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Advancing Timber for the Future Built Environment

DESIGN AND TESTING OF DUCTILE MOMENT-RESISTING TIMBER FRAME CONNECTIONS

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ABSTRACT: The use of wood products in large-scale residential and commercial construction has grown significantly in recent years. However, limitations still exist, particularly in the context of moment-resisting frames, where a lack of efficient connection systems has made design challenging. This paper presents the development of new moment-resisting beam-column connections (BCC) for large-scale timber frames. These new connections were developed using key performance targets, considering structural performance, cost, and environmental impacts. Connections were tested with glue-laminated timber (GLT) and laminated veneer lumber (LVL) beams and columns at three different scales. Using capacity design principles, ductile failure modes in the steel elements were achieved. A sample design process is presented to support the uptake and application of this new technology.

KEYWORDS: glue-laminated timber (GLT), moment-resisting frame (MRF), beam-column connection (BCC)

1 – INTRODUCTION

Timber is back on the big stage – and this time it is here to stay. After spending almost two centuries out in the cold as concrete and steel grew in prominence in the building industry, the past two decades have seen the wood products sector contribute to a major shift in construction practices. The use of mass timber in the built environment is growing for three main reasons: widespread recognition of the need for sustainable construction practices, a loosening of codified height and size limitations for timber buildings, and significant progress in the state-of-the-art of mass timber design and fabrication [1]. The WoodWorks Innovation Network now lists more than 1000 mass timber projects that are under construction or completed in North America [2].

Investigating these projects more closely shows that using wood for gravity framing is already quite common. For example, cross-laminated timber (CLT) can be used for floor panels and/or glued laminated timber (GLT) for gravity beams and columns. Another popular typology uses CLT slabs that are point-supported with GLT or laminated veneer lumber (LVL) columns. However, timber is rarely used for the lateral load-resisting system, particularly in tall buildings – only 2 of the 22 listed tall mass timber projects in North America use a timber solution for lateral load resistance.

Some buildings in Europe feature concentrically braced timber frames (e.g., the distinctive Mjøstårnet in Norway) and CLT shear wall construction has been applied in three tall buildings in Canada (Origine and Arbora in Quebec and The Hive in Vancouver) [3]. Moment-resisting frames (MRFs), on the other hand, are rarely built with timber. MRFs are inherently flexible as they do not use diagonal bracing or wall elements. Given that mass timber is less stiff than comparable steel and concrete products, this makes the execution of a timber MRF particularly complicated. Part of the challenge is the lack of robust timber beam-to-column connections (BCCs). The BCC stiffness governs the sidesway stiffness of timber MRFs [4] so it is impossible to control drifts without a stiff BCC. In addition, in timber structures, the frame elements generally have brittle failure modes in bending and shear. This means that connections are the primary ductile zones that must be designed to act as fuses, preventing catastrophic collapse of the building [5]. As a result, designing timber MRFs becomes even more challenging in seismic zones, where BCCs require a ductile failure mode and must provide the primary source of energy dissipation.

Little work has been done to investigate the performance of mass timber BCCs experimentally. Some researchers have studied dowelled or bolted connections but construction tolerances for on-site assembly require predrilled fastener holes to be oversized. This introduces initial slip and greatly reduces the initial connection stiffness [6], [7]. These problems can be addressed by using fasteners like self-tapping screws or self-drilling

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dowels which do not require predrilling [8]. Alternatively, glued-in steel rods can be used to form a fully tight connection. Another strategy to increase the connection stiffness can be found in aligning the force transfer with the wood grain. As a natural material, wood is much stronger in the direction of cell growth. If direct contact between the beam end and the column face is allowed, stresses develop perpendicular to the column grain, which can lead to large deformations. Additionally, connections with dowel-type fasteners transfer forces through embedment into the timber. The strength and stiffness can vary significantly with the angle between the bearing direction and the grain.

Ductility in timber BCCs is usually achieved with steel fasteners, brackets, or special damping devices. Dowel-type fasteners can be used, where energy is dissipated through the bending of the steel and embedment of the fasteners into the wood fibres. This is commonly done with dowel or bolt groups as in [8]. However, the hysteretic performance of this kind of connection is very pinched due to the irreversible damage done to the wood fibres as they are crushed. This leads to a low reloading stiffness. Alternatively, some researchers have introduced larger steel brackets that can deform and behave similarly to the panel zones found in steel MRFs [4], [9], [10]. Here, capacity design principles must be applied to ensure ductile failure in the steel occurs before brittle failure in the timber elements. Finally, friction or viscoelastic damping devices can be used to create resilient, damage-free connections, although connection typologies must be developed to allow their incorporation [11].

The technical challenges aside, another major barrier to the implementation of timber MRFs is the lack of design guidance. For commercial multi-storey buildings, the required grid dimension between columns is typically between 8 and 9 m. With this kind of grid pattern, the gravity loads alone require large timber section sizes for the beams and columns. Designers may be concerned that information is lacking for detailing and modelling complex connections of this size. Little is known about dealing with the anisotropic properties of the timber, its behaviour during fire, potential brittle failure modes, aesthetics, acoustic flanking of exposed columns and creep. This makes implementation in practice difficult because significant time must be spent by engineers to develop, design, comprehensively test, and verify bespoke details so that consent can be secured from building authorities. Therefore, the implementation of these systems is currently prohibitively expensive. The only large-scale moment-resisting timber frames cited in current New Zealand design guidance utilize post-tensioning [12], [13], but even this technology has not been used widely due to the complexity of the design and construction process, and issues around long-term creep-induced post-tensioning losses.

A gap remains in the state of the art, particularly when compared to the resources available in steel construction. In New Zealand, the Steel Connect guide from SCNZ provides a broad range of moment-resisting steel connection details that are supported by detailed calculations and have been validated through experimental testing. This level of standardization has reduced costs and risks, thereby increasing the market share of steel frame structures in New Zealand. A similar resource for timber design would be invaluable.

2 – PROJECT DESCRIPTION

2.1 PROJECT TARGETS

In this project, the main goal was to develop a new beam-column connection for timber moment-resisting frames and comprehensively test and verify its performance. This connection had to prioritize the aforementioned key performance goals of high stiffness and ductility along with easy ‘scalability’. In addition, the following features were considered:

- *Rapid on-site assembly*: the design of the BCC must allow for quick installation that minimizes on-site labour costs.
- *Ductile and replaceable fuse*: the subassembly must use a ductile fuse that can easily be removed and replaced after earthquakes.
- *Cost-effective, standard steel components*: standard steel sections must be used where possible to limit cost and supply times.
- *Fire protection*: detailing must consider fire protection, which precludes the use of glued-in rods for shear transfer.

2.2 INTENDED APPLICATION

The intended application of this new connection is for multi-storey mass timber buildings in high seismic regions such as New Zealand with a design life of 50 years or less. These connections are developed considering an open floor plan with a grid of up to 9x9 m and an inter-storey height of up to 5 m in typical mid-rise commercial buildings. The intended use is internal (dry) only with an in-service moisture content of less than 18%. Due to the elastic flexibility of timber moment frames, it has been anticipated from the start that the structural ductility factors which can be achieved with this typology are limited to around 3 (or less). Accurate

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evaluation of the elastic frame flexibility by the designers is essential.

The developed connections will require additional design and detailing to satisfy fire load requirements. Fire-rated linings or timber encapsulation will almost certainly be required to limit the temperature of the steel elements to 120°C for plates and 300°C for dowels and maintain gravity load stability (as required by AS/NZS1720.4). Depending on the type of epoxy that is used, glued-in rods experience significant strength loss at low temperatures around 50-75°C [14], [15].

It is the responsibility of the designer to carry out member capacity checks under combined actions. Vibration sensitivity checks on the frames and any construction loadings should also be considered.

2.3 CONNECTION CONCEPTS

Using the prescribed performance targets, nine connection concepts were developed. Next, a multi-criteria analysis (MCA) was used to choose one of these concepts for further development and testing. In the MCA, four main categories were considered and weighted based on their perceived importance: economics, environmental, utility, and market. From the chosen initial concept, three beam-column joint details were developed and tested at full-scale [16]. These three details are shown in Figure 1. The test plan is listed in Table 1.

The first concept is a continuous column detail (CC). A structural steel hub – a short beam section with necked flanges designed to be the ductile fuse – is mounted between the beams and the column faces. The necked section is designed using Eurocode 8 [17]. The flexural connections between the timber members and the steel hubs are realized using glued-in rods. Slotted steel plates with steel dowels resist shear actions and couplers at the ends of the rods make the steel hub replaceable. Two CC specimens were tested, both at medium scale (CC-M-1 and CC-M-2).

The second concept is a spliced column detail (SC). Here, a short structural steel column section is used to splice the timber column. As in the previous detail, short beam sections with necked flanges are used as the ductile fuses. As before, flexural demands in the columns and beams are transferred using glued-in rods while shear demands in the beams are resisted using slotted steel plates with steel dowels. In the column, two options were explored for the shear connection: a slotted steel plate with steel dowels (applied in one specimen) and bearing plates at the column edge (applied in the other two specimens). The second mechanism is easier to scale for different shear demands and is thus preferred. The column connections also feature an additional third row of glued-in rods to increase tension capacity and minor axis flexural strength. Three SC specimens were tested, at small (SC-S), medium (SC-M) and large (SC-L) scales.

The final concept is another continuous column detail, termed fish-tail (FT). Here, the column is block glued around a gusset plate to avoid welding of gusset tab plates near the timber column. The steel plates that are inserted in the beams and the column are fixed using steel dowels around the plate perimeter. Bolts are used at the centre of the gussets to prevent splitting by clamping the timber in the minor axis direction. Here, the ductile fuse comes in the form of necked flange plates which are bolted (in the first test) or welded (in the last two tests) to the gusset plates. These fuses yield in tension and compression and

Table 1 – Beam-column connection test matrix.

Type	Size	Specimen Designation	Material	Column Size/Grade	Beam Size/Grade
Continuous Column	Medium	CC-M-1	GLT	990 x 230 GL10	900 x 230 GL10
	Medium	CC-M-2	LVL	1000 x 343 LVL13	800 x 258 LVL11
Spliced Column	Small	SC-S	GLT	720 x 230 GL10	630 x 230 GL10
	Medium	SC-M	GLT	990 x 230 GL10	900 x 230 GL10
	Large	SC-L	GLT	1215 x 230 GL10	1125 x 230 GL10
Fish-Tail	Small	FT-S	GLT	720 x 230 GL10	630 x 230 GL10
	Medium	FT-M	GLT	990 x 360 GL10	900 x 360 GL10
	Large	FT-L	GLT	1215 x 360 GL10	1125 x 360 GL10

are restrained with anti-buckling plates above and below. These clamping plates feature slotted holes on one side to avoid contributing to the flexural strength of the connection. Splitting reinforcement screws are provided at the beam ends [13]. Three FT specimens were tested, at small (FT-S), medium (FT-M) and large (FT-L) scales, although the large specimen had to be retested due to problems with the test apparatus.

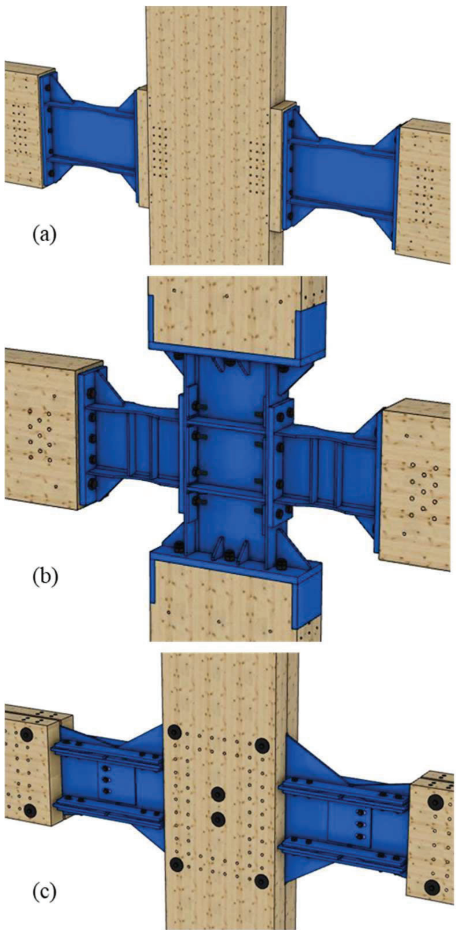

Figure 1 – 3D views for the three tested beam-column connections. (a) Continuous column. (b) Spliced column. (c) Fish-tail.

3 – EXPERIMENTAL TESTING

3.1 TEST SETUP

The three concepts (CC, SC, and FT) described above were tested quasi-statically at the BRANZ lab in Wellington, New Zealand. Reverse cyclic loads were applied at the top of the column based on the ACI 374.2R-13 displacement-controlled loading protocol [18]. All tests were conducted horizontally on the strong floor. Potentiometers were used to measure deformations at the steel-timber interfaces, ductile fuses, and reaction points.

The reference displacements for the loading protocol were calculated as predicted yield displacements. The displacement predictions considered contributions from the flexural and shear deformations of the beams (θ_b), flexural and shear deformations of the column (θ_c), shear deformation of the joint panel region (θ_j), and deformation in the connection (θ_c). Deformations in the joint panel region were ignored in the design of the SC and FT specimens due to the high stiffness of the steel joint panel region. For the connection deformation, 1 mm of strain penetration was considered for all glued-in rods in tension/compression and the deformation of the dowels in the FT concept was calculated using Eurocode 5 provisions [19]. Initial slip and elastic elongation of the necked plates in the FT specimen were also included.

Several issues were encountered in the test apparatus and loading protocol which affected some of the test results:

- The distance between the column and beam reactions (4.8 m) did not match the initial test design (4.0 m) for three of the tests (SC-S, SC-L, FT-M).
- During the FT-L specimen test, there were two reaction frame failures in the tensile beam reaction and the column-base lateral restraint.
- In the repeat of the FT-L test, the maximum capacity of the loading apparatus was reached.

3.2 TEST RESULTS & FAILURE MODES

Both CC specimens behaved in a predominantly linear-elastic manner until a brittle failure occurred in the column. The column fractured in tension parallel to the wood grain. This fracture was initiated at the ends of the glued-in rods (i.e., at the extreme fibres of the column) which indicated a flexural-type failure. The top row of Figure 2 shows the moment-drift hystereses, failure modes and the overall test setup.

All of the SC specimens showed a stable non-linear Ramberg-Osgood hysteresis which is typical for ductile steel structures. Large elastic deformations contributed to limited ductility demands. The steel fuses yielded with visible yield lines and no failure was observed in the timber elements. In the SC-M specimen, the necked steel beam failed through lateral torsional buckling. This failure mode was prevented in the other two tests using vertical stiffeners. However, the hysteretic damping in the SC-S and SC-L specimens was limited in part due to the aforementioned incorrect reaction distance (which increased elastic deformation). The middle row of Figure 2 shows the moment-drift hystereses, failure modes and the overall test setup.

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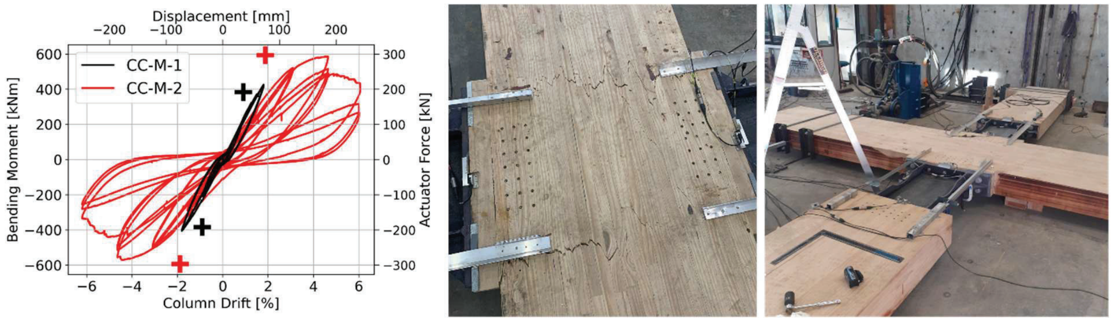
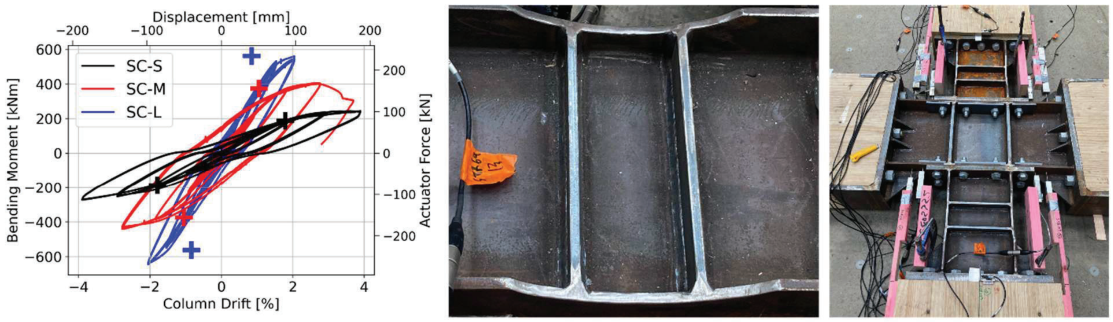
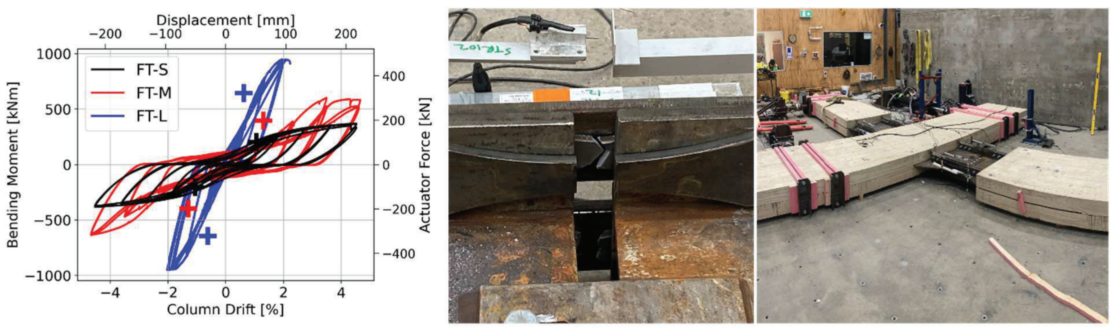


(a) Continuous Columns (CC)



(b) Spliced Columns (SC)



(c) Fish-Tail Columns (FT)

Figure 2 – Testing results. Moment-drift hystereses (left column), representative failure modes (centre column), testing subassemblies (right column). The yield moment and drift predictions are marked using + signs.

While four FT specimens were tested, only three are discussed here. The first FT-L specimen is excluded because the reaction frame failed. The other three FT specimens exhibited predominantly ductile behaviour under cyclic loading. In the FT-M specimen, all damage occurred through the yielding of the flange plates although the bolted fuses caused a significant initial slip in the connection which limited the initial stiffness and ductility, and friction contributed significantly to the hysteretic response. In the FT-S and FT-L specimens, the flange plates were welded to address this problem. Once again, these fuses yielded extensively. In the small specimen, vertical column splitting occurred at the corner of the column dowel group. In the large test, the maximum capacity of the loading apparatus was reached and the test was stopped before completion of the loading protocol. The bottom row of Figure 2 shows the moment-drift hystereses, failure modes and the overall test setup

3.3 EXPERIMENTAL ANALYSIS

Both CC (continuous column) specimens experienced brittle failure in the column. This failure appeared to be a tensile failure of the timber parallel to the grain which propagated from the ends of the epoxied rods and appears similar to a failure mode that was reported in [4]. The failure mode was surprising because the flexural demands in the columns were well within the design flexural capacity. Detailed finite element analysis in [16] demonstrated that the failure was probably the result of highly localized bursting stresses induced by the glued-in rods. These increased the tensile stresses in the regions around the rods by a factor of 2.5 (up to 41.1 MPa).

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The medium spliced column test (SC-M) exhibited lateral torsional buckling of the necked beam section, despite the details complying with Eurocode 8. [20] highlights gradual strength loss due to buckling of the reduced beam section but provides no mitigation advice. However, [21] requires lateral bracing on either side of a reduced beam section. Because fly bracing could not be readily installed, subsequent tests used full-depth flange stiffeners. These prevented buckling effectively.

While the guidance in [13] required no screw reinforcing in the timber column connections (only required at the ends of timber members), one of the fish-tail specimens exhibited vertical splitting adjacent to the corner of the dowel pattern. Anti-splitting screw reinforcement is recommended to avoid this issue in the future.

Overall, the analytical predictions for the frame stiffness were validated for the spliced column and fish-tail details. However, the yield capacity of the fish-tail prototypes was underestimated due to minor axis bending and axial restraint (via friction) from the fuses and anti-buckling plates. Future designs of the fish-tail detail should seek to limit the overstrength of the steel fuses.

4 – DESIGN AND ANALYSIS

Using the outcomes from experimental testing, a set of standardized designs has been developed. Major design revisions are described and the overall design philosophy and calculation procedures are discussed. The design procedure is explained in detail in the Appendix, where the use of the new design tables is demonstrated.

4.1 DESIGN REVISIONS/CONSIDERATIONS

Fish-Tail Continuous Column Connection

As noted in the experimental analysis, the yielding fuse for the fish-tail connection exhibited too much overstrength due to out-of-plane bending of the fuses and anti-buckling plates. A revised fuse detail is proposed that will limit the connection overstrength (see Figure 3). The fuse design is similar to the test specimen but is rotated 90° and the fused region is made uniform instead of curved. A further refinement is offered to limit the overstrength due to the interaction of the connection with the floor system. Gap opening/closing at the top of the gusset-to-gusset connection could result in significant axial restraint of the diaphragm, inducing additional strength in the frame. By introducing a steel hinge at the top of the connection between the gussets, deformation demands in the floor are reduced. However, the revised fuse will be subject to higher axial strain demands than the tested details. Therefore, a strain limit of 10% should be considered in the new detail. For the designs provided below, the following aspects should be considered:

- While section capacities are provided for the timber beam and column in the tables, designers must verify the member capacity based on the length, restraint condition and applicable section size effects
- The connection system follows a damage-avoidance design philosophy. Only the ductile fuse is expected to undergo significant inelastic deformation and all other components are expected to remain elastic. Therefore, designers shall consider the expected overstrength of any plug-and-play ductile fuse (e.g., ductile-yielding elements, friction-slip connectors or Tectonus devices). Physical testing of selected fuses is required to ensure stable cyclic response and limited overstrength.
- The dowel connections in the beam/column gussets are a significant source of frame flexibility. The dowels should be plugged, and holes in the timber shall be close-fitting following [19]. It is recommended that the holes in the gusset plate are only 1 mm oversized.
- The holes for the top-hinge steel pin shall be a maximum of 1 mm oversized.
- Perpendicular-to-grain shrinkage reinforcement shall be considered on a case-by-case basis (see [13]).

Spliced Column Connection

Based on the favourable experimental performance, no major design revisions are required for the spliced column connection. However, the following aspects shall be considered for the designs provided:

- While section capacities are provided for the timber beam and column in the tables, designers must verify the member capacity based on the length, restraint condition and applicable section size effects
- The design of the glued-in rods follows similar procedures to the beam-end connections but considers column axial forces.
- The design of the steel column section, joint panel region and steel beam end-to-column connections shall follow the relevant steel design standards.
- Due to the buckling of the reduced beam section in the spliced column connection test (SC-M), lateral bracing should be provided under [21].

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- Fire-rated linings or timber encapsulation will be required to limit the temperature of the steel plates and dowels to maintain gravity load stability. Additionally, pull-out capacity loss under elevated temperatures of the glued-in rods must be considered for post-fire stability load cases.
- The connection system follows a damage-avoidance design philosophy. Only the ductile fuse is expected to undergo significant inelastic deformation and all other components are expected to remain elastic. Therefore, designers shall consider the expected overstrength of any plug-and-play ductile fuse.
- Holes for the glued-in rods in the steel end plates have been oversized to allow for shrinkage movement up to 4 mm. This requirement can be assessed on a case-by-case basis (see [13]).

4.3 CONNECTION MODELLING

As noted in the introduction, the flexibility of timber MRFs has been one of the main design challenges in the past. While this study has helped to address part of this issue by providing a relatively stiff and efficient connection system, the flexibility of this connection must be captured and modelled accurately so that deformation limits can be achieved reliably. This section describes modelling approaches for capturing the elastic stiffness of the continuous column fish-tail connection and the spliced column connection. These approaches are based on the findings from experimental testing and analyses.

For both the continuous column fish-tail connections and the spliced column connections, the following recommendations are made for capturing the flexibility of the frame:

- Timber beam and column sections should be modelled as elastic frame elements with anisotropic material properties. Shear deformations should be captured.
- Beam and column sections should be modelled to the centrelines. Rigid or semi-rigid joint regions should not be considered.
- Beam-end nodes should incorporate a rotational spring to capture the connection and joint panel flexibility. Appropriate rotational stiffness values are provided in the design tables available from Red Stag Timberlab.

5 – CONCLUSION

In this project, the development of a novel connection system for the beam-to-column joints of moment-resisting mass timber frames has been described. A comprehensive literature review identified the current state of the art and noted that current solutions lack stiffness and ductility. Through a process of brainstorming and conceptual design, a set of potential new connection systems was developed. A multi-criteria analysis was used to choose the most promising option.

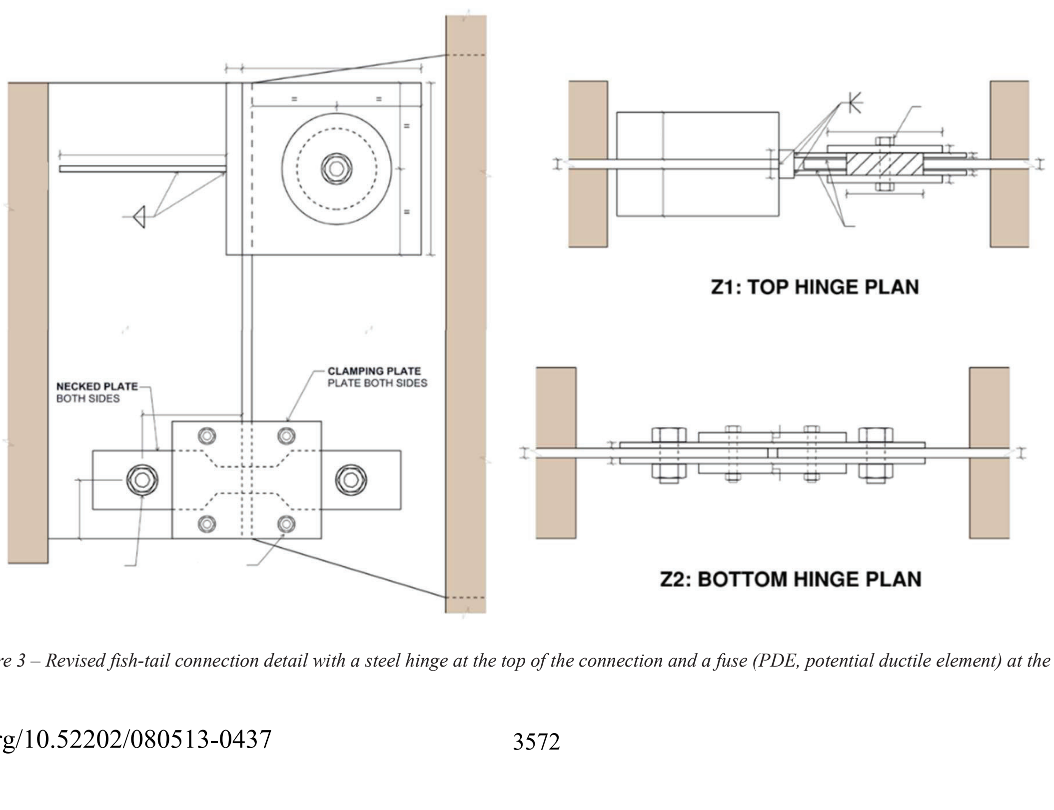

Figure 3 – Revised fish-tail connection detail with a steel hinge at the top of the connection and a fuse (PDE, potential ductile element) at the bottom.

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This connection concept was developed further into three distinct details. Each of these details was tested cyclically at different scales. The results showed that:

- A continuous column detail with glued-in rods is not recommended due to the large bursting stresses generated around the rods which are difficult to quantify and would result in significantly larger column size requirements.
- The steel fuses need to be braced sufficiently to prevent lateral torsional buckling as this inhibits the ability of the fuses to dissipate energy.
- Screw reinforcing is required to prevent splitting in the column dowel groups.

The test results were used to develop a design procedure with design tables that provide information on the frame strength and stiffness. These tables are available from Red Stag Timberlab to support the uptake of this new technology. The appendix of this article provides an overview of the intended design method. Overall, the test and analysis results show that the developed connection typology can achieve the required strength, stiffness, and ductility to allow its application in timber moment frames. Further experimental testing and investigation of the fire performance of this connection is recommended to reduce the uncertainties in its behaviour.

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8 – APPENDIX: DESIGN EXAMPLE

This appendix provides an overview of the intended use of the design method and tables that have been developed. It is important to note that the design information developed in this project is subject to change and *the most up-to-date guidance can be obtained from Red Stag Timberlab*. As usual, it is the responsibility of the design engineers to complete all necessary design checks to verify a compliant design. Specifically, designers must check the member capacities of the beams and columns because the design tables focus only on section capacities.

As noted in the main body of the article, the design manual intends to facilitate the use of this new connection technology. Overall, the use is intended to be similar to that of the SCNZ steel design guide. When designing a new building and considering material options, engineers should begin by building a simple frame finite element model in the software of their choosing. This involves choosing an expected initial column and beam depth from the provided set of details. They can use the provided design tables to find the connection rotational stiffness associated with this beam and column size as demonstrated in Table 2 and include this in the structural model as described in Section 4.3 of this study. The model can then be used to determine the design building's natural period and the design demands in the beams, columns, and joints.

Using the outputs from the structural model, the designer must then ascertain that the flexural demands in the beam fall within the bounds indicated in Table 2. For spliced column connections, the designer must also ensure that the column demands fall within the bounds of the provided M-N interaction diagram, as shown in Table 3.

Then, the dimensions of the fish-tail hinges or reduced beam sections (depending on whether a continuous or spliced column is used) can be detailed to match the model demands using simple equations provided in the design guide. All remaining connection components are fully detailed and specified in the provided tables, as shown in Table 4 and Table 5.

Finally, fully detailed connection drawings are provided that include the variables defined in the tables. A sample of this is provided in Figure 4.

Table 2 – Beam design capacities and connection stiffness.

Connection ID: SC630 - Design Capacities and Stiffness			
Beam Minimum Design Flexural Capacity	$\varphi M_{rb,min}$	[redacted]	kN.m
Beam Maximum Design Flexural Capacity	$\varphi M_{rb,max}$	[redacted]	kN.m
Beam Design Shear Capacity	φV	[redacted]	kN.m
Beam Connection Equivalent Rotational Stiffness	K_{con}	[redacted]	kN.m/rad

1: Minimum and maximum flexural capacities consider the minimum and maximum allowable flange necking respectivly to the reduced beam section

2: The design flexural capacity is taken at the reduced beam section centreline

Table 3 – Column epoxy rod connection capacity & moment axial interaction diagram.

Connection ID: SC630 - Column Epoxy Rod Connection Capacity & Moment Axial Interaction Diagram			
Column Epoxy Rod Connection Design Axial Compresion Capacity (M* = 0)	φN_{nc}	[redacted]	kN
Column Epoxy Rod Connection Design Axial Tension Capacity (M* = 0)	φN_{nt}	[redacted]	kN
Column Epoxy Rod Connection Design Moment Capacity (N* = 0)	φM_n	[redacted]	kNm
Column Shear Bracket Design Shear Capacity	φV_n	[redacted]	kN

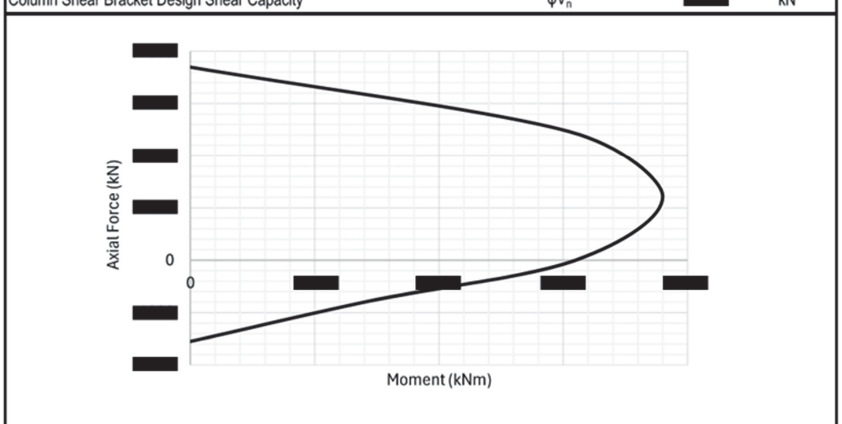

Table 4 – Connection component sizing.

Connection ID: SC630 - Connection / Member Components				
Timber Members	Beam Depth	d_b	[redacted]	mm
	Beam Width	b_b	[redacted]	mm
	Column Depth	d_c	[redacted]	mm
	Column Width	b_c	[redacted]	mm
Steel hubs	Beam hub section		530UB82	
	Column hub section		795CS	
Beam hub endplate	Depth	$d_{b.ep}$	[redacted]	mm
	Width	$b_{b.ep}$	[redacted]	mm
	Gusset height	$h_{b.g}$	[redacted]	mm
Beam shear bracket	Shear plate depth	$d_{b.sp}$	[redacted]	mm
	Backing plate depth	$d_{b.bp}$	[redacted]	mm
	Total number of dowels	n_d	[redacted]	
Column hub endplate	Depth	$d_{c.ep}$	[redacted]	mm
	Width	$b_{c.ep}$	[redacted]	mm
	Gusset height	$h_{c.g}$	[redacted]	mm
Column shear bracket	Length	$L_{c.sb}$	[redacted]	mm

The column hub section and timber column section provided above are a minimum recommendation only. Designers are to verify these using the relevent provisions of the NZS3404 and NZS AS 1720.1 respectivly.

Table 5 – Detailing parameters.

Connection ID: SC630 - Detailing Parameters				
Beam hub	Reduced beam section variable	a	[redacted]	mm
	Reduced beam section variable	b	[redacted]	mm
	Reduced beam section variable	c	[redacted]	mm
	Reduced beam section variable	g_{min}	[redacted]	mm
	Reduced beam section variable	g_{max}	[redacted]	mm
Column Hub	Length of hub	$L_{c.h}$	[redacted]	mm
Epoxy Rods	Section edge to rod centerline distance	x_b	[redacted]	mm
	Mimimum total rod embedment	L_r	[redacted]	mm
Beam shear backing plate	Shear bolt spacing	s_p	[redacted]	mm

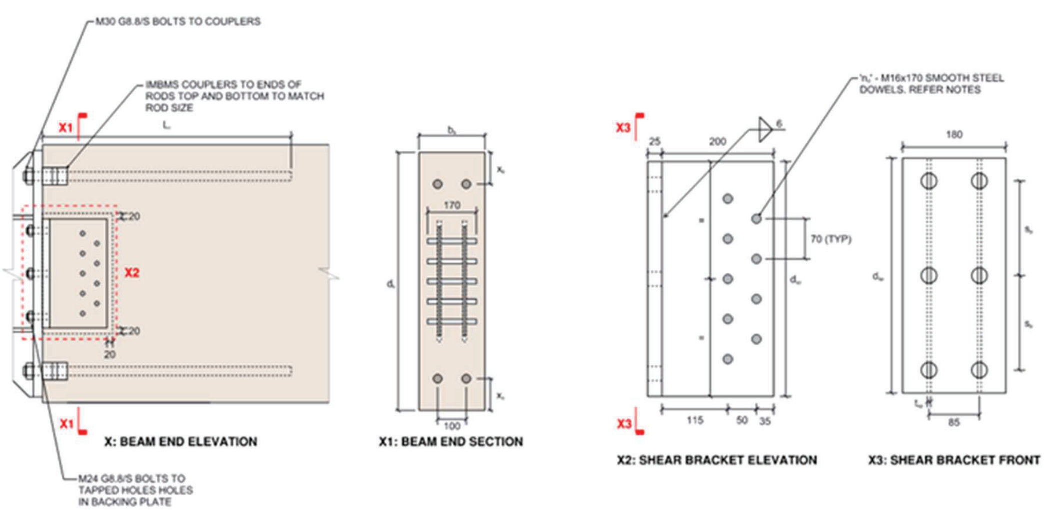

Figure 4 – Detailed connection drawing for the beam end of the spliced column connection.

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