Brief Update on Steel and Composite Steel Concrete Seismic Projects Underway at UofA June 2017

QuakeCoRE Flagship 4 June 2017 monthly meeting
Presentation by Charles Clifton

The University of Auckland

Scope of Presentation

- Brief overview of structural steel and composite steel/concrete projects underway at UofA
- More details on the BRBF gusset plate project
- Mostly covers new systems that could be applied to exemplar building or improvements to current systems
- One project on post earthquake evaluation
- One on light gauge steel systems

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Buckling Behaviour and Design of Gusset Plates in Braced Steel Structures

Behnam Zaboli, PhD student, starting second year of research

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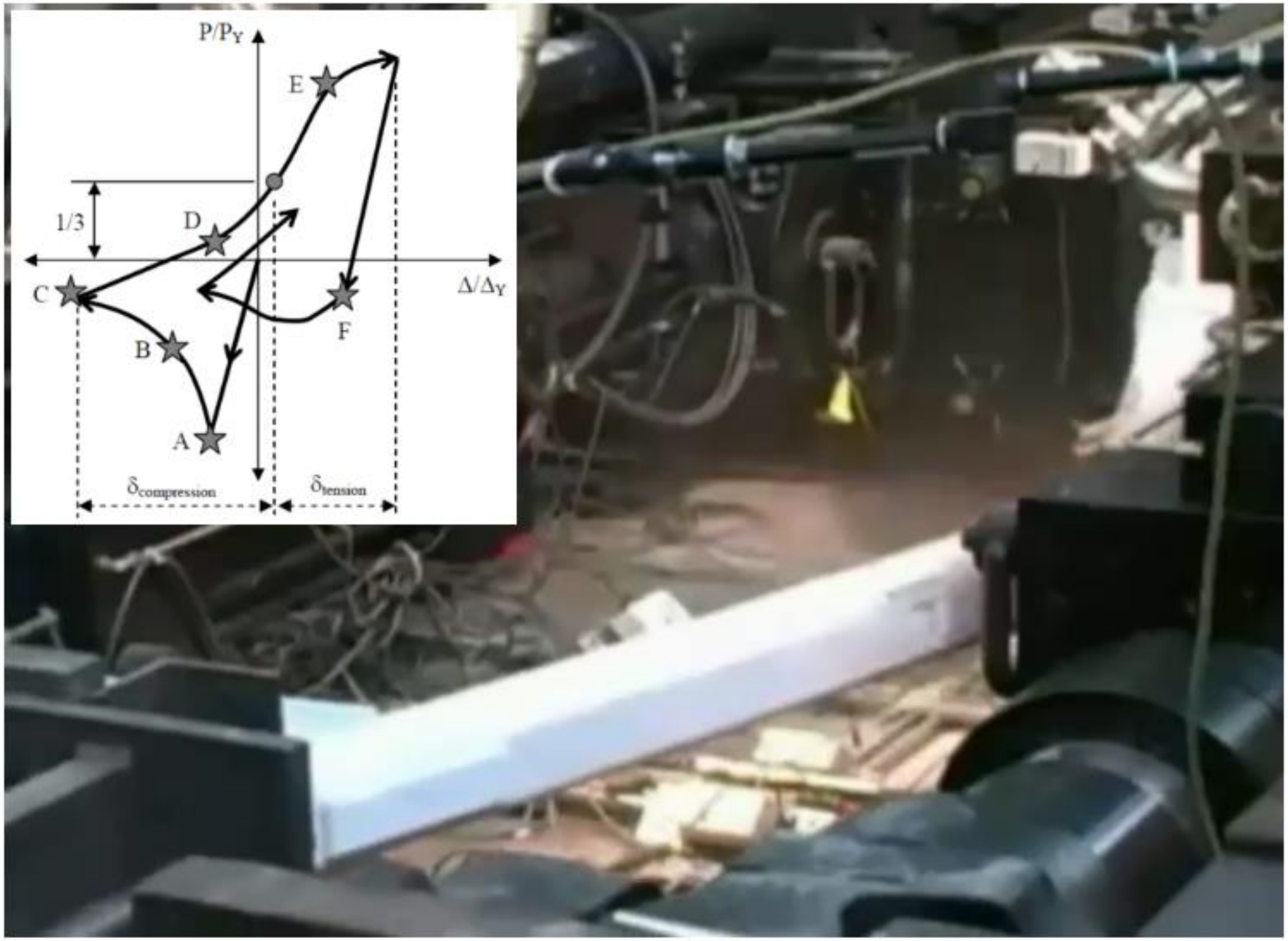
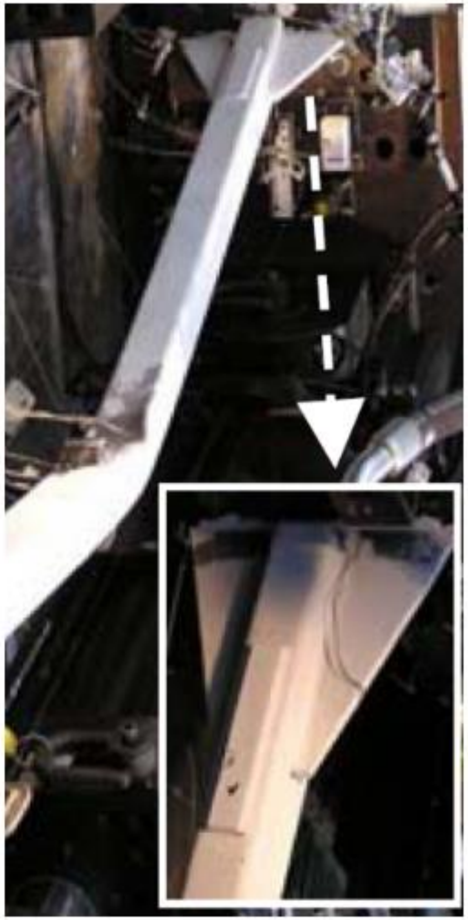
Motivation & Objectives

- **Motivation:**
  - Inadequacy of existing design equations in predicting the gusset plate buckling capacity especially for BRBFs and RSFJ braces (which don’t buckle out of plane)
  - Frame action imposes severe demand on gusset plate and framing components

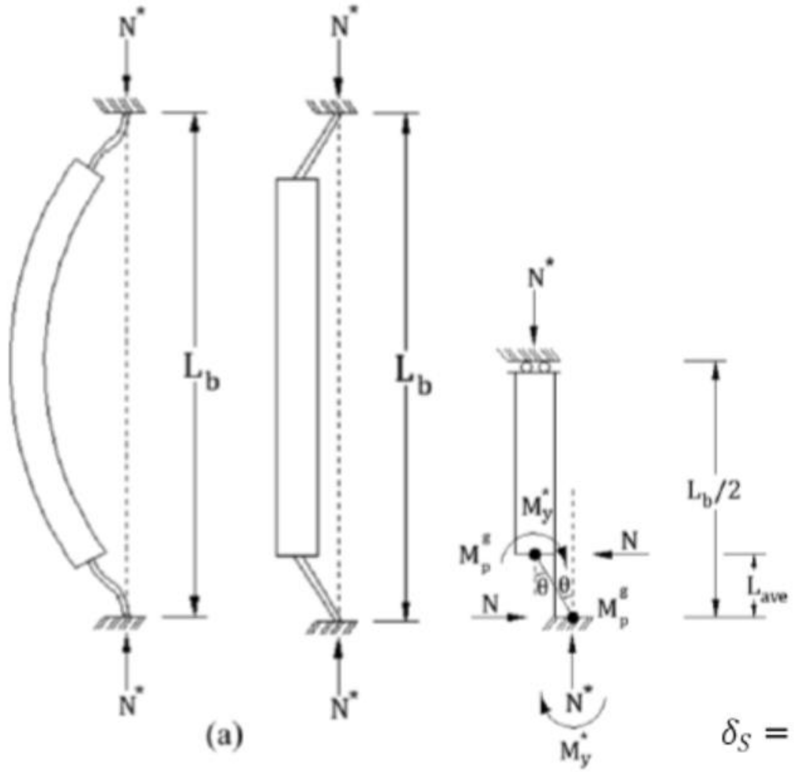
- **Objectives:**
  - Develop a reliable and practical approach so that engineers can readily employ it in practice
  - Enable structural engineers to have a more clear understanding regarding the stability of gusset plates
  - Investigate the influence of proposed low-constraint gusset plates to beam/column connections on the frame action interaction with the in-plane action from the brace

Traditional Concentrically Braced Frames (CBFs) Behaviour

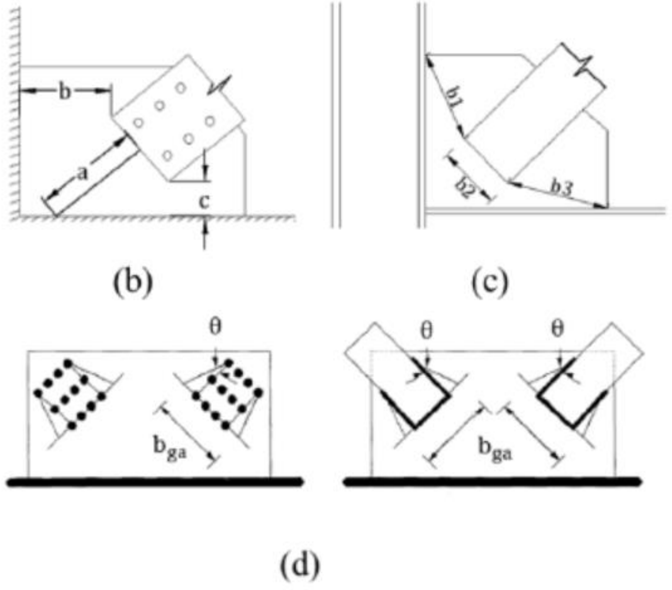




Proposed Design Procedure for CBFs



(a)



(b) (c)

(d)

$$N = \theta_i N^*$$

$$N L_{ave}\delta_S\theta = 2M_p^g\theta \Rightarrow M_p^g \ge \frac{N L_{ave}\delta_S}{2} = M_y^*$$

$$L_{ave} = min\begin{cases}(a+c)/2\\(a+b+c)/3\end{cases}$$

$$\delta_S = \frac{1}{1-\frac{N^*}{N_e}} \ge 1$$

$$N_e = f_e t_g b_{ga}$$

$$f_e = \frac{\pi^2 E}{\left(\frac{L_{ave}}{r_g}\right)^2}$$

$$IN:\begin{cases}\left(\frac{N^*}{\phi N_s^g}\right)^2 + \frac{M_y^*}{\phi M_{sy}^g} \le 1 & regular\ gusset\\ \left(\frac{N^*}{\phi N_s^g}\right)^2 + \frac{M_y^*}{\phi(1.19)M_{sy}^g} \le 1 & stiffened\ gusset\end{cases}$$

$$N_s^g = t_g b_{ga} f_y$$

$$M_{sy}^g = S_g f_y = \frac{b_{gf} t_g^2}{4} f_y$$

$$b_{gf} = b_1 + b_2 + b_3$$

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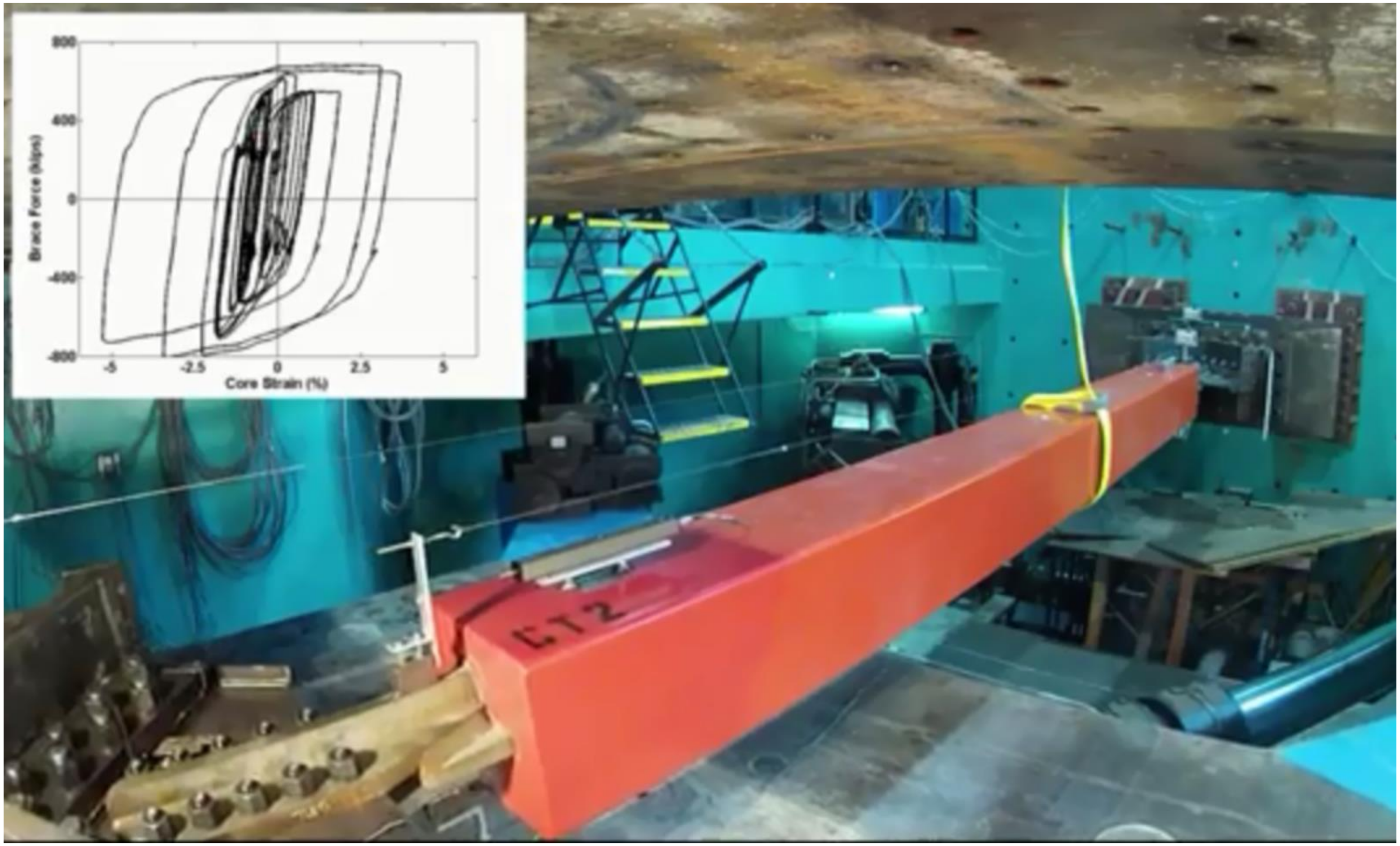
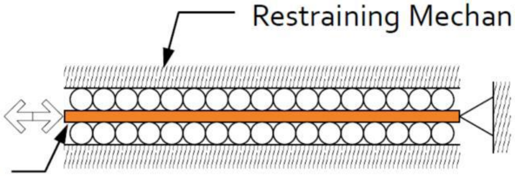
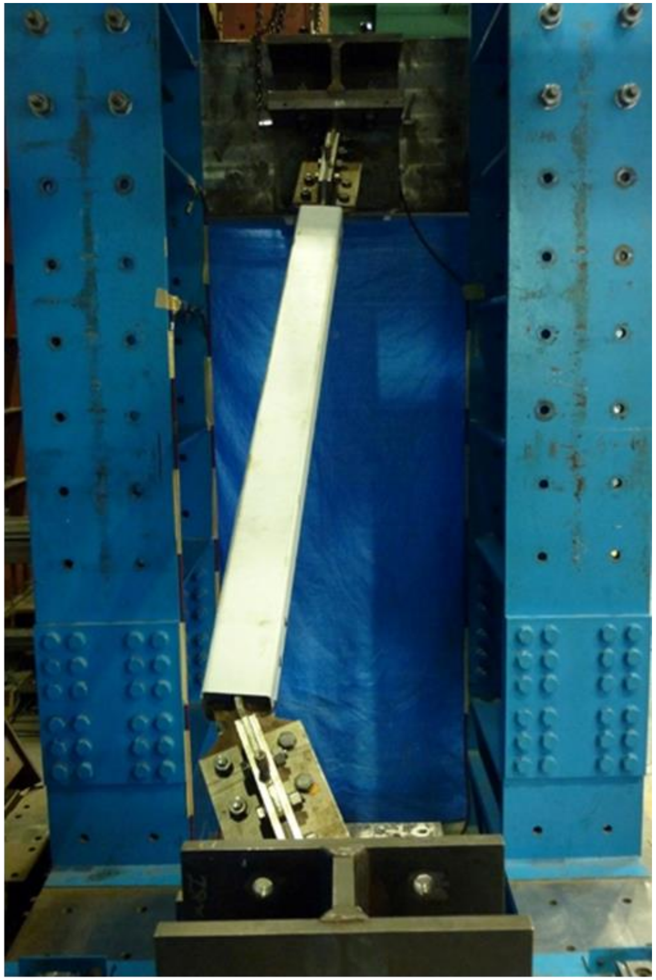
Buckling Restrained Brace (BRBs) Behaviour

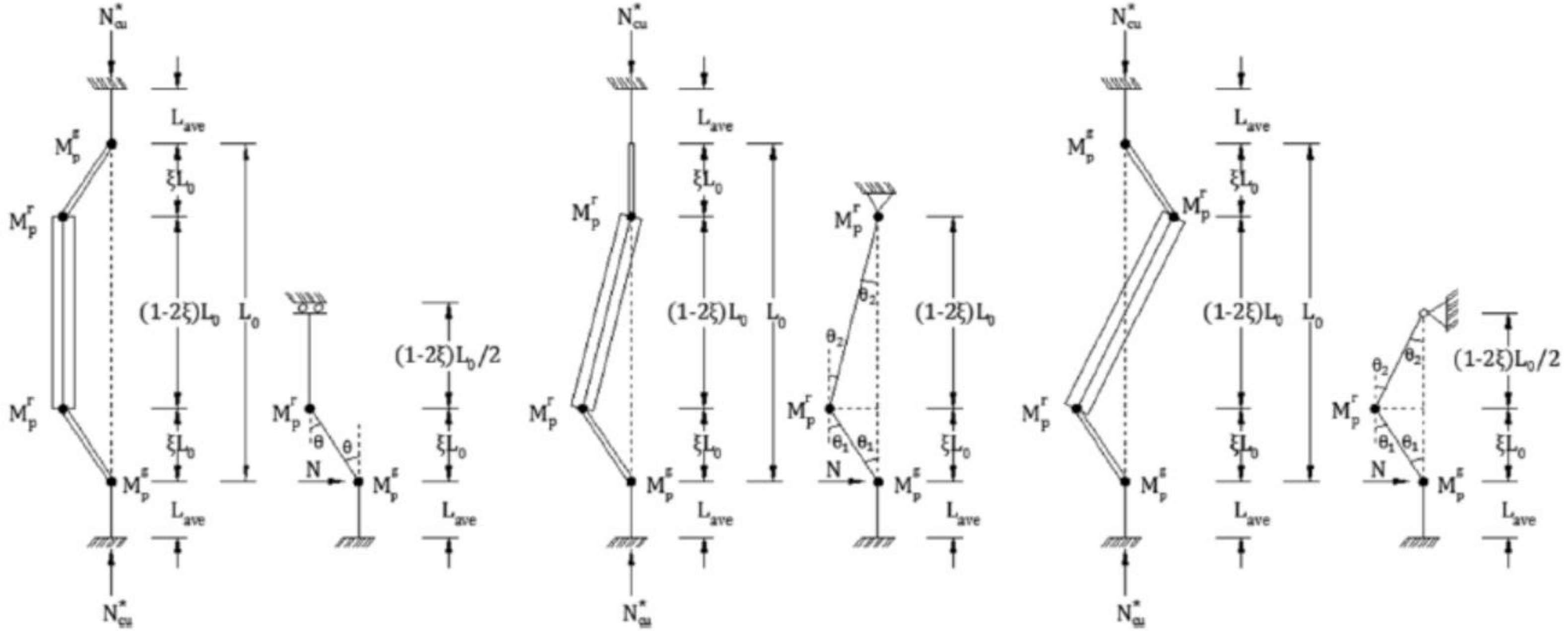




BRB Connection and Stability Issues



Proposed Design Procedure for BRBs



Symmetrical mode:

$$N\xi L_0\delta_S\theta = M_p^g\theta + M_p^r\theta$$
$$N\xi L_0\delta_S \le M_p^g + M_p^r$$

$$N = \theta_i N_{cu}^\star \qquad \delta_S = \frac{1}{1-\frac{N_{cu}^\star}{N_{cr}^B}} \ge 1 \qquad N_{cr}^B = \frac{\pi^2 E I_B}{(kL_0)^2} \longrightarrow \begin{cases} k = 1 & \textit{regular gusset} \\ k = 0.7 & \textit{stiffened gusset} \end{cases}$$

One sided mode:

$$N\xi L_0\delta_S(\theta_1+\theta_2) = M_p^g\theta_1 + M_p^r(\theta_1+2\theta_2)$$
$$N\xi L_0\delta_S(1-\xi) \le (1-2\xi)\,M_p^g + M_p^r$$

Asymmetrical mode:

$$N\xi L_0\delta_S(\theta_1+\theta_2) = M_p^g\theta_1 + M_p^r(\theta_1+\theta_2)$$
$$N\xi L_0\delta_S \le (1-2\xi)\,M_p^g + M_p^r$$

$$M_p^g = \begin{cases} \phi M_{sy}^g\left(1-\left(\frac{N_{cu}^\star}{\phi N_s^g}\right)^2\right) \\ 1.19\phi M_{sy}^g\left(1-\left(\frac{N_{cu}^\star}{\phi N_s^g}\right)^2\right) \end{cases}$$

$$M_p^r = min\{M_p^{r-neck},\ M_p^{r-rest}\}$$

$$M_p^{r-neck} = \begin{cases} \phi M_{sy}^n\left(1-\left(\frac{N_{cu}^\star - N_{wy}^n}{\phi(N_y^n - N_{wy}^n)}\right)^2\right) \\ \phi M_{sy}^n\left(1-\left(\frac{N_{cu}^\star - N_{wy}^n}{\phi(N_u^n - N_{wy}^n)}\right)^2\right) \end{cases}$$

Comparison to test data

- **CBF Specimens**

Specimen	Failure axial force (kN)				
	AISC $(k = 0.5, l_b = l_1)$	NZS 3404 $(k = 0.7, l_b = l_1)$	NLYL	Experiment (Gross 1990)	
				General yield capacity	Ultimate buckling capacity
No. 1	319	221	380	396	516
No. 2	319	221	380	400	614

- **BRBF Specimens**

Specimen	Failure axial force (kN)					
	Gusset buckling under the yield-line			Plastic failure over the yield-line		Experiment (Takeuchi 2014)
	AISC $(k = 0.5; l_b = l_1)$	NZS 3404 $(k = 0.7; l_b = l_1)$	$\mathrm{NLYL}_{\mathrm{gb}}$	$\mathrm{NLYL}_{\mathrm{pf}}$	Takeuchi method	
MRL1.0S1H	620	561	778	725	818	-
MRL2.0S1	620	561	608	436	520	535
MRL2.0S2	620	561	556	367	410	507
MCL2.0S2	620	561	562	385	432	375
MRL1.0S1	620	561	565	326	345	362
MRL1.0S2	620	561	482	207	217	300

* MRL1.0S1H did not fail in experiment

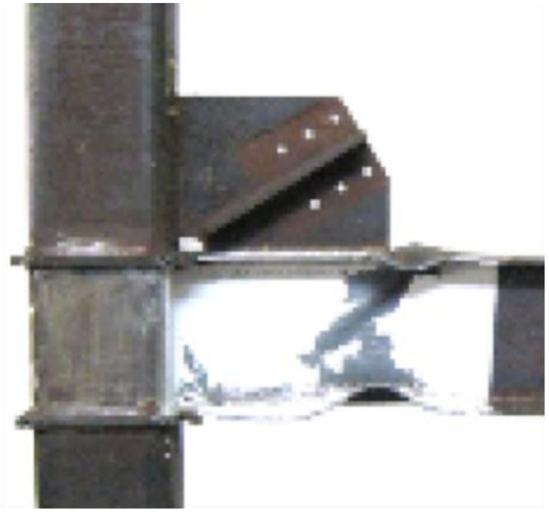
Research outcomes

- For CBFs both the AISC and NZS 3404 methods are too conservative and overdesign the gusset plate. In several past experiments, column and beam hinging or fracture was observed as a result of the generation of a large in-plane bending moment. Designing a gusset plate by the NLYL method will yield the minimum required thickness and reduce the mobilization of this in-plane stiffness.

- For BRBFs neither the AISC nor NZS 3404 can predict the failure axial force conservatively owing to a larger initial imperfection in BRB specimens, however the NLYL method produces suitably conservative results

Some Possible Damage by Frame Action

(a)



Low-constraint Gusset Plate Ideas

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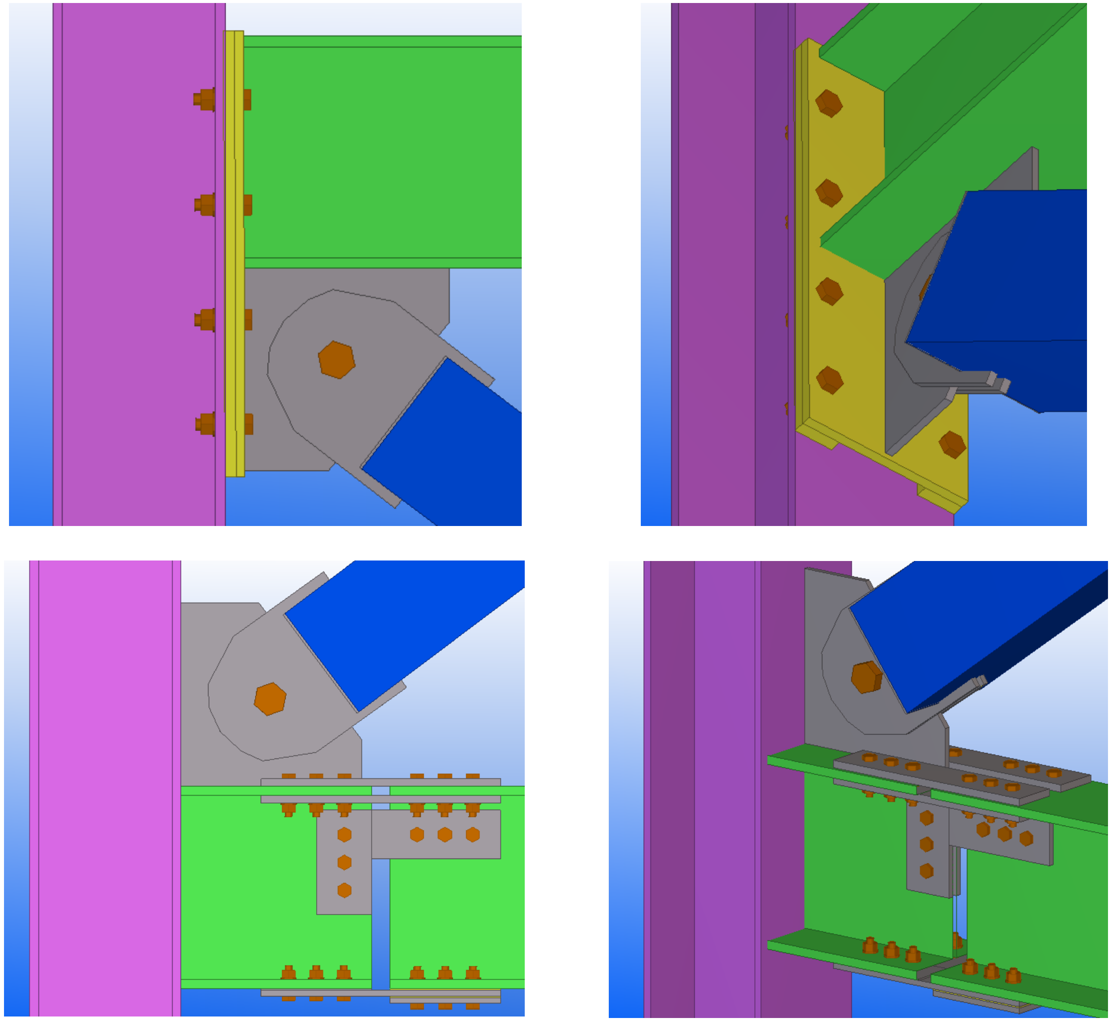

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Near future plan

- Experimental investigation will be conducted to research further the proposed NLYL procedure and behavior of low-constraint gusset plates
- Appropriate finite element analysis models will be carefully verified with the results of full-scale experimental tests
- Parametric study on different types of connections will be carried out using the calibrated finite element analysis models
- The final design procedure and detailing requirements which are robust, logical and consistent with the observed behavior will be developed

Proposed Test Setup

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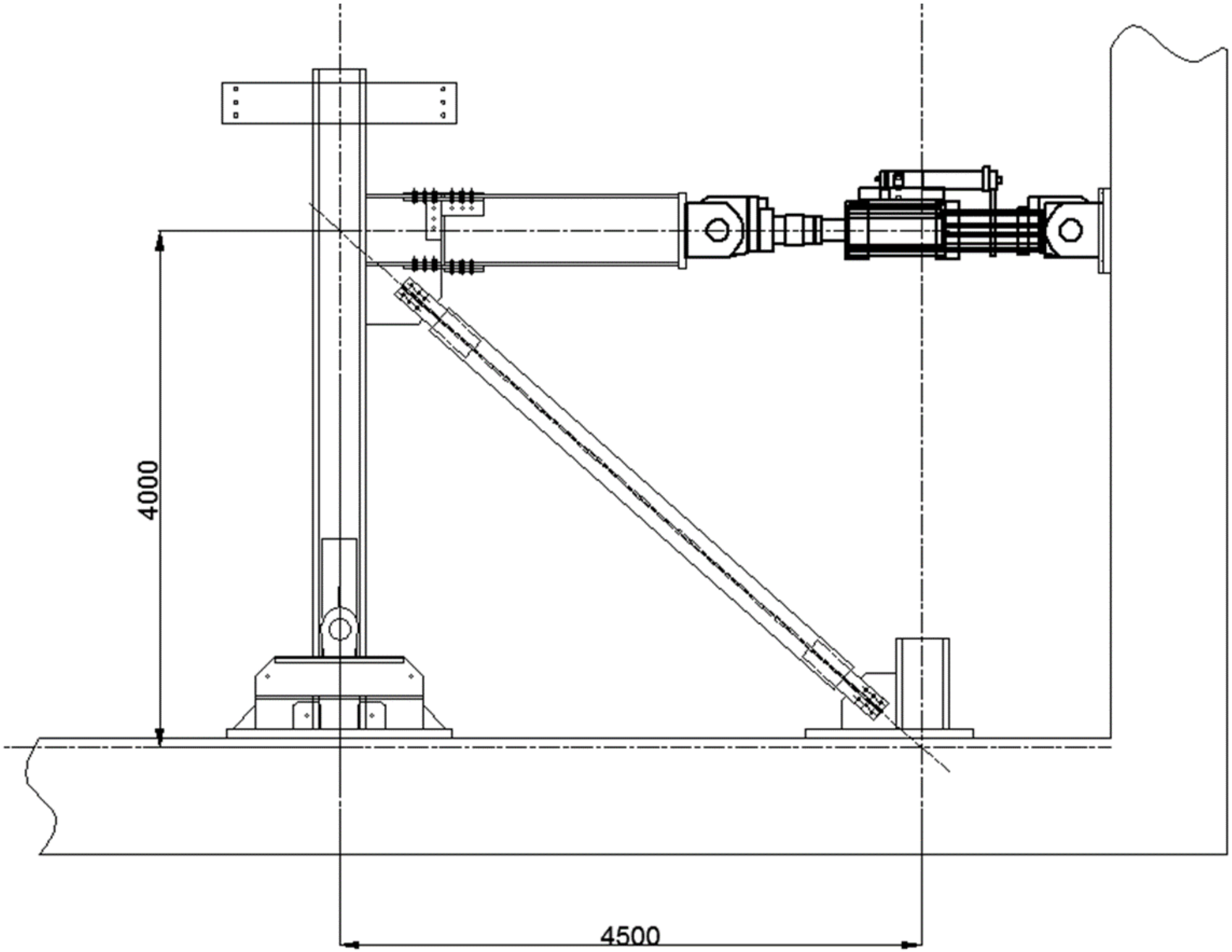

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Behaviour and Design of Composite Metal Deck Diaphragms Subjected to In-Plane Shear Forces

Hooman Rezaeian, PhD student, starting second year of research

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Status of Knowledge, Research

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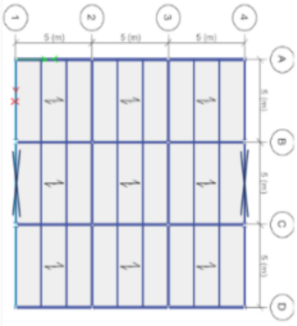
- Currently no agreed design procedure for diaphragm design taking into account inertia, displacement incompatibility and transfer forces
- Each round of design procedure development increases the diaphragm design actions
- For steel concrete composite floors requires now special detailing of adjacent bays with drag beams
- Composite floors have performed well without these details

⇒ are we underestimating capacity? No testing is available



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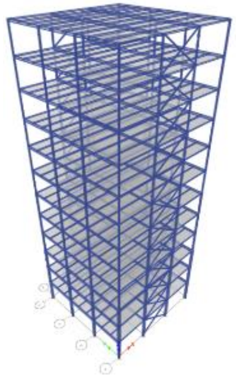
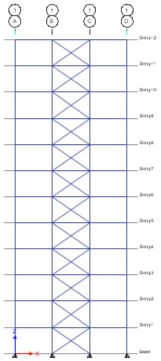
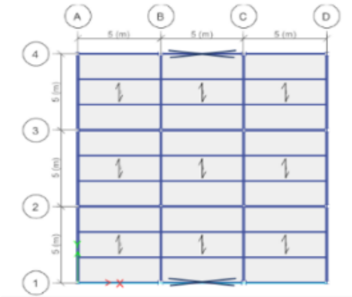
Nonlinear Time history Analysis



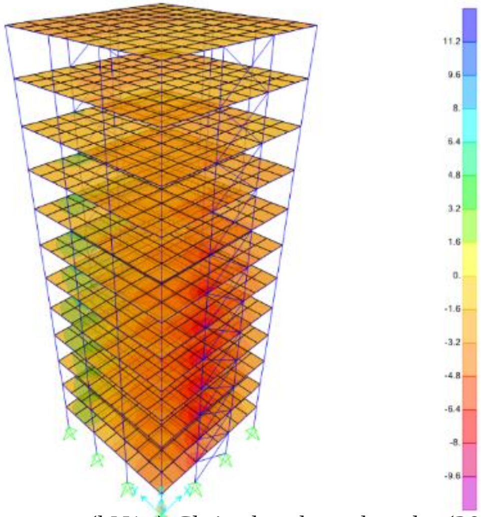
(a) 3D view



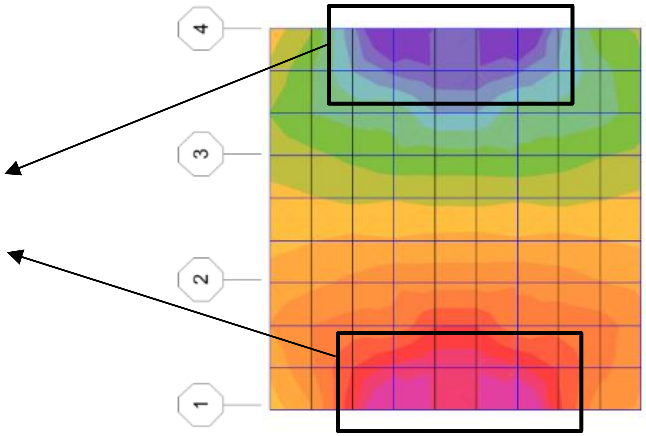
(b) Elevation view (Axis 1)



c) Plan view

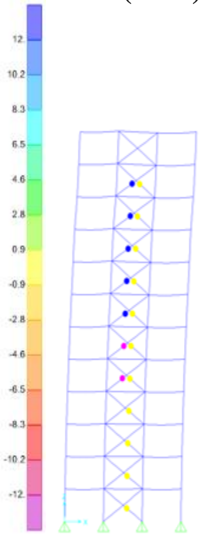


F12 contour (kN/m)-Christchurch earthquake (2011)



Concentrated shear forces

12th floor-step: 12s



F12 contour (kN/m)-Christchurch earthquake (2011)

Finite Element modelling

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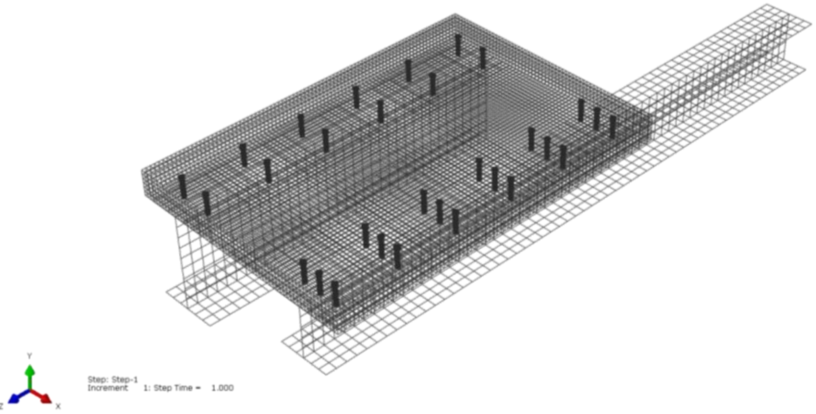
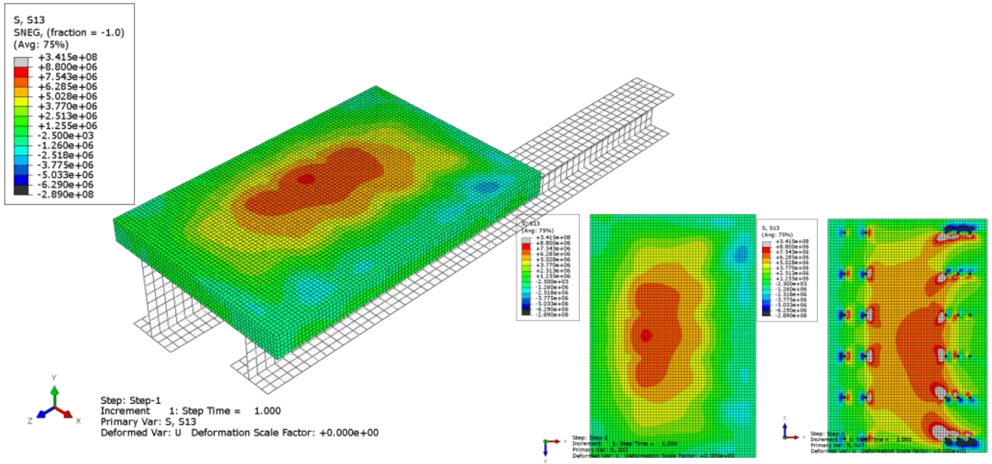


3D Finite element model

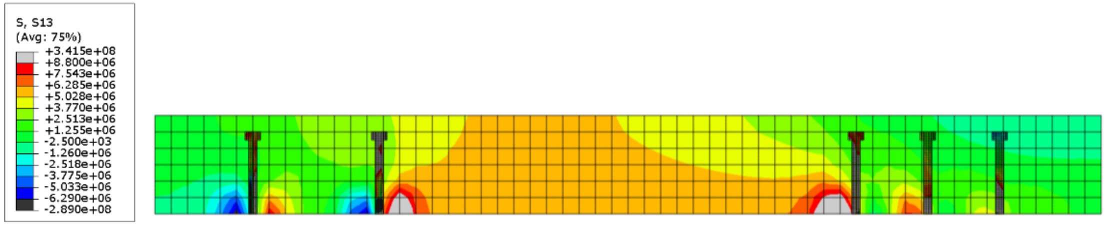


3D view

Top View

Bottom View

S13 stress



S13 pattern in a section of composite metal deck slab

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Test Setup

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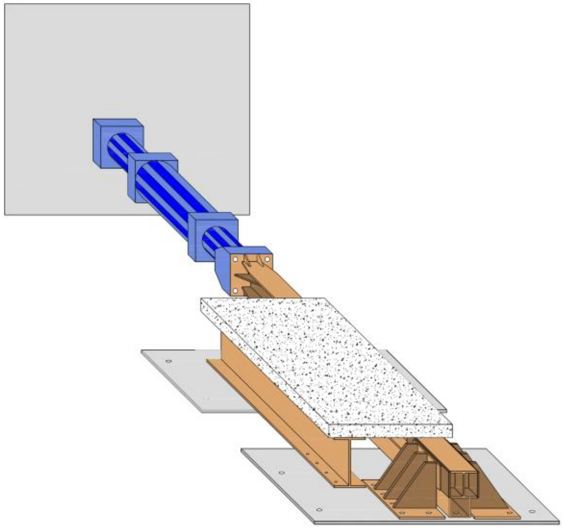
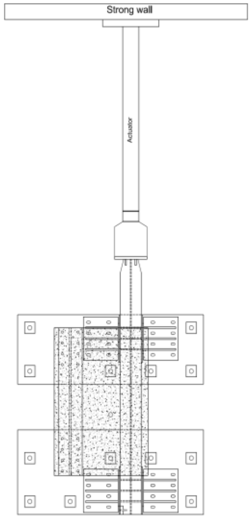
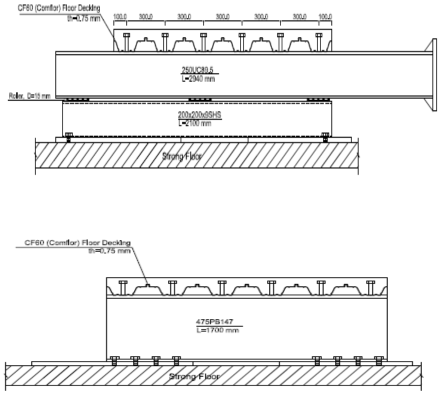


3D view

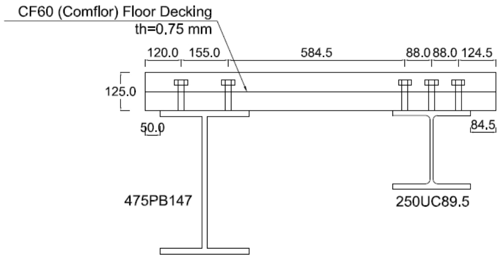


Top view

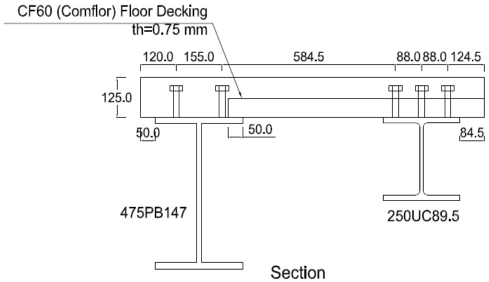


Side views

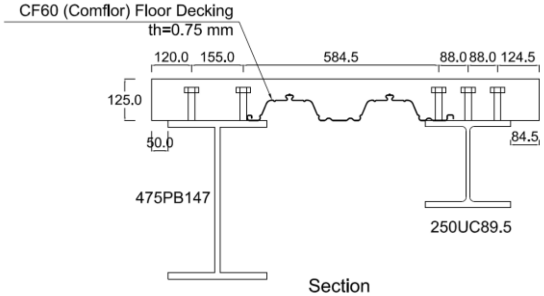
Test Setup



Section
1



Section
2



Section
3

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Section of Specimens

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Nine Tests Planned

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- Decking orientation primary and secondary beams
- Decking shear stud welded though or just pinned to beam
- 1 Monotonic and 2 Cyclic tests on each configuration
- 9 tests total
- EQC funding being sought

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Optimising the Sliding Hinge Joint with Belleville Springs

Shahab Ramhormozian, PhD student, final year of research

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The Sliding Hinge Joint Connection (SHJ)

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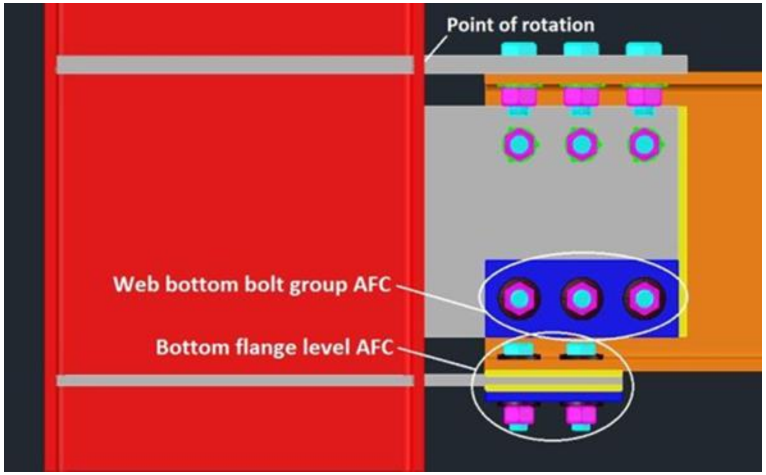
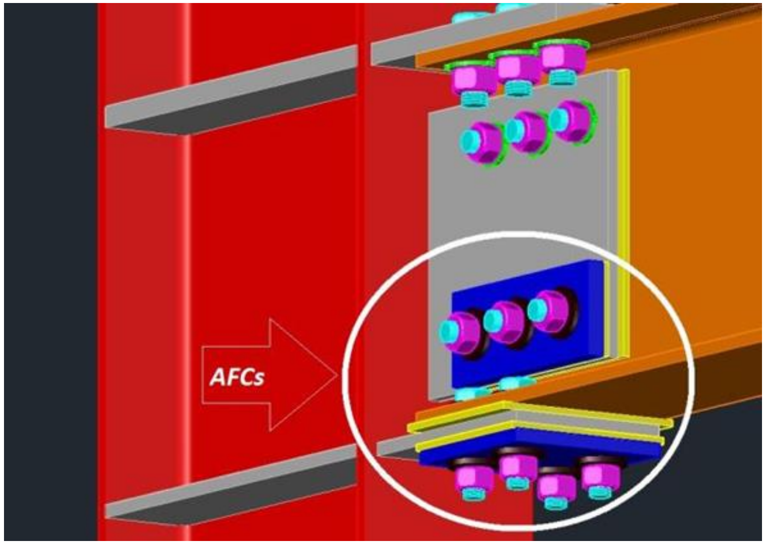
- Developed by Clifton at UoA (1998-2005);
- Further development at the UoA and UoC;
- Widely used in New Zealand;
- Rigid up to ULS and sliding under severe events with minimal damage through dissipating energy by the Asymmetric Friction Connections (AFCs).

- **Key benefits:**
  - Decoupling joint strength and stiffness.
  - Isolating the floor slab.
  - Confining yielding to the bolts. "*Intended to be improved or ideally avoided*"
  - Pinched hysteresis behaviour. "*intended to be improved*"

- Current research to develop true low damage system





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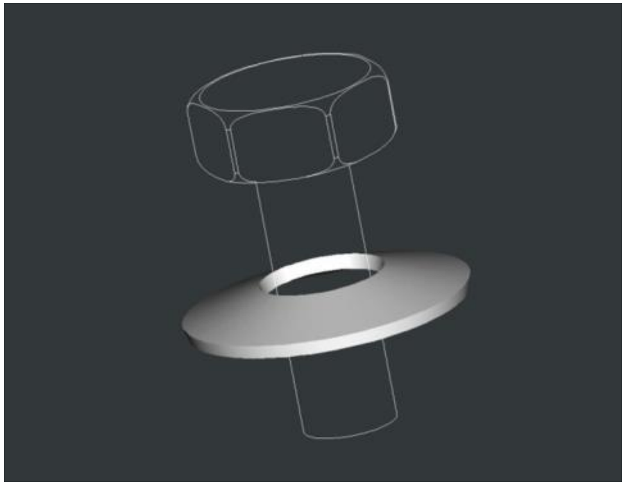
Using Belleville Springs to retain the post earthquake strength and provide better self-centering



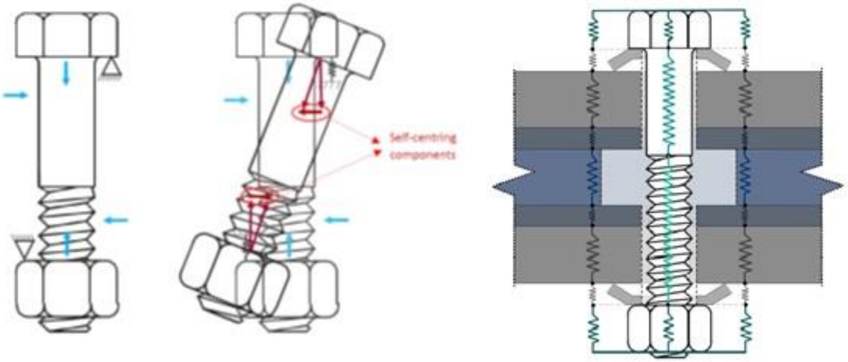
- Installing the bolts in the elastic range

+

- Using partially squashed Belleville springs

=

  - *Improved self-centering*
  - *Retaining the clamping force following severe earthquakes*
  - *More stable sliding behaviour*
  - *Eliminating damaging prying effects*
  - *Higher frictional resistance*
  - *Less surface degradation*

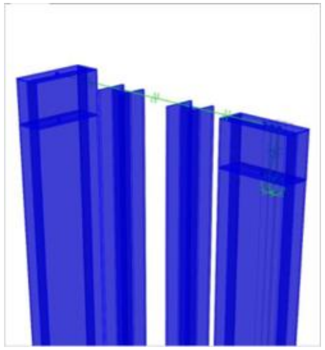


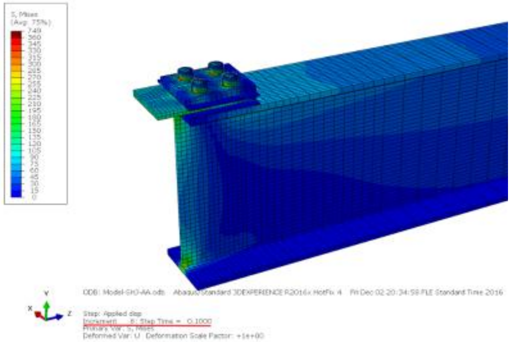
Recent Undertaken research

- **Analytical:** based on first principles;

- **Experimental:** using customized high precision tools to reach the highest level of accuracy and completeness in the experiments;
  - *More than 50 AFSHJ real scale component level dynamic tests with and without Belleville springs on two AFSHJ test setups*
  - *More than 200 bolt tightening and direct tensioning tests with and without Belleville springs.*

  - **Numerical:** using ABAQUS and SAP2000 softwares





Established findings (to be published in Journals in 2017/18:

- Developing a practical turn-of-nut-based methodology to tighten the HSFG bolts with BeSs in the bolt's elastic range using an AFC bolt tightening test setup. This removes the concerns about the delivered installed clamping force in the friction sliders.

- Establishing the optimum surface preparation/roughness level for the AFC plies sliding surfaces. This also removes the CoF variability concerns about the friction sliders.

- Establishing the optimum use of the Belleville springs. This removes the concerns about the post-earthquake elastic strength loss, damaging prying effects, and the variations of the bolt tensions during sliding.

- Establishing the optimum level of installed bolt tension in the AFSHJ. This is presented at the NZSEE2017 conference.

- Proposing required changes on current NZS3404 recommended method of bolt tightening. This removes the concerns about the reliability of the HSFG bolts delivered installed tension.

- Developing a dynamic SDOF SHJ model to investigate the effect of dynamic loading frequency, mass, and wind down on the static and dynamic self-centering capability at component level. This is presented at the NZSEE2017 conference.

- Experimentally investigating the shim-less AFC, AFC with TiN coated shims, and AFC with abrasion resistant cleat and shims.

Ongoing research

- FEM modelling of the SHJ AFC with and without BeSs to numerically investigate the effect of BeSs, optimum bolt tension, effect of number of bolt rows, effect of prying actions, effect of plies thickness reduction.

- Developing the AFC bolt model to design the AFC.
  - This is based on the first principals, will explain in details the behaviour of the AFC, and gives the modified design procedure.

- Developing a simplified but accurate MDOF building model of the SHJ to research the SHJ dynamic self-centring capability using SAP2000 considering the several parameters such as column base rotational stiffness, type of the friction damper, the additional linear elastic spring between the column and beam, and stepping column base.

- Pre and post earthquake system identification of the Te Puni Village SHJ building using SHM data.

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Elastically Rocking CBF Frames

Gary Djojo, PhD student, final year of research

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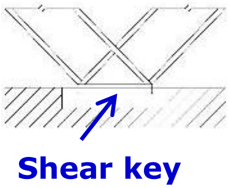
Timeline

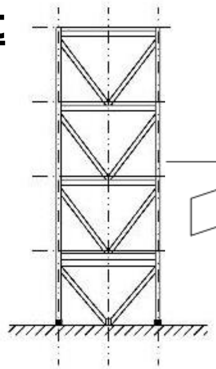
The Development of The Centralised Rocking CBFs (CRCBFs) with Double Acting Ring Springs

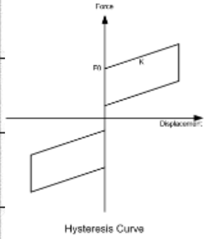
2012

Concept

2012-2013

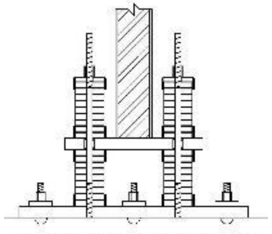


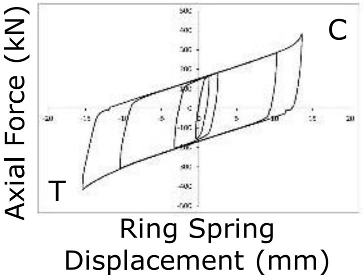




The Double Acting Ring Springs Type I

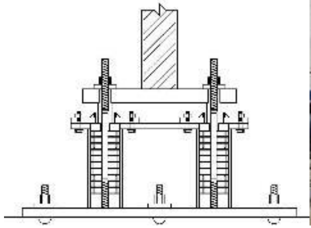
2013-2014

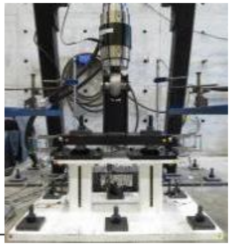


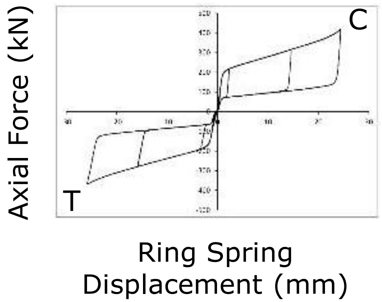


The Double Acting Ring Springs Type II

2015-2016

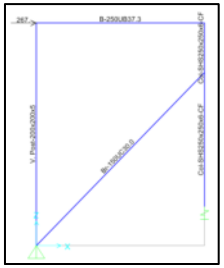




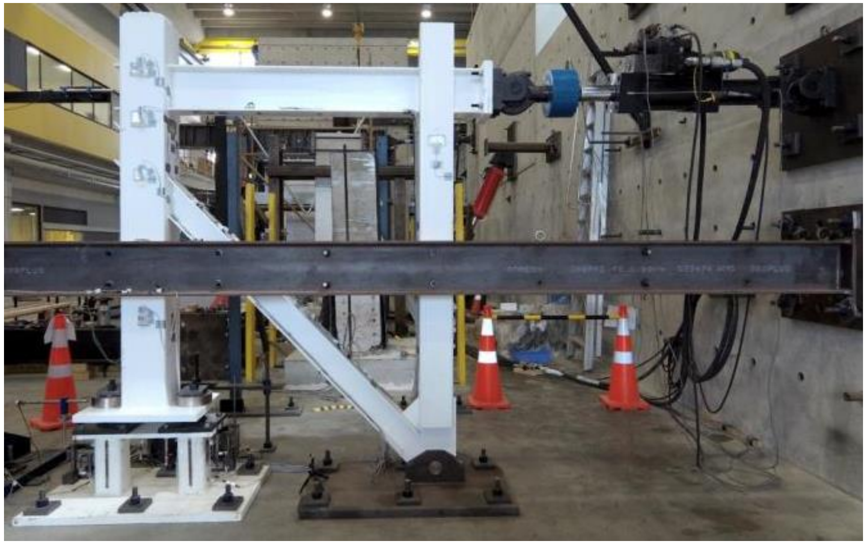


Experimental Validation of The CRCBF with Double Acting Ring Springs Type II

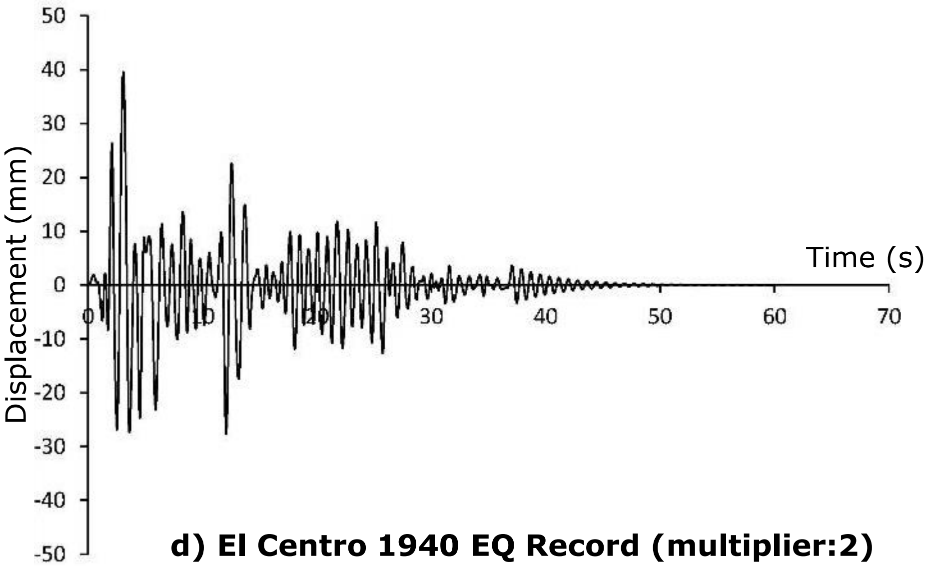
2016/2017



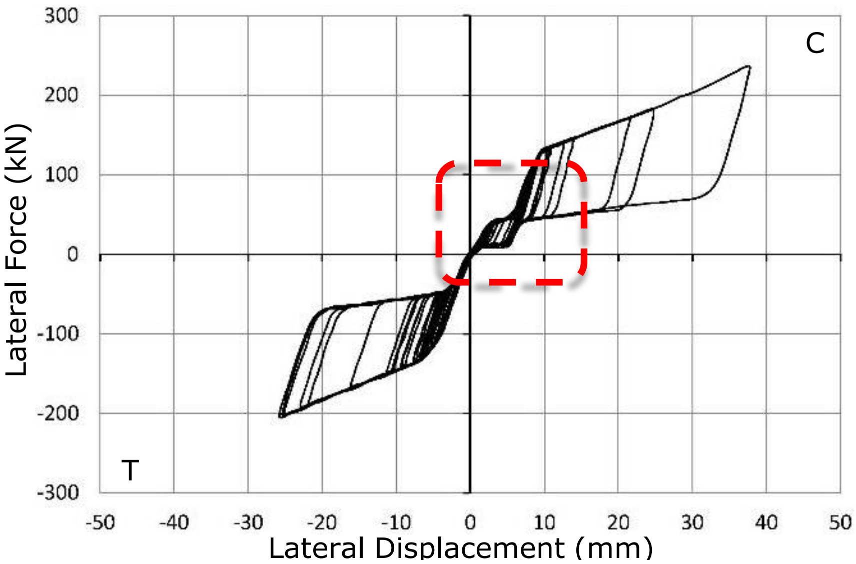
Loading Protocol & Test Result



Bottom Storey RCBF Testing



d) El Centro 1940 EQ Record (multiplier:2)



d) El Centro 1940 EQ Record (multiplier:2)

Investigate:

1. The global behaviour of CRCBFs and rocking pivot under static and dynamic loading.
2. Double acting ring spring movement due to an arc of the frame during rocking.
3. The internal forces from test results compared with analysis results.

Analysis Results VS. Test Results

Axial Force		Compression			Tension		
At 267kN		Test Results	Analysis Results	%	Analysis Results	SAP Results	%
Beam	kN	246	246	0%	-250	-246	-2%
Brace	kN	-372	-373	0.3%	363	373	3%
V. Post	kN	-59	-72	18%	40	72	44%
Bot. Col.	kN	179	355	50%	-324	-355	9%
RS reaction	kN	327	355	8%	-344	355	3%

Notes: 1. Compression is (+); Tension is (-); all readings are from steel components.
2. The subtraction of Analysis Results and Test Results in Compression gave the amount of Concrete Contribution in Compression.
3. The difference between Analysis Results and SAP2000 Results was considered minimal.

Conclusion

1. The test results showed a good match with the analysis results.
2. CRCBF system remained elastic and self-centred precisely to the initial position proving that it is working properly as intended in the concept.

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Post Earthquake Condition of Plastically Deformed EBF Active Links Including the Effect of Strain Ageing

Robert Currie, ME Student, submitted thesis end May

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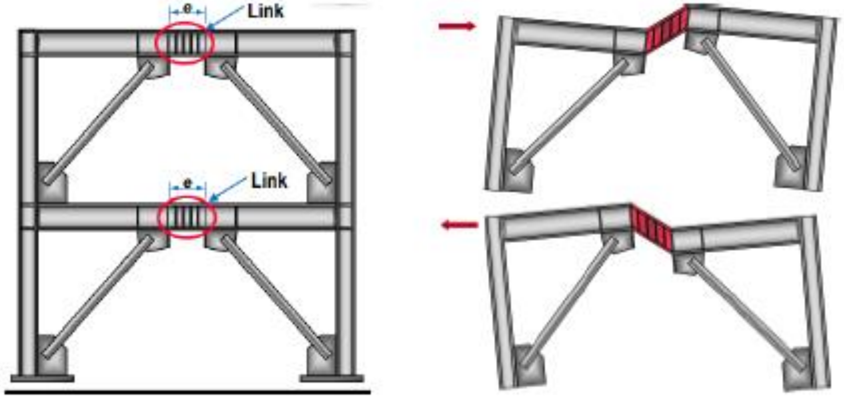
EBF Active Links in EQ

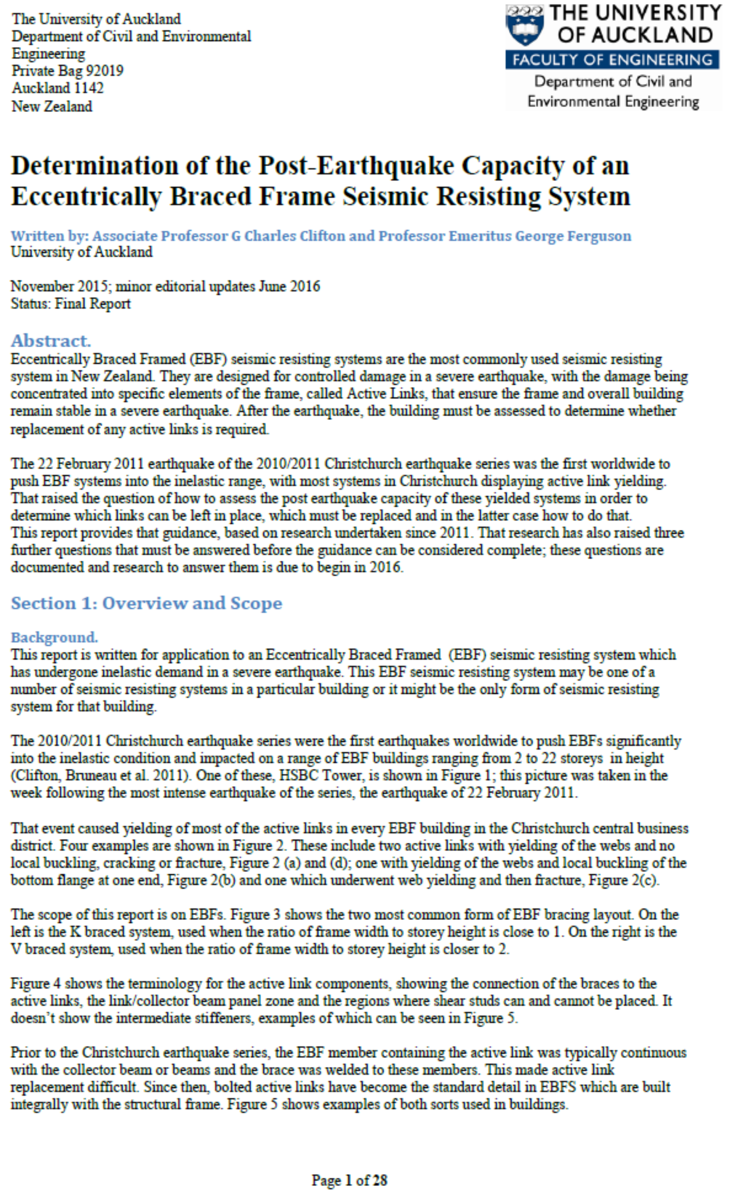
- EBF active links are designed and detailed to deform plastically in shear in a severe earthquake
- After earthquake how to assess whether to leave in place or to replace



Relationship Between Hardness and Plastic Strain

- Initially determined by Nashid
- Procedure developed and applied to several buildings post earthquake

However effect of strain ageing on link performance not quantified



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Determination of the Post-Earthquake Capacity of an Eccentrically Braced Frame Seismic Resisting System

Written by: Associate Professor G Charles Clifton and Professor Emeritus George Ferguson
University of Auckland

November 2015; minor editorial updates June 2016
Status: Final Report

Abstract.

Eccentrically Braced Framed (EBF) seismic resisting systems are the most commonly used seismic resisting system in New Zealand. They are designed for controlled damage in a severe earthquake, with the damage being concentrated into specific elements of the frame, called Active Links, that ensure the frame and overall building remain stable in a severe earthquake. After the earthquake, the building must be assessed to determine whether replacement of any active links is required.

The 22 February 2011 earthquake of the 2010/2011 Christchurch earthquake series was the first worldwide to push EBF systems into the inelastic range, with most systems in Christchurch displaying active link yielding. That raised the question of how to assess the post earthquake capacity of these yielded systems in order to determine which links can be left in place, which must be replaced and in the latter case how to do that.
This report provides that guidance, based on research undertaken since 2011. That research has also raised three further questions that must be answered before the guidance can be considered complete; these questions are documented and research to answer them is due to begin in 2016.

Section 1: Overview and Scope

Background.

This report is written for application to an Eccentrically Braced Framed (EBF) seismic resisting system which has undergone inelastic demand in a severe earthquake. This EBF seismic resisting system may be one of a number of seismic resisting systems in a particular building or it might be the only form of seismic resisting system for that building.

The 2010/2011 Christchurch earthquake series were the first earthquakes worldwide to push EBFs significantly into the inelastic condition and impacted on a range of EBF buildings ranging from 2 to 22 storeys in height (Clifton, Bruneau et al. 2011). One of these, HSBC Tower, is shown in Figure 1; this picture was taken in the week following the most intense earthquake of the series, the earthquake of 22 February 2011.

That event caused yielding of most of the active links in every EBF building in the Christchurch central business district. Four examples are shown in Figure 2. These include two active links with yielding of the webs and no local buckling, cracking or fracture, Figure 2 (a) and (d); one with yielding of the webs and local buckling of the bottom flange at one end, Figure 2(b) and one which underwent web yielding and then fracture, Figure 2(c).

The scope of this report is on EBFs. Figure 3 shows the two most common form of EBF bracing layout. On the left is the K braced system, used when the ratio of frame width to storey height is close to 1. On the right is the V braced system, used when the ratio of frame width to storey height is closer to 2.

Figure 4 shows the terminology for the active link components, showing the connection of the braces to the active links, the link/collector beam panel zone and the regions where shear studs can and cannot be placed. It doesn't show the intermediate stiffeners, examples of which can be seen in Figure 5.

Prior to the Christchurch earthquake series, the EBF member containing the active link was typically continuous with the collector beam or beams and the brace was welded to these members. This made active link replacement difficult. Since then, bolted active links have become the standard detail in EBFS which are built integrally with the structural frame. Figure 5 shows examples of both sorts used in buildings.

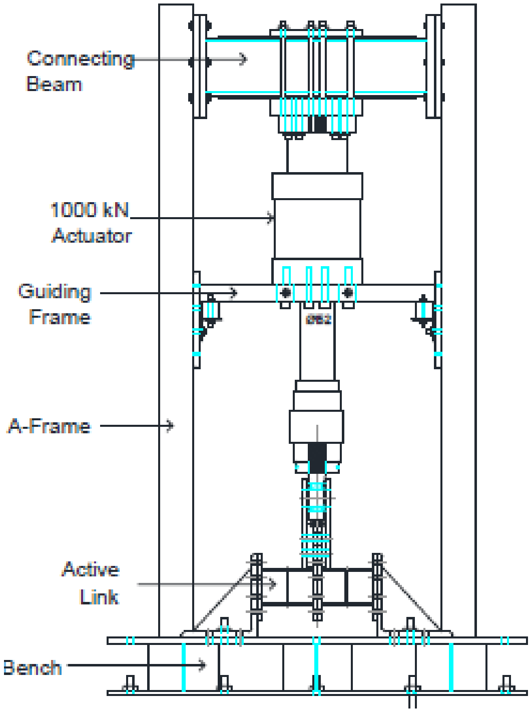
Current Research Answered Following Questions

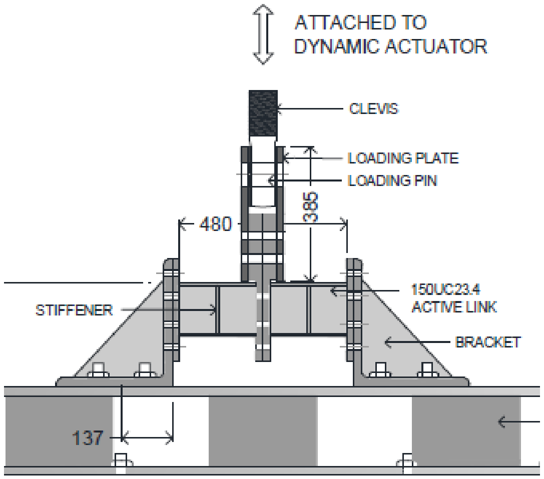
Q1 What is the effect of strain ageing on active link hardness and strength increase in subsequent cyclic loading

A1 Negligible; after first cycle of subsequent plastic loading, performance same as first time around

Q2: How conservative are the Cumulative Plasticity Limits proposed by Nashid?

A2: By approx a factor of 2





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Enhanced Seismic Resilience of Pallet Racking Systems

Zhenghao Tang and James Maguire, PhD students, second to last year of research

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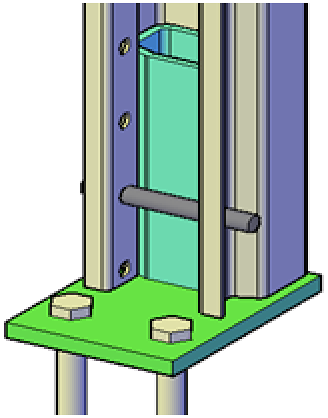
Overview

- Storage rack collapse has been observed in all recent major earthquakes
- We developed Friction Sliding Baseplate that significantly increases seismic performance
- Adds negligible additional cost to the system
- Will achieve no-damage performance during design level earthquakes and increase the collapse level earthquake intensity



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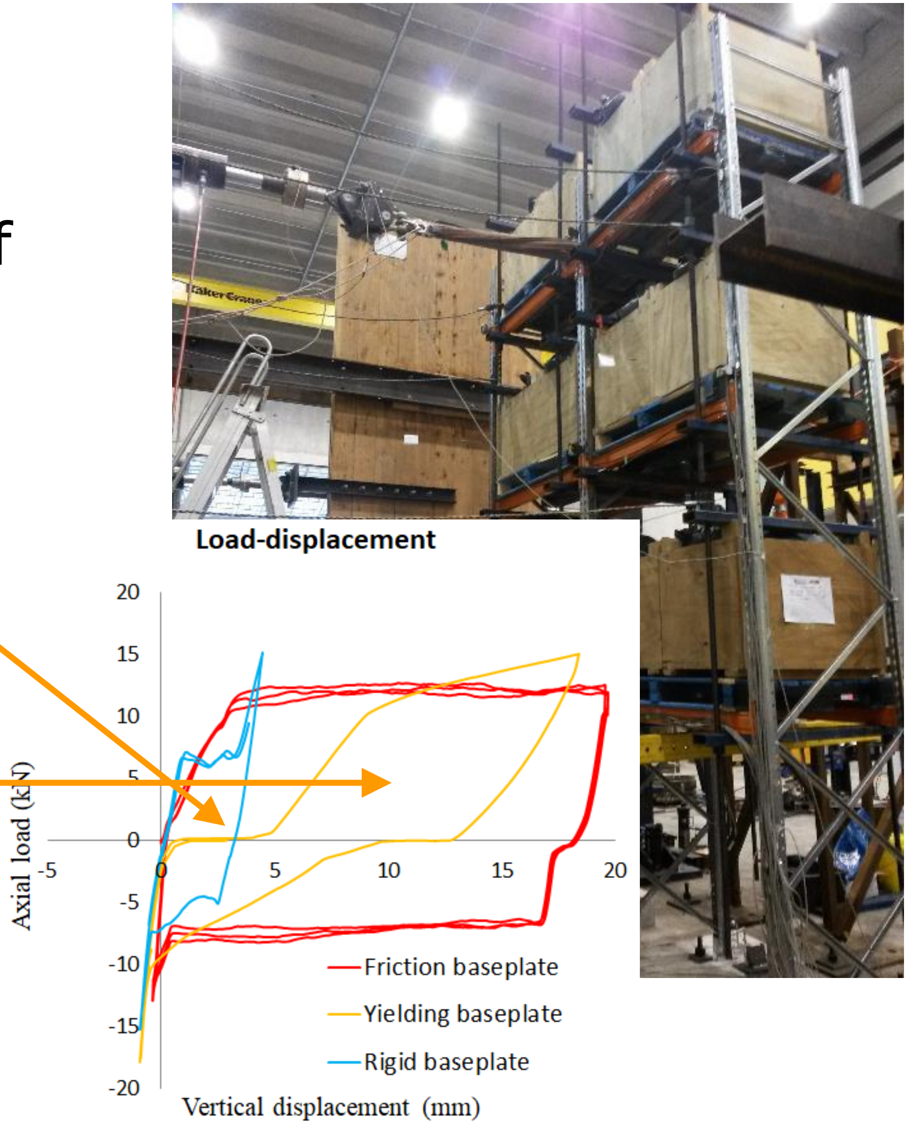
Current progress

Testing

- Component tests and pushover/snapback testing of 4.8m rack complete

Current results

- Energy dissipating capacity 890% higher than standard baseplates and 215% higher than ductile baseplate
- Increase in system damping from 5-7% to over 20%



Load-displacement

Upcoming

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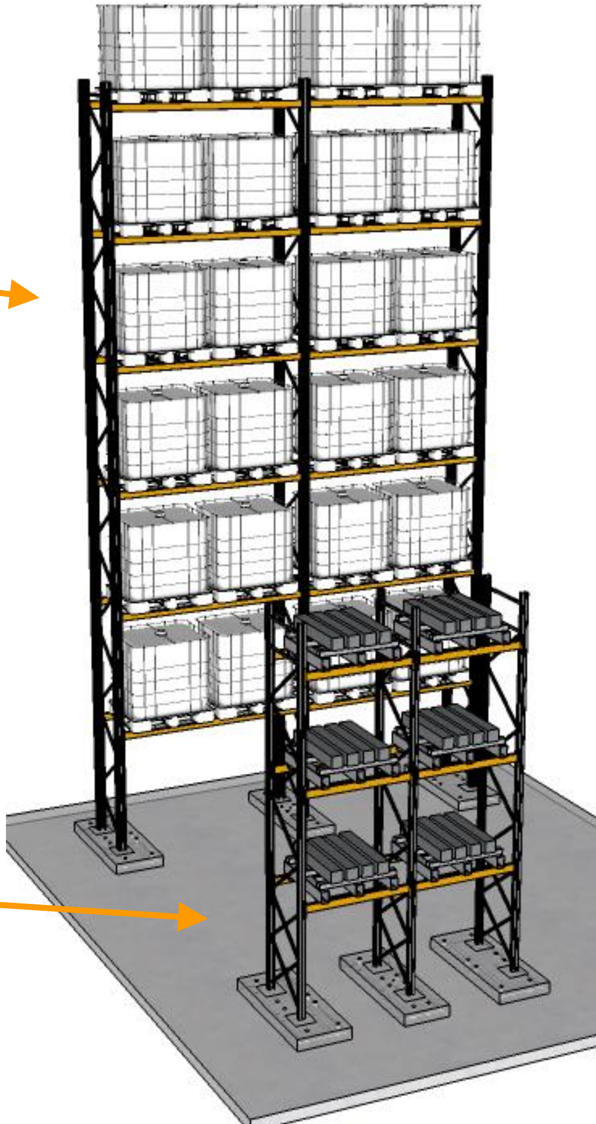

June-July 2017

- Full-scale 10.5m tall rack snapback test using the Friction Slipper Baseplate to validate performance for commercial application

July 2017

- Commercial application proposed for Wellington warehouse

July-September 2017

- Full-scale 4.8m tall rack shaking table test *to collapse*

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Thank you and Questions