Standardised timber moment-resisting frames for multistorey buildings

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ABSTRACT

The paper presents preliminary findings of research and experimental testing on prototype large-scale moment-resisting timber frames. The objective of this research was to provide standardised moment-resisting timber frame connections to support the increased application of timber in multistorey structures in NZ (akin to the 'Steel Connect' guide by SCNZ).

The project was led by Red Stag Timberlab with support and funding by Callaghan Innovation. Enovate provided structural engineering design/detailing services for several prototype internal beam-column joint subassemblies, consulted on the experimental test set-up, apparatus, loading protocol and preliminary findings. Experimental testing on the sub-assemblies was performed by BRANZ.

The prototype sub-assemblies incorporated either Glue-laminated (Glulam) or Laminated Veneer Lumber (LVL) beams/column elements, and capacity-designed connections consisting of ductile steel plastic hinges/fuses designed to suppress brittle failure in the timber elements and provide energy dissipation/damping.

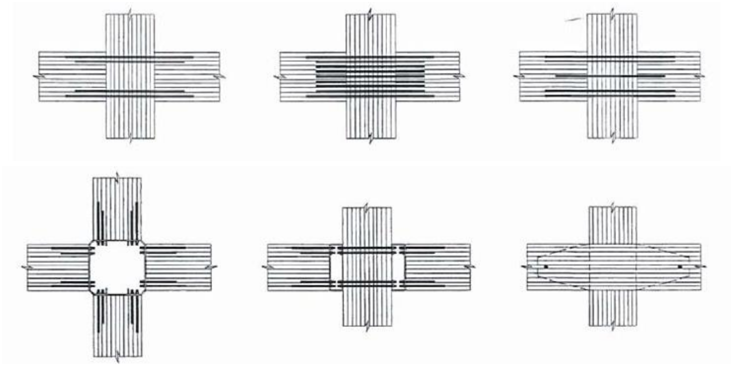
This paper presents preliminary findings from the experimental testing to-date, highlights some critical design/detailing issues (identified through experimental testing), compares predicted versus observed frame flexibly, and makes recommendations for future design and research.

1 INTRODUCTION

The application of engineered or mass timber frames as the primary gravity and/or seismic structural system in multistorey buildings is increasing throughout New Zealand and worldwide. A primary driver for this increased uptake is regulation or incentives to reduce embodied carbon in building projects/developments.

For commercial multistorey buildings, the required grid pattern/dimension between columns are typically between 8 to 9m. To resist just gravity loads for this size grid pattern, section sizes in glue-laminated timber (Glulam) or Laminated Veneer Lumber (LVL) become large and beam-column connection details can be challenging/complex; requiring careful consideration of anisotropic properties of timber (weakness perp-to-grain), the char characteristics of the timber during fire, the potential brittle failure modes due to connection detailing, aesthetics and acoustics, creep etc. Add into the mix the requirement for the timber frame to provide resilient lateral resistance under seismic loading, and the design challenge increases greatly. Designers are typically concerned by the lack of the lack of robust/comprehensive design guidance, lack of experimental testing and validation, the complexity of detailing (required to satisfy many performance/loading criteria), the inherent flexibility of moment-resist timber connections, and the risks associated with an alternative solution (non-prescriptive) compliance pathway. This is the likely reason that large timber moment frames are placed in the 'too-hard basket' for most projects.

The structural engineers or researchers that have designed seismic-resistant large-scale moment-resisting timber connections (in New Zealand) have come up with a broad spectrum of details/solutions; none of which one would consider standard or broadly applicable. In the 1990's, Buchanan and Fairweather (1993) designed and tested several beam-column joint details for Glulam frames (see Figure 1a), some of which exhibited brittle failure modes (see Figure 1b). This research and the connection details where subsequently referenced by the Timber Design Guide (Buchanan 2007), but this did not appear to lead to significant uptake by designers.



a)

b)

Figure 1. Beam-column joint details tested by Buchanan and Fairweather (1993): a) Beam-column joint details, b) Brittle failure mode of continuous column with epoxied rods

In 2005, research commenced at the University of Canterbury on post-tensioned timber frames (termed PresLam). This research (Buchanan et al. 2008; Newcombe 2012; Palermo et al. 2005) provided a novel approach for providing moment-resisting timber connections, which has been applied to several buildings within New Zealand (and overseas). But, challenges remain with complexity of the seismic design process and connection detailing, the potential for long-term creep induced post-tensioning loses, the requirement for specialist subcontractors during construction (for post-tensioning), complications for fire performance/design. Hence, the application of PreLam moment-frames since 2005 has been limited, when compared to growth of steel moment frames during the same period.

In 2020, NZ Wood provided some guidance on the seismic design of timber structures (Smith 2020) (including moment frames) and the design/detailing of post and beam timber construction (Oliver and White 2020). These documents summarize the 'state-of-play' for timber moment frames in New Zealand. The only large-scale moment resisting timber frames cited/referenced in these documents utilize post-tensioned connections. Further, international publications or applications of large timber moment frames for seismic conditions are sparse (Rebouças et al. 2022).

Over the past few years Enovate has developed large-scale ductile moment resisting timber frames that build on the research by Buchanan and Fairweather (1993) and do not require post-tensioning. These connection details were developed for two projects Arawa Street (for Rotoma No. 1 Trust) and the Otago Polytechnical Trades Training Centre (He Toki Kai Te Rika), illustrated in Figure 2a and Figure 2b. Both projects incorporate a single bay moment frame with an overall span of 12 meters (at external beam-column joints on each side), with a spacing of 4.6 meters and 6 meters respectively.

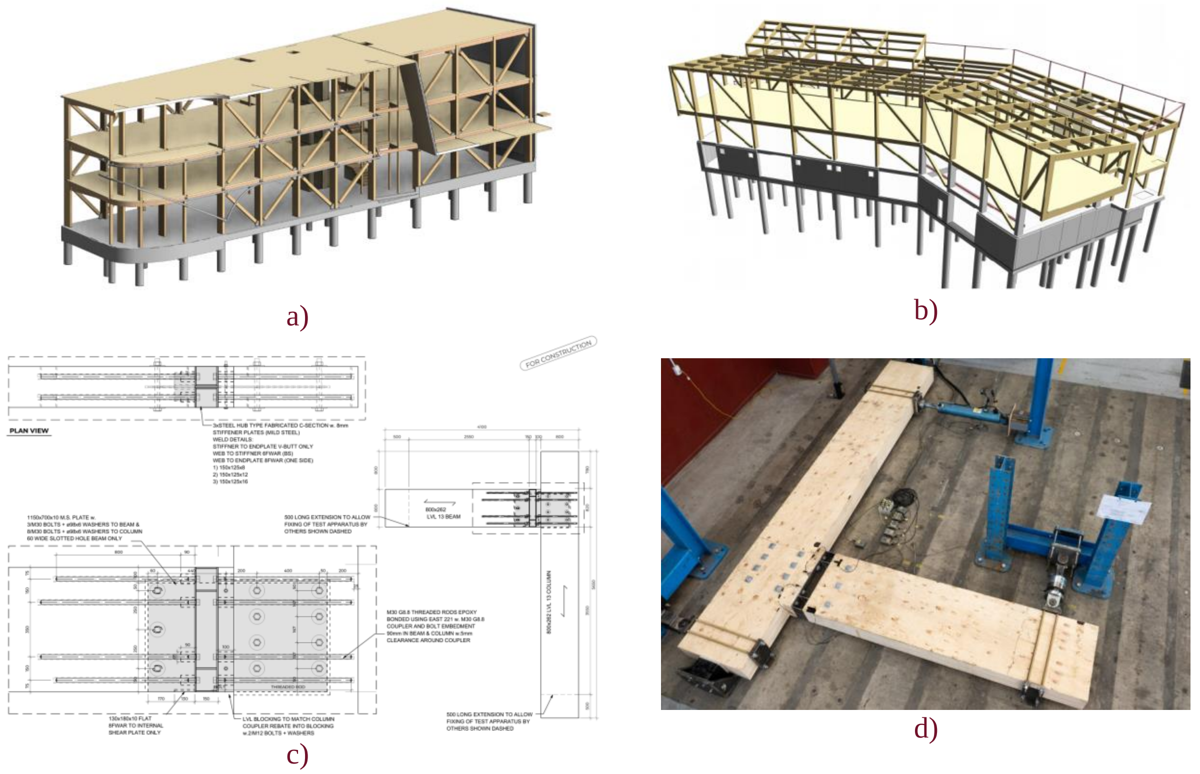


Figure 2. External Beam-Column Joint Hub Connection: a) Arawa Street Project, b) Otago Polytechnic Trades Training Centre Project, c) Test Specimen (c.o. Red Stag Timberlab), and d) Test set-up

It was recognised that experimental testing of the external beam-column connection detail was required to validate the design. The detailing of the external beam-column subassembly is shown in Figure 2c. The prototype was manufactured by Red Stag Timberlab and was tested at Holmes Solution's Structural Testing Lab (see Figure 2d). The experimental testing did validate design assumptions for the project; achieving well over the design bending/shear demands (without a brittle failure) within allowable displacement/drift limitations. However, it is recognised that the future application of this connection is limited due to experimental verification being limited to only external beam-column joints.

On these projects, a lot of design time was spent on connection design/detailing/verification due to a lack of guidance on standard/optimal large timber moment connections. The details are likely one-off/bespoke, are complex, costly and have heightened risk of failure (if sufficient experimental verification was not carried out). This appears to be a common theme for other engineers that have attempted to design/detail large-scale timber moment frames on other projects. Comparatively, the design and specification of moment resisting steel frames is straight forward by utilising the 'Steel Connect" guide by SCNZ (2007). Steel Connect includes a broad range of moment-resisting steelwork connections/details supported by detailed calculations and validated by experimental testing. Consequently, moment-resisting steel connections have become standardised, reducing cost/risk and increased market share of steel frame structures. The timber industry needs to follow this example to increase the uptake of large timber frame structures.

A key step in this direction began in 2021, when Red Stag Timberlab pursued and was granted funding through Callaghan Innovation to perform a R&D initiative termed "Project Skyscraper". Project Skyscraper aims to provide design/specification guidance for standardized moment-resisting timber connections (backed by experimental testing). Enovate were engaged to provide structural design/detailing of prototype internal beam-column connections/subassemblies, consult on the experimental testing, analyse test results, and assist with the development of a proprietary design/specification guide (for Red Stag Timberlab). The testing on beam-column joint subassemblies was performed by BRANZ at their Structural Testing Lab. This paper is the first publication to come from this research initiative, and subsequent papers are planned soon.

2 PROTOTYPE TEST SPECIMENS

Three different types of internal beam-column joint subassemblies were designed and tested; a continuous column with steel hubs at beam-ends (termed Continuous Column), a spliced column with beam-end and column steel hubs (termed Spliced Column) and a continuous column with internal beam-end and column gussets (termed Fish-Tail). The connection details are illustrated in Figure 3. Note: critical connection details are omitted (to protect Red Stag Timberlab I.P.) and application of similar details without experimental testing/verification is not recommended.

The design of prototype beam-column joints aimed to achieve the following key performance criteria:

1. *Rapid on-site assembly.* Beam/column connections are to be quickly bolted together on site. All steel plates, dowels, epoxied rods are expected to be factory installed. This minimises on-site labour cost and speeds up assembly.
2. *Incorporate a ductile fuse that is resilient, replaceable and allows for disassembly.* Allow for ductile steel (potential ductile elements) to be removed and replaced after an earthquake (reducing probable losses and repair time after a large earthquake). Replaceable connections also allow for disassembly, re-use/recycling of the structure (enhancing sustainability and reducing embodied carbon).
3. *Steel components to be cost-effective and standard (without compromising 1 and 2).* Use standard universal steel sections (UB's) and bolts were used where possible to limit cost and supply lead-times.
4. *Allow for fire protection/performance.* Epoxied rod connections were not considered for shear (to avoid a loss stability) and where possible steel components were detailed to sit within to beam/column section dimensions, allowing for timber covers or fire-rated linings to be applied over steel components.

For each type of beam-column joint, three sizes were designed/tested (large, medium and small). The following table summarizes all the tests/test specimen to-date. It is noted that the Continuous Column testing did not proceed beyond Test 3 for reasons cited in section 4. Only Glulam was used for the test specimen, except for Test 3 which used LVL. Glulam was considered worst-case for the risk of brittle failure in the timber sections (i.e. the material strength is lower and section is less homogenous).

Test No.	Type	Size	Material	Column size/grade	Beam size/grade
Test 1	Continuous Column	Medium	Glulam	990 x 230 GL10	900 x 230 GL10
Test 2	Spliced Column	Medium	Glulam	990 x 230 GL10	900x230 GL10
Test 3	Continuous Column	Medium	LVL	1000 x 343 LVL13	800 x 258 LVL11
Test 4	Spliced Column	Small	Glulam	720 x 230 GL10	630 x 230 GL10
Test 5	Spliced Column	Large	Glulam	1215 x 230 GL10	1125 x 230 GL10
Test 6	Fish Tail	Medium	Glulam	990 x 360 GL10	900 x 360 GL10
Test 7	Fish Tail	Large	Glulam	1215 x 360 GL10	1125 x 360 GL10
Test 8	Fish Tail	Small	Glulam	720 x 230 GL10	630 x 230 GL10

2.1.1 Continuous Column (Test 1 and 3)

This first subassembly designed was the continuous column (see Figure 3a). Between the beam-end and column-face is a steel hub; a short beam section with necked (or dog-boned) flanges. The necked flanges are designed to be the potential ductile element or fuse within the system. The degree of necking is specifically tailored to maximise the strength and efficiency of the frame while ensuring that the timber elements and other components of the connection are capacity protected. The necking design/detailing follows Eurocode 8 (1996), similar to provisions from ANSI/AISC 358 (2022) for reduced beam sections

The hub is connected to the column-face and beam-end with a single row of two epoxied rods top and bottom (to resist flexural actions). An internally slotted plate with mild steel dowels at the centre of the beam-end is provided to resist shear. The hub is bolted to couplers at the ends of the epoxied rods and tapped holes in the shear bracket (that allows it to be removeable). A block of timber is provided on each side of the column to house the large couplers and shear bracket recess to prevent a large notch/void in the column face.

The Test 3 specimen was designed after Test 1 was complete. The design of the Test 3 specimen attempted to avoid the column fracture observed during Test 1 (see section 4) by using LVL and by increasing the column section size/modulus.

2.1.2 Spliced Column (Test 2, 4 and 5)

The second prototype subassembly designed was the Spliced Column (see Figure 3b). A short steel column section is used to splice the timber column together. Removeable beam hubs are used as the fuse (like the Continuous Column). Three rows of two epoxied rods are provided to the column-ends. The outer rows are to resist flexural actions, while the middle row provides increased tension capacity and additional minor axis flexural strength (to avoid buckling). Two options for the column shear connection were explored; the first incorporated slotted cleats with mild steel dowels (applied to the medium specimen) and the second incorporated bearing plates at the edge of the column (used on the large and small test specimen). The latter option is preferred as it is easily scalable for different column shear demands. The steel beam-column joint is designed to NZS3404 (1997), using SCNZ connections (2007) where possible. Note: for Test 2 the joint panel region was not reinforced with doubler plates to NZS3404 (see section 4). This was not the case for Test 4 and 5.

2.1.3 Fish Tail (Test 6, 7 and 8)

The third prototype subassembly was the Fish Tail (see Figure 3c). The column is made in two pieces and block-glued around the column gusset plate (to avoid welding of gusset tabs plates near the timber column). The column and beam-end gussets are fixed into the timber around the perimeter using mild steel dowels, and bolted in the centre (to provide clamping in the minor axis of the beam/column and prevent splitting).

The column/beam gussets are connected to fuses at the top/bottom of the beam to resist flexural action and bolts (in slotted holes) at the centre to resist shear. The fuses are dog-bone shaped mild steel plates, designed to yielding in tension and compression. Clamping/anti-buckling plates are provided each side of the fuses. The clamping plates are slotted on one side to avoid contribution to the flexural strength of the connection. Note, any fuse could be used within this connection arrangement including friction-slip plates or Tectonus devices etc.

The mild steel dowels fixing the gussets resist overstrength moment/shear and axial forces using a Rivet Group Analogy based on equations from the Timber Design Guide (Buchanan 2007) and anti-splitting screws are provided to beam-ends in accordance with recent guidance (Moroder and Smith 2020).

After analysis of the Test 6 results, it was identified that the slop in the bolted connections for the dog-bone steel plates significantly contributed to the flexibility of the subassembly (see section 4). Hence, for Test 7 and 8, the dog-bone plates were welded in place to the gusset tabs.

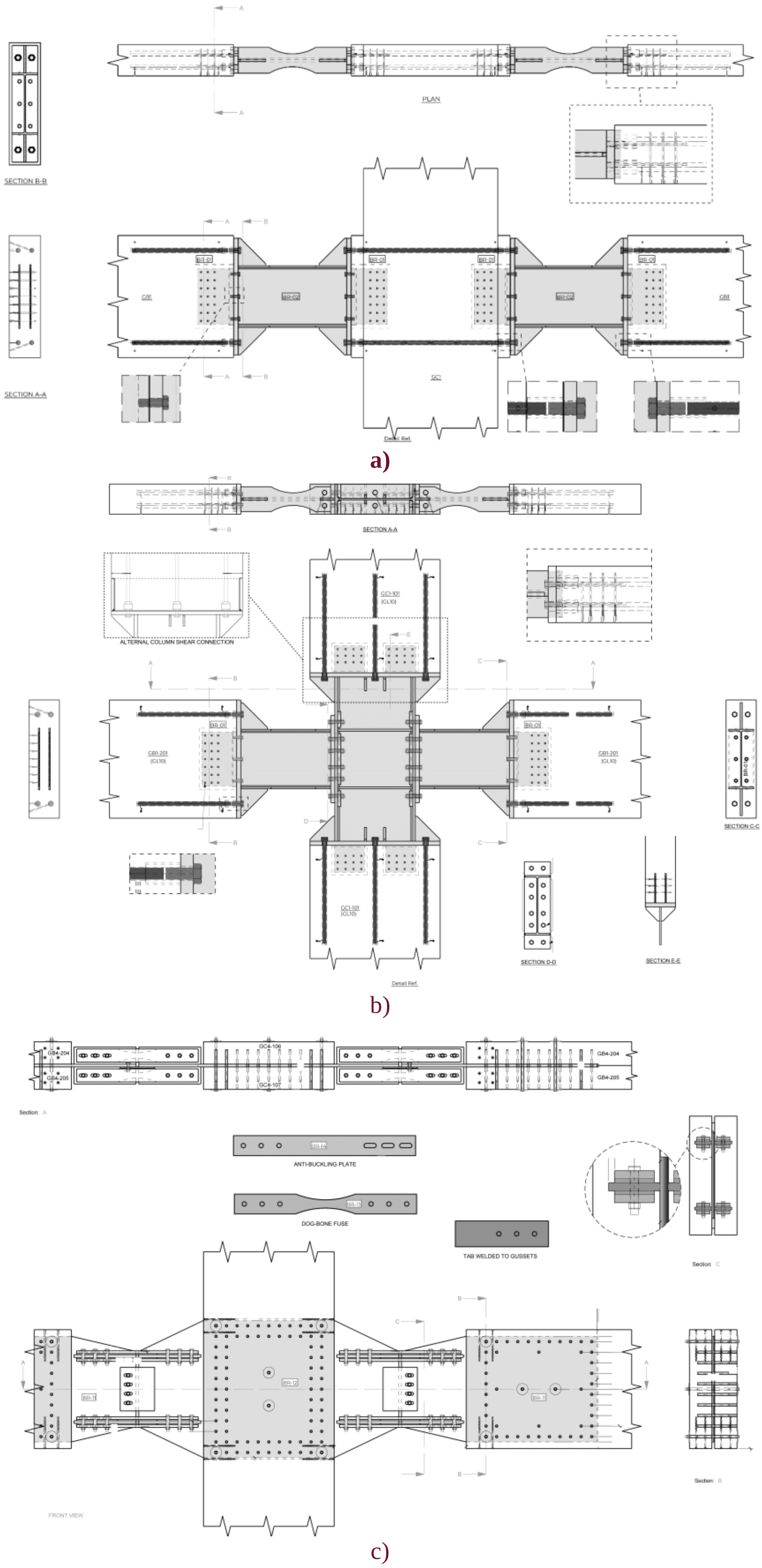
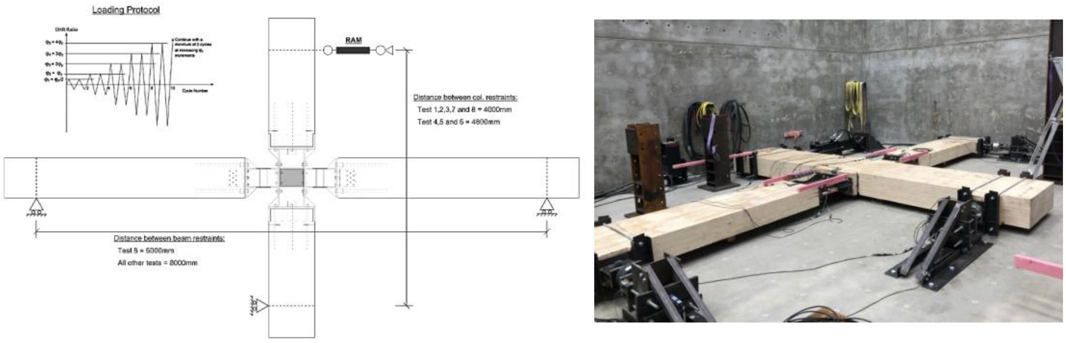


Figure 3. Red Stag Timberlab shop drawing models for three types of internal beam-column joint details tested (under Project Skyscraper): a) Continuous Column, b) Spliced Column, c) Fish-Tail

3 TEST SETUP AND LOADING PROTOCOL

Quasi-static loading was applied to the internal beam-column joint subassembly at the top of the column in accordance with the ACI 374.2R-13 (ACI 2013) displacement protocol (see Figure 4a and Figure 4b). Potentiometers were placed around the steel to timber interfaces to allow assessment of connection slop (see Figure 4c), over the ductile fuses and at all reaction points.



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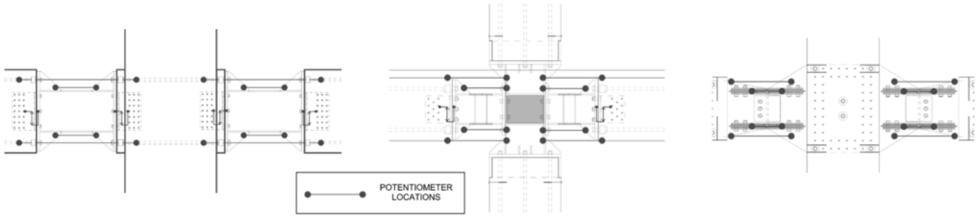


c)

Figure 4. Test set-up and loading protocol: a) Loading protocol and beam/column reaction points, b) Image of test setup, c) Beam-column joint potentiometers.

Some irregularities in the test set-up/loading protocol are listed below:

- The distance between column/beam reactions did not match the prototype design. The distance between column reactions was 4.8m (not 4.0m as designed) for Test 4/5/6, and the distance between beam reactions for Test 4 was 8m (not 6m as designed). While this did not affect the validity of the experimental response/data, initial predictions of the yield displacements (used for the loading protocol) were underestimated.
- There were two beam reaction failures for Test 7. First, the steel components providing the tension reaction failed, then the column-base lateral restraint failed. These failures potentially effected the small cycle test data (that indicates the initial stiffness); this is still being investigated.

4 EXPERIMENTAL RESULTS

The hysteretic response for Tests 1 to 8 is shown in Figure 6 and Figure 7. The primary axes are column moment (at the centreline of the beam) versus column drift (between column restraints). The secondary axis gives the ram force versus ram displacement at the top of the column. Key experimental observations are summarized in the table below and in Figure 5.

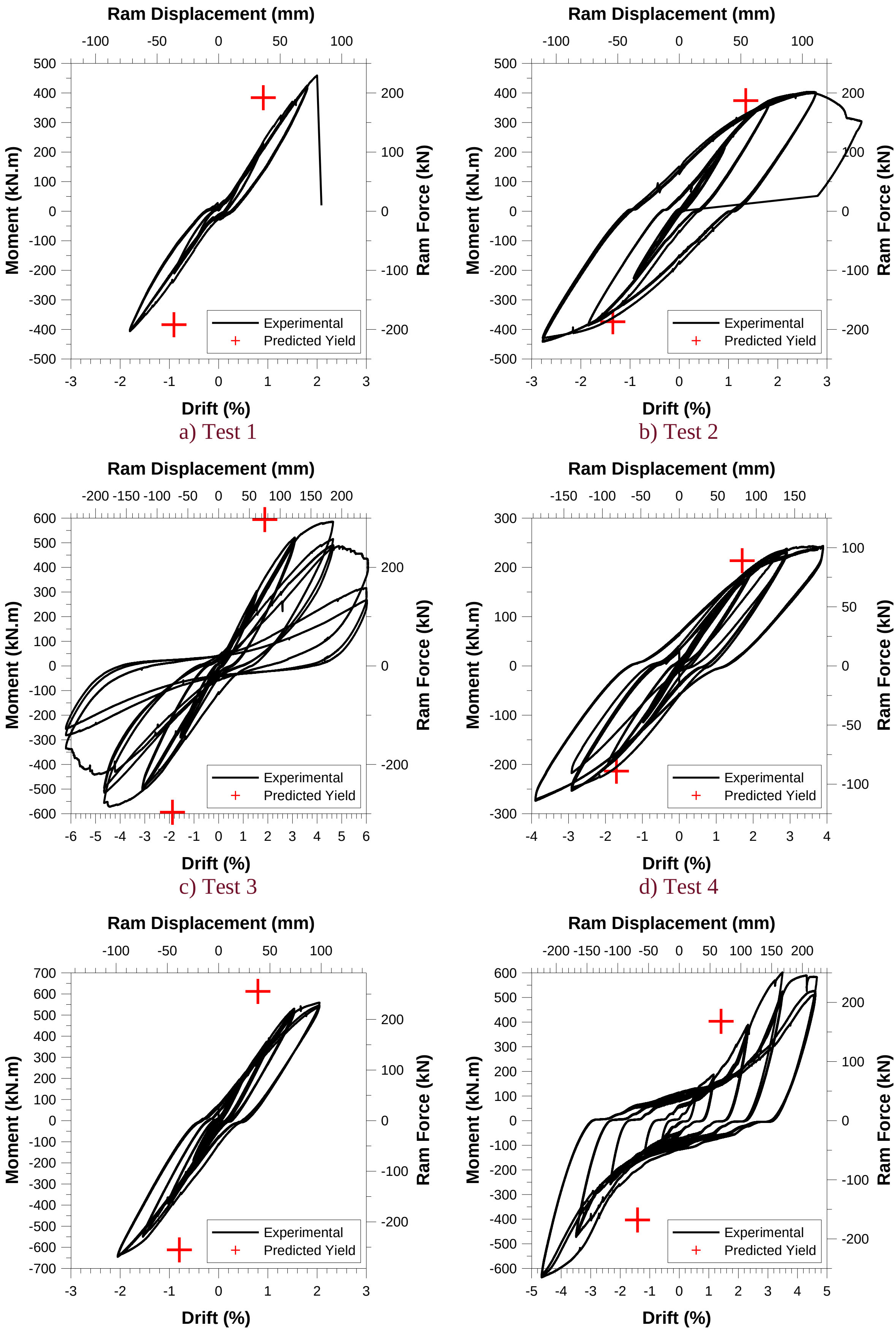
Test No.	Type/Size	Key Observations
Test 1	Continuous Column/ Medium	Test 1 exhibited essentially linear-elastic response until a sudden brittle failure/fracture of the column occurred (just above the predicted yield force/moment). The failure appeared to be in a flexural mode, but the flexural demands on the column were well within the design flexural capacity. See Figure 5a, where the fracture is highlighted. See section 5.2 where this failure is discussed further.

Test 2	Spliced Column/ Medium	Test 2 exhibited a non-linear Ramberg-Osgood style hysteresis (typical for ductile steel structures), but with a high portion of elastic deformation. During the first cycle beyond 3% drift, the dog-boned flange of the beam hub exhibited lateral torsional buckling (see Figure 5b). Yielding of the joint panel region was observed below the predicted yield force due to the lack of double plates (required by NZS3404 but omitted for the design of the specimen). This is expected to have increased the ductility and hysteretic damping of the test.
Test 3	Continuous Column/ Medium	Test 3 began to exhibit non-linear behaviour at low levels of ductility, then quickly began to lose strength as the column exhibited a brittle failure in a similar manner to Test 1 (see Figure 5c).
Test 4	Spliced Column/ Small	Test 4 exhibited a similar response to Test 2, but with less hysteretic area/damping (potentially because the joint panel region did not yield at low force/displacement like Test 2). The level of ductility achieved during the test was limited due to the distance between beam reactions being greater than intended (see section 3).
Test 5	Spliced Column/ Large	Test 5 exhibited a similar response to Test 4, but ductility was more limited due to increased flexibility of the frame (see section 5.1.2) as well as the incorrect column reaction distance being used (see section 3).
Test 6	Fish Tail/ Medium	The hysteretic response for Test 6 was pinched but appeared to have a high friction component (resulting in high hysteretic damping/area). The pinching was primarily due slop in the bolted connections for the dog-bone steel plates. Slop could be easily eliminated by modifying the fixing details of the fuse to the gusset tab or by using friction-slip or other proprietary devises. Future small-scale testing/investigation on suitable fuses between gusset tabs is recommended. The friction component was potentially generated by the anti-buckling plates, observed to undergo minor axis bending during the testing (see Figure 5d). The dog-bone plates to yield in tension and compression (see Figure 5e) at large drifts.
Test 7	Fish Tail/ Large	The Test 7 hysteresis exhibited less pinching than Test 6 as the dog-bone plates were welded to the gusset tabs. However, the dowels around the perimeter of the gussets are expected to contribute some slop/pinching. Due to experimental issues/error (see section 3), the initial stiffness is expected to be under represented.
Test 8	Fish Tail/ Small	The Test 8 hysteresis exhibited low levels of pinching and high levels of energy dissipation/damping. Displacement cycles proceeded to an approximate ductility of 4, when dog-bone plates fractured due to low-cycle fatigue and vertical column splitting occurred at the corner of the column dowel pattern (see Figure 5f).

a) Test 1 b) Test 2 c) Test 3

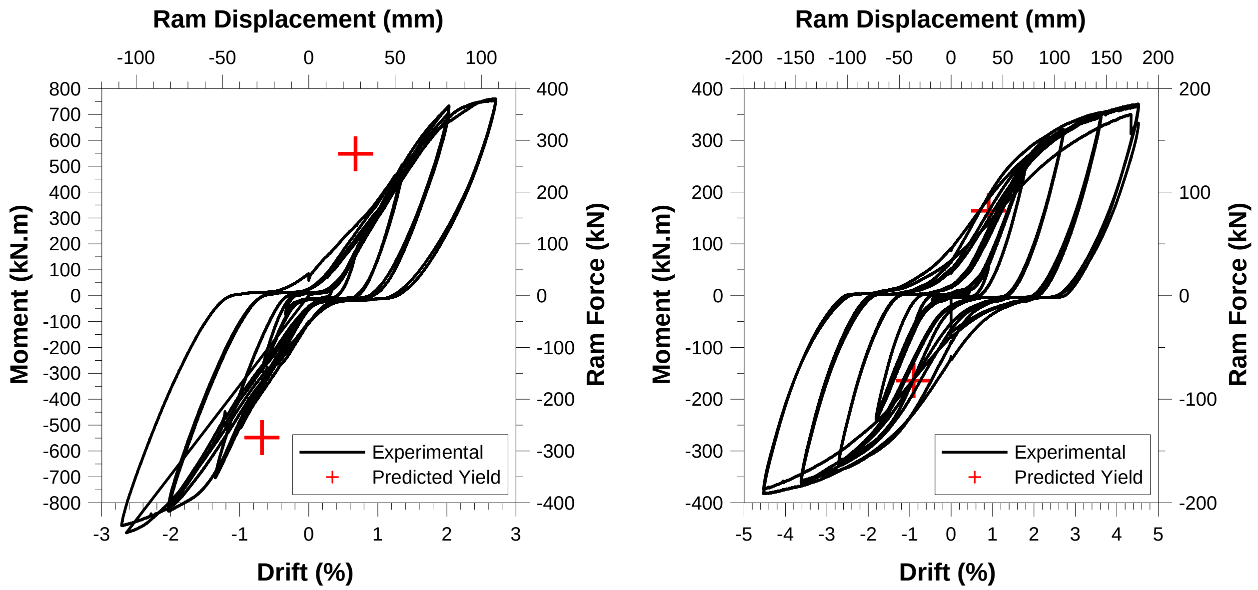
d) Test 6 e) Test 6 f) Test 8

Figure 5. Key experimental observations for Test 1, 2, 4, 6 and 8



a) Test 1 b) Test 2 c) Test 3 d) Test 4 e) Test 5 f) Test 6

Figure 6. Lateral ram force versus displacement hysteresis plots for Test 1 to 6.



a) Test 7 **b) Test 8**

Figure 7. Lateral ram force versus displacement hysteresis plots for Test 6 to 8.

5 EXPERIMENTAL ANALYSIS

Limited experimental analysis has been completed to-date and further research is required. However, key aspects that have been investigated include the predicted versus actual frame flexibility, the brittle failure of the Continuous Column test specimen, the lateral buckling of the dog-boned flanges in the beam hubs and need for column reinforcing screws for the Fish Tail connection.

5.1 Frame flexibility

Analytical equations by Newcombe (2012) were used as the basis to be predict the drift for each test specimen at the anticipated yield point of the fuse. For each type of subassembly, different assumptions were made to predict contribution of the drift components described in Equation 1:

$$\theta = \theta_b + \theta_c + \theta_j + \theta_{con} \tag{1}$$

where θ_b= the flexural and shear deformation of the beams; θ_c= the flexural and shear deformation of the columns; θ_j= the shear deformation of the joint panel region and θ_{con} = the beam-column connection.

Connection Type	**Drift Component Assumptions**		
	$\boldsymbol{\theta_b + \theta_c}$	$\boldsymbol{\theta_j}$	$\boldsymbol{\theta_{con}}$
Continuous Column	Included	Included based on Newcombe (2012)	Sum of 1mm slop/strain penetration assumed for all epoxy rods in tension or compression
Spliced Column	Included	Ignored, determined to be insignificant	Sum of 1mm slop/strain penetration assumed for all epoxy rods in tension or compression
Fish Tail	Included	Ignored, assumed insignificant due to presence of gusset plate	Dowel deformation (to EC5 equations) of column/beam-end patterns combined, dog-bone plate bolted connection slop and elastic elongation.

5.1.1 Continuous Column (Test 1 and 3)

The predicted frame stiffness is too high relative to the experimental results. This could be due to significant strain penetration of the epoxied rods in the column section that is not accounted for in the analytical equations (i.e. greater than 1mm per rod). This is supported by preliminary review of potentiometer data (at the connection interface) which indicates up to 2.5mm of slop/strain penetration per rod. The predicted yield force/moment appears to be relatively accurate but premature column fracture does limit a more accurate assessment.

5.1.2 Spliced Column (Test 2, 4 and 5)

The predicted stiffness is fairly accurate for Test 2 and 4. However, for Test 5 the predicted stiffness is too high. Preliminary investigation of experimental data indicates that connection deformation on the right beam-end to column connection was significantly higher than the left beam-end connection, indicating a potential anomaly in the fixings or detailing. The predicted yield force/moment appears to be relatively accurate for Test 2 and 4. However, Test 5 was terminated too early to give a clear indication of the post-yield response.

5.1.3 Fish Tail (Test 6, 7 and 8)

The predicted frame stiffness is too high for Test 6 and 7, but is more accurate for Test 8. One key difference between Test 6/7 and 8 is the column width. This could indicate that rolling shear deformation of the columns induced by the central gusset connections could significantly contribute to frame flexibility but more investigation is needed. The predicted yield force/moment appears to be low for all tests. Therefore, it is anticipated that there are other contributions to the bending strength of the beam to column gusset connections. Potentially minor axis bending of the fuse and anti-buckling plates resulted in friction forces. Further investigation is needed (including coupon tests of steel used for the fuses).

5.2 Continuous column failure

Tests 1 and 3 both experienced a brittle failure of the column. The failure appeared to be a tensile failure of the Glulam/LVL parallel to grain that propagated from the ends of epoxied rods in the column. This failure mechanism appears to be similar to that exhibited during testing by Buchanan and Fairweather (1993) shown in Figure 1. After Test 1, a Finite Element Analysis (FEM) of the column section was undertaken to identify why this may have occurred. The FEM model evaluated the tension stress in the column section due to both flexural demands and the localized development of the epoxied rods. The peak tensile stresses were compared to another FEM model which only considered flexural stresses (no epoxied rods).

This FEM study will form part of future publication(s) but the results demonstrate that vertical/tensile stresses in the column are greatly increased due to the ‘bursting stresses’ from the epoxied rods (by a factor of 2.5 in this case). This may be due to the low strength/stiffness of perpendicular to grain timber (that requires the parallel to grain timber to resist bursting stresses). The peak tensile stress of 41MPa was evaluated; far in exceedance of the characteristic bending strength of GL10 Glulam (22 MPa). **Caution is advised designers that might be using similar details.**

5.3 Beam hub lateral torsional buckling

Test 2 resulted in a lateral torsional buckling in the dog-boned flange of the beam hub (despite the dog-bone details complying with EC8). SCNZ published guidance for reduced beam sections (Cowie 2010) highlights that strength is gradually lost due to buckling of the reduced beam sections, but gives no advice on mitigating this issue. On further review, ANSI/AISC 358 (2022) does require lateral (fly) bracing on either side of dog-boned flange and is recommended as best-practice moving forward for these connections. For tests 4 and 5, two full depth flange stiffeners were positioned at third points within the reduced beam section to provide some degree of restraint to the critical flange (as fly braces could not be readily installed) and buckling of the beam hub was not observed.

5.4 Column screw reinforcing for the Fish Tail connection

Tests 6, 7 and 8 were performed without screw reinforcement at the corner of the column dowel pattern. This was in accordance with recent design guidance on reinforcing timber members (Moroder and Smith 2020), which indicates the screw reinforcement is only required at the ends of timber members. However, Test 8 exhibited splitting along the length of the continuous column adjacent to the corner of dowel pattern (see Figure 5f). To supress this brittle failure mode, its recommended that column splitting screws are also provided in accordance with Moroder and Smith (2020) for future applications of the Fish Tail connection.

6 CONCLUSIONS

Preliminary findings of research and experimental testing on standardised large-scale moment-resisting timber frames have been presented. Three types of beam-column connections were designed and tested with varied results.

The Continuous Column Prototype (with epoxied rods through the width of the column) exhibited an unexpected brittle failure in the column joint panel region. This failure is expected to be due to bursting stresses created by the epoxied rods into the column, combining with flexural stresses. Caution is advised to any designers considering or specifying similar connection details.

The Spliced Column Prototype exhibited a stable and ductile seismic response, and suppressed any brittle failure in the timber sections. Suggestions are made for the design of reduced flanges on beam hubs, to limit lateral buckling.

The Fish Tail Prototype exhibited a stable and ductile seismic response. Minor modifications to the original design were required to limit connection slop in the system and reinforce the column section (at the corner of dowel patterns).

This research represents a significant contribution to the standardisation of large-scale moment resisting timber frames in New Zealand.

7 FURTHER RESEARCH

The experimental testing described herein requires more in-depth analysis. Further, the authors intend to publish a design guide for standardised large-scale moment-resisting timber frames with a format akin to the 'Steel Connect' guide by SCNZ and include details, specifications, and example calculations.

The adaptability of the Fish Tail Prototype is a significant advantage. Different plug-and-play potential ductile elements fixed to the beam/column gusset tabs (such as friction-slip or Tectonus devices) can be readily applied. Future small-scale testing/investigation on suitable potential ductile elements between gusset tabs is recommended.

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