nonlinear time history analyses were conducted using RUAUMOKO (Carr, A. J., 2010).

The ground motion records for analysis selected from recommended time history records, based on the studies conducted by (Oyarzo-Vera, 2011), which have been selected and scaled for time history analysis in the North Island of New Zealand, Wellington, and the return period for the scaled records is 1/500.

There were three categories of analyses undertaken: this includes rotational active links with rotation only, rotation and sliding, and conventional active link matched closely with rotational active link.

In RUAUMOKO, the rotational active link was modelled as two springs, a horizontal and a rotational spring, which act in parallel to simulate the characteristics behaviour of rotational active link as shown in Figure 5c. Note that horizontal spring was used to simulate the extension of the link through horizontal slotted holes as shown in Figure 3. The hysteresis loop or characteristics of the rotational and the horizontal springs was based on experimental work of sliding hinge joint, conducted by (Clifton, 2005). One of the advantages of the hysteresis loop of sliding hinge joint is the pinched hysteresis loop, which leads to lower response and lower permanent displacement. This effect can be demonstrated using analytical analysis of rotational active link shown in Figure 11, where it shows response of rotational active link is lower and inelastic cycles concentrated around the centre as compared to conventional active link, which demonstrates that the self-centring ability of the rotational active exceeds the conventional active link. Rayleigh damping was applied with 5% specified for the 1st and 10th modes.

Because, the investigation aimed at comparing the performance of conventional active links and rotational active links using a ten storey building, therefore wherever possible the members of the conventional and rotational active link systems were made the same size. Such as, the collector beams for the rotational active link designed to have identical members (410 UB 59.7) for all levels as opposed to the conventional active link as shown in Figure 5a and table 2. The distribution of the rotational active links strength differs slightly from the conventional active link along the levels of the building, because it is convenient to change the strength of the rotational active link just by manipulation the number bolts in the link as shown in Table 2 and 3. The result shows that the axial load on collector beams and braces is approximately identical in the case of both conventional and rotational active link as shown in Figure 10.

Other prominent difference is length of these two different active links. The length of the conventional active link is 800 mm and the length between centres of the rotation of the rotational active link is 500 mm, therefore the rotational demand for rotational active link will be greater than the plastic rotation angle of a conventional active link. This difference can be explained with reference to the model of the rotational active link in Figure 5(c). In the conventional active link, the rigid member to the left of the left active link shown in that figure would be absent; similarly on the right hand side. Each rigid member has a length of 150 mm and represents the distance from the stiffener face to the centre pin of the bolt group.

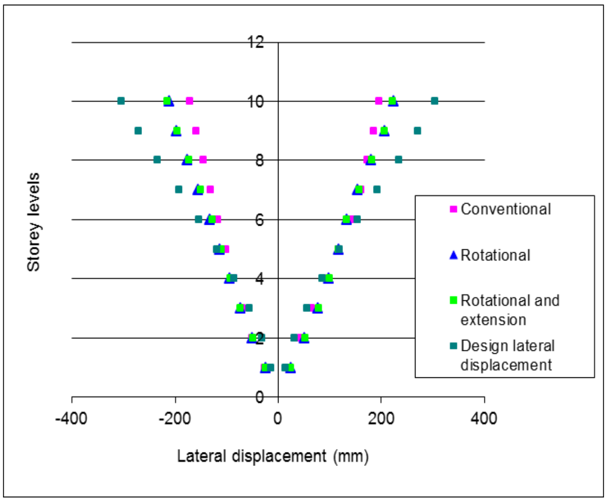


Figure 6: ***Maximum positive and negative lateral displacement of each level of the building.***

The analysis shows that the maximum average horizontal displacement of the ten storey building with three different types of link is similar as shown in Figure 6 and within design lateral displacement; however, the displacement in the case of conventional active link is slightly lower than rotational active link. This is due to closely matching the demand on the links with their design capacity, which assisted distributing the demand on the active links towards top storeys as opposed to conventional active links. The inter storey drift of the building are within the design limit, and with the exception of bottom two storeys, where the drift exceeds the design drift. as shown in Figure 7.

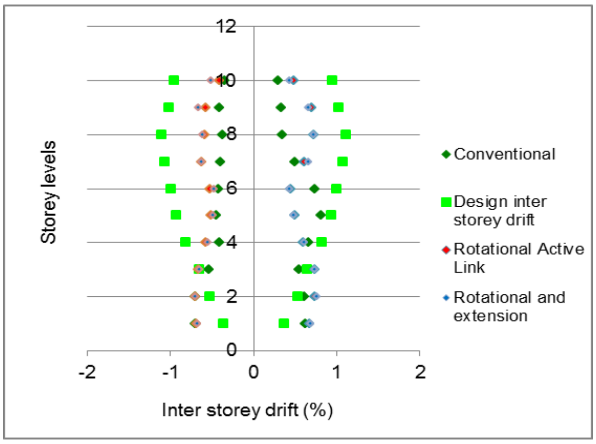


Figure 7: ***Maximum average positive and negative inter storey drift of each level.***

The result from active links rotations, as shown in Figure 8, indicate that the maximum active link rotation occurs towards the bottom of the building as a typical inelastically responding system. The maximum average rotation of rotational and rotational + extension active link is almost similar. However, maximum average rotation in conventional active link is very similar to rotational link at the bottom storey, but the rotation of conventional active link decreases towards the top storeys. This is due to closely matching the demand on the links with their design capacity, which assisted distributing the demand on the active links towards top storeys as opposed to the conventional active links. The higher rotational demand in the rotational active links is due principally to their shorter length between points of rotation, as described previously

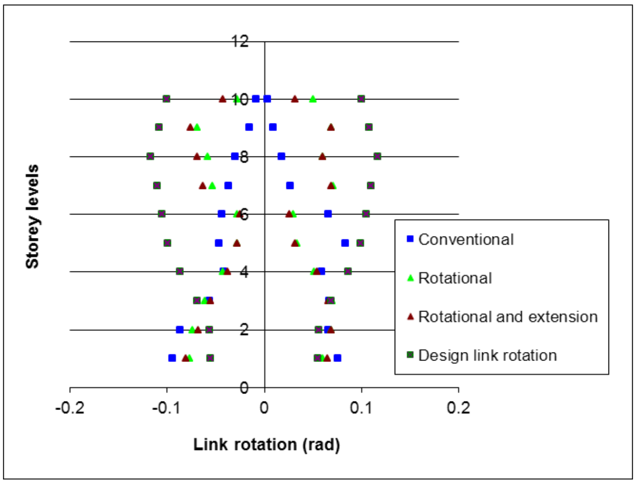


Figure 8: ***Maximum average positive and negative rotation of each of the active link types.***

The average maximum extension of the rotational active link, in the case of rotational link + extension, does not exceed 1.5 millimetres in total, as shown in Figure 9. The extension does not have any significant effect on the results when compared with rotational active link (only with rotation), which can be seen in Figure 6, 8 and 9, where it shows no differences in between two rotational active links. Figure 11 shows the moment-rotation from all three types of active link on level 1, which sustain the greatest inelastic demand. The results from Figure 11 also demonstrate that there are no significant differences in energy dissipating mechanism between rotational active link and rotational active link with extension, and the hysteresis curve for conventional active link is well within the range of rotational active links.

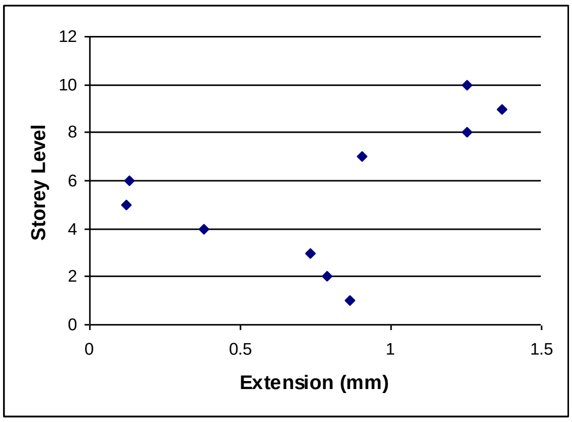


Figure 9: ***Maximum average extensions of rotational active link + extension.***

The residual drift was determined by allowing 10 seconds of free vibration at the ends of the earthquake records. The residual drifts from the rotational active link system are lower than those from the conventional active link system. However, as no allowance was made for the self-centering capability of the floor slab in either instance, the results would not be representative of an EBF building with floor slab and so are not included in this paper.

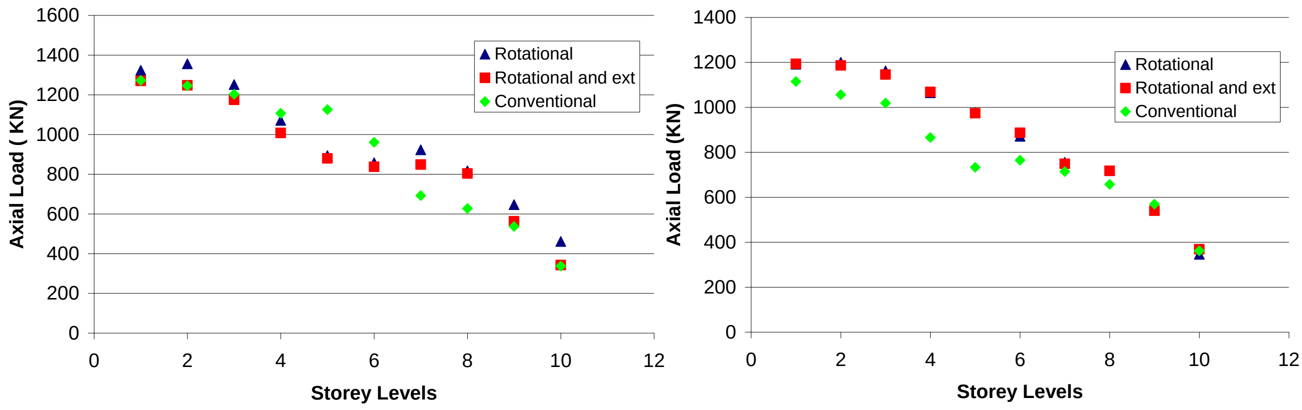


Figure 10: The axial load of rotational, rotational and extension, and conventional active link with ten storey building: (a) Right hand side collector beams, (b) right hand side braces.

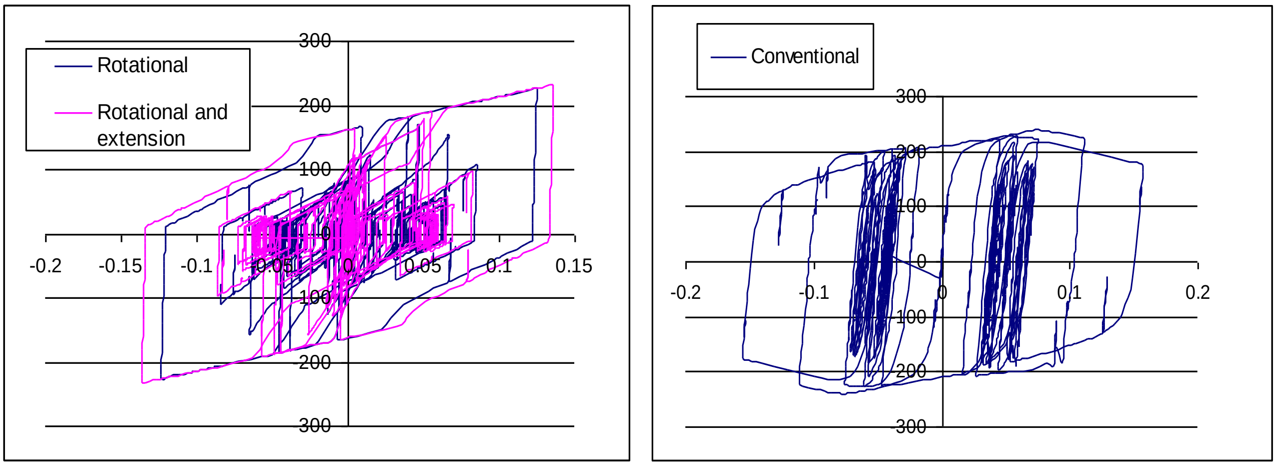


Figure 11: The moment-rotation from all three active links on level 1 from analytical results.

EXPERIMENTAL RESEARCH PROGRAMS

Development of this proposed EBF system with rotational active links will require a programme of experimental testing and numerical modelling of the proposed active link, followed by numerical simulation of complete building response using these new rotational links to a representative range of severe earthquake events to determine that the performance of the system meets or exceeds expectations. Overview details only are given in the project plan below.

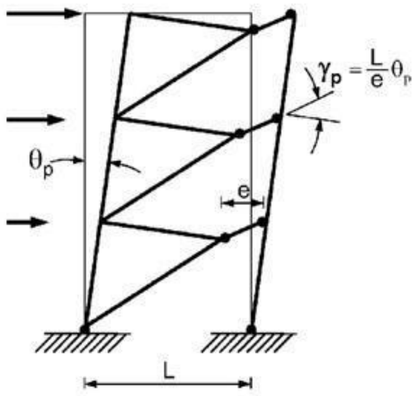
The experimental work will be based on two main category of EBF systems, which are the V braced and D braced system. It is proposed to initiate the experimental work by using a 460UB for the active link stubs, as the preliminary calculations show that this size allows development of the shear capacity required in a medium rise building active link..

In a D-braced system, one end of the active links is attached to column and other end to the collector beam. However, it is not physically practicable to construct the whole system, as shown in Figure 12a, for testing.

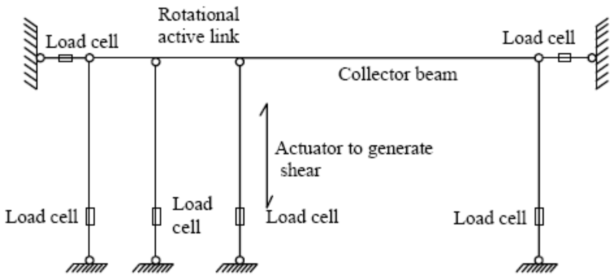
Therefore, the rotational active links of this D braced system will be tested individually under cyclic loading using the test setup shown in Figure 12b. This models realistically the support condition, relative deformation and extension across the active link.

D-braced links have support conditions with different strength and stiffness at each end of the active link, whereas V-braced active links have similar strength and stiffness at each end as demonstrated in Figure 1a. Figure 13 shows the potential set-up to test a V-braced system. This set-up comprises of a conventional EBF being supported with a dynamic actuator at one end imposing a push/pull horizontal displacement to generate the inelastic shear demand in the active link.

The proposed experimental setup of the active link in a D-braced configuration is demonstrated in Figure 12b, where the cyclic loading is applied using a hydraulic actuator. Beam and column sizes will selected to match the 460UB active link size and the test layout will be slightly different to suit the laboratory strong floor and wall configuration. The revised loading protocol, from the 2005 AISC Seismic Provisions (AISC 2005), will be used for cyclic loading on the rotational active links. This revised loading protocol has been developed by Richards Paul and Ming U Chia in order to better represent the actual inelastic demand on an EBF active link compared with the previous testing protocols, such as ATC-03 (Richards & Uang 2003). The paper details and NITH concentrates on the V braced system, as there is insufficient time and resources in this project to satisfactorily address both systems and the V braced has been chosen as it is the most commonly used (over 75% of EBFs in NZ are V braced). The issues with the V braced and the development of the slotted bolted system will be relevant in general and in many instances in particular to D-braced systems.



(a)



(b)

Figure 12: ***An example of: (a) a D braced system of EBF (Okazaki & Engelhardt 2007) and (b) Test set-up for a D braced system of EBF which extends the active link to represent the rigid attachment to the column.***

EFFECT OF FLOOR SLAB ON EBF BEHAVIOUR

The behaviour of the overall floor slab, above the active link, when subjected to active link vertical movement due to earthquake motion, is not well understood. It will offer significant resistance to the vertical movement of the collector beam/active link system, increasing the effective strength of the EBF and reducing the inelastic demand. It will also increase the axial forces on the braces. None of these effects has been experimentally quantified, although the benefits have been clearly observed in the 22nd February Christchurch earthquake (Bruneau *et al*, 2011) through significantly reduced inelastic demand on the active link compared what that expected for the design ductility level and earthquake intensity. The effects on this system will be determined from experimental testing as shown in Figure 14 and three dimensional numerical modelling.

FINITE ELEMENT MODELLING TO EXTEND EXPERIMENTAL RESULTS

The experimental results will be based on a typical size active link used in New Zealand, being a 360UB50, Grade 300. A hysteretic model will be developed and validated against the test results, through obtaining close agreement between the hysteresis curve and the magnitude and pattern of post-elastic behaviour for a short active link and a long active link.

The model will then be used to determine the behaviour of the largest size of active link used in practice and to compare the performance of D-braced active links (as tested) and V-braced active links, for each of the three behaviour regimes; short, intermediate and long. This will ensure the influence of scale effects is captured by the research programme as well as differences between K-braced and V-braced active links.

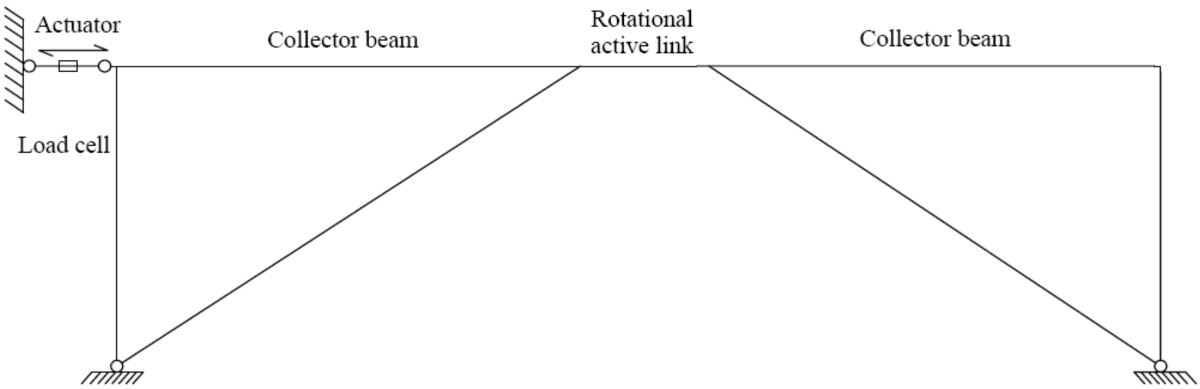


Figure 13: ***An example of a test setup of an active link of a V braced system when subjected to a cyclic loading.***

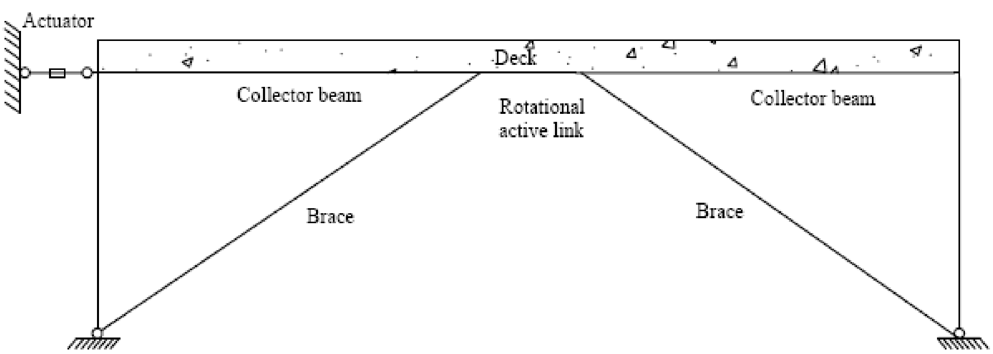


Figure 14: ***An example of a test setup for a rotational active link with slab in a V braced mode.***

CONCLUSIONS AND OUTCOME OF PROJECT

This paper describes the concept behind a low damage form of active link (rotational active link) for eccentrically braced frames and analytical analyses on a ten storey building with built in conventional and rotational active link to undertake initial proof of concept. From the analyses, it can be concluded that:

The lateral displacement of the building is almost identical for three types of active link as mentioned above. The rotation for these three types of active link is similar towards the bottom storeys, however the rotation for the conventional active link is much lower as compared to the rotational active links towards the top storeys, and this is due to the fact that inelastic demand on the rotational active link is distributed to some extend towards top storeys. The analysis also shows that there is no significant extension of rotational active link, and no significant differences on the axial demand of collector beams and braces.

The results are positive and will now lead to the programme of research described above. The outcome of this project will be a new form of damage resisting EBF with design procedure and detailing requirements which will offer all the advantages of existing systems in terms of strength and stiffness, with the following additional advantages:

1. Minimum or no post-earthquake repair required and this easy to undertake off the critical path.
2. Decoupling of strength and stiffness allowing the same active link stub/collector beam sizes to be used at all levels in the case of rotational active link.
3. While providing varying shear strength at different levels to match the design earthquake shear strength
4. Ability to enhance the self centering capability of the EBF system.

An additional benefit is that fabrication and erection of the system will use well established methods and not require any new skills or equipment.

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