EXPERIMENTAL COLLAPSE RESPONSE OF POST-AND-BEAM MASS TIMBER FRAMES UNDER A QUASI-STATIC COLUMN REMOVAL SCENARIO

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ABSTRACT

Mid-rise to tall mass timber buildings, which are constructed from engineered solid wood products, such as Laminated Veneer Lumber (LVL), Glued laminated timber (Glulam) and Cross Laminated Timber (CLT), have recently gained international popularity. As the height of timber buildings increases, so do the consequences of a progressive collapse event. While collapse mechanisms of concrete and steel buildings have been widely researched, limited studies have been carried out on mass timber buildings. This paper presents and discusses the experimental results performed on a series of 2D timber frame substructures, used in post-and-beam mass timber buildings and scaled down to fit the purpose of this research, under a middle column removal scenario. The behaviour of the frames and the ability of three types of commercially available beam-to-column connections and a proposed non-commercial novel connection, to develop catenary action under large deformations are reported. Furthermore, the system capacity in terms of the uniformly distributed pressure is also discussed. The test results showed that only the proposed connector was able to sustain the design pressure in international design specifications if no dynamic increase factor was considered, and therefore presented a potential solution to improve the robustness of post-and-beam timber buildings.

KEYWORDS

Beam-to-column connectors, Disproportionate collapse, Progressive collapse, Mass timber post-and-beam buildings
1. INTRODUCTION

1.1. Background

Mass timber buildings, referred to as buildings which are constructed from engineered solid wood products such as Laminated Veneer Lumber (LVL), Glued laminated timber (Glulam) or Cross Laminated Timber (CLT), are becoming popular internationally. Changes in legislations facilitate the use of timber in mid-rise constructions. In Australia, under the “deemed-to-satisfy” provisions in [1], no specific and additional design checks related to the material (such as fire safety requirements) are required to be performed, enabling timber to directly compete with steel and concrete.

Numerous examples of mid-rise mass timber buildings have already been built or are under construction, with some of the most prominent examples under 10-storey including: 9-storey Stadthaus (UK, 2009), 4-storey BskyB Building (UK, 2014), 7-storey T3 (USA, 2016), and 7-storey International house Sydney (Australia, 2017). Buildings up to 20-storey are also being or have been built, such as: 14-storey Treet (Norway, 2015), 18-storey Brock Commons Tallwood House (Canada, 2017), 10-storey “25 King St” (Australia, 2018) and 18-storey Mjøstårnet (Norway, 2019).

1.2. Progressive collapse

“Progressive collapse” is characterised by a disproportionate and catastrophic failure of a structure due to a local damage caused by an abnormal event, i.e. a low-probability high-consequence (LPHC) event such as explosions, vehicle impacts, fire, natural disasters, malicious actions and deterioration phenomena [2-5]. Although progressive collapses of buildings are rare, these events can lead to significant social and economic consequences, and must be considered, for instance in Australia, in the design of Classes 2 - 9 buildings (i.e. multi-residential, commercial, industrial, and public assembly buildings) [1]. Infamous cases of progressive collapse include those of the University of Aberdeen – Zoology Department (1966), Ronan Point Apartment Tower (1968), Alfred P. Murrah Federal Building (1995), World Trade Centre (2001) and Rana Plaza (2013). While these were concrete and steel buildings, collapse cases of mass timber buildings have also been reported in the literature [6]. Two major cases have been principally investigated, namely the Ballerup Siemens-
arena and the Bad Reichenhall Ice-arena, both long-span timber buildings. The former, located in Denmark, collapsed in 2003 due to incorrect connection designs. Two of the twelve glulam trusses collapsed without any early warning signs [7, 8]. Collapse could have further propagated if the roof purlins were continuous over multiple spans. In 2006, the roof of the Bad Reichenhall Ice-arena (Germany) collapsed, killing 15 people and injuring another 30. Several problems and flaws were revealed, including the glue-lines and finger joints of the girders which were significantly damaged by exposure to relatively high humidity. Upon failure of one of the roof box-girders, the load was transferred to the neighbouring girders which were also weakened and could not carry any additional load. This eventually triggered the progressive collapse of the building [7-9].

Current progressive collapse design guidelines [4, 10-12] cannot fully satisfy the requirements needed to consider the specificities of mass timber buildings as they are based on studies performed on steel and concrete buildings. Indeed, wood is a complex anisotropic material with brittle failure modes in bending, shear and tension, different to the more ductile behaviour of reinforced concrete and steel materials. Multi-storey timber buildings are deemed to be more elastic, typically having less rotational capacity at the joints, and limited load redistribution possibilities. Therefore, they have less potential than steel and concrete buildings to enable catenary action [13], a necessary stage to sustain high loads under large deformations [14]. Catenary action is defined as the ability of the horizontal elements to sustain enough deformation to resist the vertical loads through tensile axial forces (or catenary forces). This action is considered as the last structural defence mechanism to resist progressive collapse [15, 16]. Additionally, ductile connectors, reaching their plastic phase before brittle failure occurs in the timber elements, are deemed necessary to limit the probability of timber building collapsing [17, 18]. The ability of connectors currently used in mass timber building to develop large rotations is unknown, and little is known about the actual capacity of mass timber buildings to resist progressive collapse. The only guidelines focussing on timber buildings were solely derived from tests performed on lightweight timber constructions [13, 19], i.e. manufactured from small cross-sectional timber elements.
Limited published studies have looked at the ability of mass timber buildings to resist progressive collapse and were either theoretical [13, 20, 21] or numerical [22-24]. Experimental studies were usually limited to lightweight constructions or one-storey buildings [17-19, 25-27]. The Timber Frame 2000 (TF2000) research [19] showed that the connections are the key to ensure structural stability and robustness (i.e. the ability of a structure to sustain local damage, remain stable and not developing damage disproportionate to the initial failure) [19, 28, 29].

While post-and-beam mass timber buildings, which are the focus of the study, are currently designed in Australia under a column removal scenario, the design is based on best engineering knowledge, not on scientifically derived procedures. Therefore, there is a need to experimentally investigate the behaviour of mass timber buildings under large deformations to quantify the load transfer through the building and the contribution of each structural element in resisting the load. Such studies would potentially contribute to the development of much-needed design guidelines specific to mass timber buildings.

1.3. Current Work

This study aims at experimentally investigating the behaviour of 2D frames used in post-and-beam mass timber buildings under a middle column removal scenario, i.e. the middle column of an internal frame. Quasi-static tests were carried out on ¼-scale 2-bay frames assembled from three different types of commercially available beam-to-column connectors and a newly proposed novel connector. The different failure modes, the moment-rotation curves and the ability of each connector to develop first compressive arch action and second catenary action under large deformations are presented and discussed. The capacity of the frames to sustain the uniformly distributed pressure (UDP) transferred from the building is examined. This study represents an essential first step to fully understand the behaviour of mass timber buildings under a column removal scenario, and to comprehensively provide insights into (1) the type of connectors to be used in these buildings, (2) their associated collapse resistance mechanisms, and (3) how commercial connectors could potentially be improved. Additionally, while progressive collapse is essentially a dynamic event, static tests must be performed to eventually derive dynamic amplification factors commonly used in design [10], either by
comparing static and dynamic test results [30] or from the energy conservation principle [31]. The extent of the research due to the scale factor used are also discussed.

2. MATERIAL AND METHOD

2.1. Specimen Design and Timber Used

A six-storey tall, 5×5-bay on an 8,000 mm×6,000 mm grid, representative timber office prototype building, with 295 mm thick CLT slabs, was designed based on the Australian timber standard AS1720.1 [32] and industry feedback. The cross-sectional dimensions of the beams and columns were 600 mm×252 mm and 360 mm×360 mm, respectively. For the purpose of this research, both beams and columns were LVL structural products. The design dead load (self-weight of CLT panels), superimposed dead load and live load acting on the prototype building were 1.48 kPa, 1 kPa, and 3 kPa (representing offices for general use [33, 34]), respectively. The representative building is similar in essence to the 52 m tall timber building “25 King St” opened in 2018 in Brisbane, Australia.

Next, the ¼-scale, 2-bay substructure to be tested, was extracted from an internal frame of this representative prototype building (See Figure 1). The beams and columns consisted of Radiata pine (Pinus radiata) or Douglas fir (Pseudotsuga menziesii) LVL structural products manufactured by Carter Holt Harvey [35] with cross-sectional dimensions of 150 mm×63 mm and 90 mm×90 mm, respectively. Note that the values of the Modulus of Elasticity (MOE) and characteristic bending Modulus of Rupture (for a 95 mm deep LVL) provided by the manufacturer are 13,200 MPa and 50 MPa, respectively [35]. The actual MOE values of the LVL used in the experiments were measured in Section 2.4.1.

All LVL structural products were delivered, cut to required lengths and then stored inside the laboratory at ambient temperature and humidity. As the value of the moisture content of the timber as the time of testing (see Section 2.4.2) was measured, the environmental conditions inside the laboratory were not recorded. Since the thickness of the delivered LVL products were either 45 mm or 63 mm, two 45 mm thick LVL were glued together in the laboratory using polyurethane structural adhesive, under a pressure of 1 MPa, to manufacture the 90 mm thick columns.
2.2. Extend of Present Study and Size Effects

Due to available laboratory space and budget, testing of scaled down structures under column removal scenarios has been well-established and commonly practiced, see [36-41] for instance. In such experiments, the UDP applied to the scaled down floors (i.e. in kPa) is kept the same as the full-scale structure. With a scale factor of $\alpha$, the scaled down structure would therefore experience shear forces and bending moments $\alpha^2$ and $\alpha^3$ times lower, respectively, than the ones exhibited by the full-scale structure. As all dimensions are scaled down, the shear area and section modulus of a scaled down structural element are also $\alpha^2$ and $\alpha^3$ times lower, respectively, than the same of the full-scale element. The full-scale and scaled-down structures consequently experience the same stress values and distributions under the same UDP. Due to this stress matching and using the same material, the scaled down structure theoretically behaves as the full-scale one. Capacities of the full-scale system can also be theoretically calculated from the scale factor.

Notwithstanding, fibres, knots and other defects in timber structures cannot be scaled down, which may affect the experimental results. Furthermore, it is well known that a size effect exists in timber elements, with the strength of an element decreasing as its size increases [42-44]. Therefore, caution must be taken in testing scaled down timber structures. While our laboratory would have been able to accommodate full-scale post-and-beam specimens, not without difficulties, a scale factor of $\frac{1}{4}$ was considered in this study for the following reasons: (1) it allows consistency with 3D experimental tests, performed on $\frac{1}{4}$-scale 2×2-bay substructures [45] (total area of tested sub-structures and test rig of 6.5 m×4 m – an optimum size for our strong floor), in which the same scaled down frames were used, and (2) it enables a large variety of connections to be tested under a given budget, therefore obtaining a comprehensive picture of the behaviour of mass timber buildings under a column loss scenario.

With the above extend in mind and to correctly interpret the results in both this study and when testing 3D substructures, the $\frac{1}{4}$-scale and full-scale bending and shear responses of the three types of commercial beam-to-column connectors considered in this paper were experimentally investigated in
Focus of the experiments [46] was on connections, as under a column removal scenario most of the deformation occurs in the connections, they are key to ensure robust timber structures [17, 19, 28]. Taking into consideration the scale factor, results showed that the non-linear responses, stress distributions and failure modes of the ¼-scale and full-scale connectors were similar. Nevertheless, the shear and bending relative capacities of the ¼-scale connections tended to be typically about 20% higher, partially due to the different ductility of the aluminium material used in the ¼-scale and full-scale tests and also to the aforementioned nature of the timber material. Similar conclusions were made in Kasal et al. [47] who found that ¼-scaled fibre-reinforced and densified beam-to-column timber connections represented an “excellent indicator” to the properties of the full-scale connections. Keeping in mind that the system capacity may be overestimated, the work in [46, 47] provided evidences that tests performed on ¼-scale structural mass timber systems are valid and adequate to satisfactory reproduce the overall non-linear responses and gain an in-depth understanding of the full-scale structural system. The approach followed in this paper is consistent with the tests performed on scaled down concrete structures which are also sensitive to size effects [48-50]. Design guidelines [3, 4, 10] were mainly developed from such scaled down tests and the overall understanding of the structural behaviour they provided. In this paper, when applicable, a discussion is made on the differences to be expected between ¼-scale and full-scale test results from the work in [46].

2.3. Beam-to-Column Connectors

Three types of commercially available beam-to-column connectors, currently used in mass timber buildings, were investigated in this study. Due to the specificity of timber buildings in which the horizontal stability is usually ensured by shear walls and cross-bracing elements instead of frame actions, commercial beam-to-column connectors used in post-and-beam mass timber buildings are typically designed as shear connectors, not moment resisting ones. Despite been used in buildings, the three types of commercially available beam-to-column connectors being investigated were not especially designed for robustness. Additionally, a fourth connector is proposed and was designed as part of this study to resist the loss of a column through catenary action. Its performance was also examined.
As the design of connections is more comprehensive in the EN1995-1-1 [51], when compared to the AS1720.1 [32], all connectors were designed based on the former specification to sustain the factored design shear force of the representative building. The factored design shear force of the beam-to-column connectors under medium-term actions is 183 kN for the representative building.

2.3.1. T-section connector (T)

The first connector type is based on the Alumaxi connector manufactured by Rothoblaas [52]. It consists of a perforated T aluminium (AW6005-A) bracket with (i) one row of ten 16 mm dowels (s355) connecting the bracket web to the beam and (ii) two rows of six M16 bolts (class 8.8) going through the column and sandwich the column between the flanges of two brackets. In practice, a slot is cut in the middle of the beam to accommodate the bracket web and the beam is slid on top of the bracket into position.

For the ¼-scale tests, an exact scaled down version of the full-scale T aluminium bracket was manufactured with grade T6-6061 aluminium. 4 mm dowel (s355) and M4 bolts (class 8.8) were used. All connectors were manufactured from the same extruded aluminium block. Figure 2 shows a ¼-scale connector, with detailed dimensions, connected to the column.

2.3.2. Double beam connector (D)

The second connector type is composed of two LVL beams connected to two opposite faces of the column through one row of seven M16 bolts (class 8.8). This type of connection is used for instance in the BSkyB building in the UK. Two configurations were tested for this type of connector. Configuration 1 (DC1) did not provide continuity through the building and each beam spanned only one bay. The gap distance between two adjacent beams was set to 8 mm for the representative building, hence 2 mm for the scaled down specimens. Continuity was provided in Configuration 2 (DC2) with each beam spanning two bays. The beams were staggered so that at least one of the two beams was continuous over a column. The two configurations are shown in Figure 3 on the tested ¼-scale configuration.
For the ¼-scale tests, an exact scaled down version of this connector was tested with M4 bolts (class 8.8). Note that for this type of connection, the beams were a pair of 150 mm×31.5 mm LVL. Figure 4 shows a ¼-scale double beam connector (DC1) with detailed dimensions.

2.3.3. Megant type connector (M)

The third connector type is similar in principle to the Megant type connector manufactured by Knapp [53]. This connector is commonly used in Australia such as in the International House Sydney and 25 King St. The full-scale connector consists of two aluminium plates, two clamping jaws and two M20 threaded rods. To form a connector, the first and second plates are attached with eight 8 mm horizontal and twenty 8 mm inclined screws to the end of the beam and to the column, respectively. The two plates are then sandwiched together in-situ through top and bottom clamping jaws connected by two M20 threaded bars.

Note that 2 mm screws are not available in the market. Therefore, unlike the previous two connector types, an exact scaled down version of this connector could not be made. The tested version was therefore re-designed following the manufacturer design recommendations [53] using available screws to match the targeted design load. The connectors were then manufactured especially for this project by Knapp, consisting of two aluminium plates (AW6060) connected to the beam ends and columns with two 4 mm (CS4) horizontal and six 5 mm (CS5) inclined screws. A pair of clamping jaws were placed on the beam ends (at the top location) and columns (at the bottom location) with two 5 mm (CS5) horizontal screws and finally the two plates were sandwiched together with one M8 bolt (class 8.8) going through the two clamping jaws. Figure 5 shows the details of the ¼-scale redesigned Megant type connector, screwed to a beam end and a column side, separately.

2.3.4. Proposed double plate connector (DP)

A novel double plate connector (DP) (Figure 6) was designed to enable ductile failure to occur in the connector, rather than brittle failure in the timber. The connector was manufactured and tested using either steel (Grade 250) or aluminium (T6-6061) plates, as described hereafter. The connector consists of two pairs of 10 mm thick plates inserted into pre-cut slots in the beams (1st pair) and the columns
(2nd pair). All plates are assembled “off-site” to the timber elements through overdesigned 16 mm steel dowels (s355). While the beam plates are flush with the beam edges, the column plates overhang on both sides of the columns and each plate has four slotted holes to allow the beams to rotate up to 0.2 rad (Figure 7 (b)). “On-site”, the beams are slid into the overhang column plates and are connected to them by six M16 bolts (class 8.8). The proposed connector has the advantage of being cost effective in terms of manufacturing as it is only made of steel or aluminium plates, which can be easily waterjet cut. However, the work to install the dowels off-site will negatively impact on the overall cost. On-site connections are quick and effective.

The tested DP connector was an exact scaled down version of the full-scale one, and three different design alternatives were investigated: (1) DPC1 consisted of steel plates and had six steel spacers between the two beam plates to maintain appropriate spacing between them; (2) DPC2 consisted of steel plates and no spacer; and (3) DPC3 consisted of aluminium plates with spacers. The plates were cut out from the same steel or aluminium sheets.

2.4. Material Properties

2.4.1. Non-destructive testing of timber

Before manufacturing the tested frames, the dynamic MOE of each LVL element was measured using a non-destructive acoustic method [54]. As shown in Figure 8, the LVL elements were simply supported by two rubber strips and were impacted by a hammer. A microphone was used to record the free vibration of the LVL and the signal was analysed using the software Beam Identification by Non-destructive Grading (BING) [55]. Both the longitudinal (i.e. impacting the LVL in compression) and edge bending (i.e. impacting the LVL in bending with the LVL on its edge) dynamic MOE of the LVL beams were recorded, while only the longitudinal dynamic MOE of the LVL columns was measured.

2.4.2. Moisture Content of Timber

To determine the moisture content (MC) of the timber at the time of both non-destructive (dynamic MOE) and destructive (under column removal scenario) testing, samples were cut from selected
specimens immediately after each test and weighted. The MC of the timber was then determined by the oven-dry method in the Australian standard AS/NZS 1080.1 [56].

2.4.3. Tensile testing of bolts and dowels

Nine 4 mm dowels, six M4 bolts and five M8 bolts were randomly selected from the batches used in the experiments to measure the material properties of the dowels and bolts. The middle section of the 4 mm dowels and M4 bolts was machined to a nominal diameter of 3.5 mm and 3.1 mm, respectively, over a gauge length of 30 mm. Similarly, the middle section of the M8 bolts was machined to 6.8 mm in diameter over a gauge length of 50 mm. The samples were tested following the Australian standard AS4291.1 [57] in a 100 kN Instron universal testing machine. The strain rates were chosen to best match with those encountered by the material during the column removal scenario tests and are summarised in Table 1. The Young’s modulus of the samples was determined from a 25 mm gauge length extensometer. The ductility of the materials was measured using the percentage reduction of area (RA) of each coupon, calculated following the Australian standard AS4291.1 [57]. Measured dimensions of the samples were used to determine their Young’s modulus, yield stress and ultimate strength.

2.4.4. Tensile testing of steel and aluminium plates

Six coupons were cut from the T connectors, four coupons from the plates in the M connectors, and three coupons from the steel and three coupons from the aluminium plates used in the DP connector. The T connector and M connector coupons had a nominal width of 10 mm and a gauge length of 25 mm, while the DP connector coupons had a nominal width of 10 mm and a gauge length of 80 mm. To accurately measure the Young’s modulus of the material and avoid measuring out-of-straightness deformations, two coupon samples in each set were fitted with two 3 mm and 5 mm gauge length strain gauges for steel and aluminium, respectively, glued on each face of the coupons and at mid-length. The average strain deformation of these gauges was used in measuring the Young’s modulus from the stress-strain curves.
Similar procedures and measurements to the ones applied to the dowels and bolts were implemented
to the coupon tests following the Australian standard AS1391 [58]. Strain rates applied to the coupons
are given in Table 2.

2.5. Test Set-up

2.5.1. General

The frame test set-up is shown in Figure 9, consisting of a 2-bay frame with a removed middle column
to simulate the loss of an interior column. Horizontally, to simulate the restraints provided by the
adjacent bays of the building, each edge (side) column was connected to a rigid frame through two
pinned, 75 kN capacity load cells (LC$_1$ to LC$_4$) positioned above and below the beam centreline axis
and at a distance $d = 285$ mm (Figure 9), i.e. away from the studied connectors so as not to influence
their behaviour. This arrangement allowed the column to stay vertical and not displace horizontally,
therefore mimicking what would primarily occur in a complete building. The load cell lay-out is
shown in Figure 9. Therefore, the ability of the system to resist progressive collapse through catenary
action can be quantified through the axial force $F_{\text{axial}}$ developed in the beams as:

$$ F_{\text{axial}} = \frac{1}{2} \sum_{i=1}^{4} F_{LCi} $$(1)

where $F_{LCi}$ is the force measured by load cell number $i$. The moment $M_c$ resisted by each connector
is calculated with respect to the centreline of the edge column as:

$$ M_c = \frac{d}{2} \left[ (F_{LC1} + F_{LC3}) - (F_{LC2} + F_{LC4}) \right] $$

(2)

where all symbols are given in the paragraph above. In the vertical direction, a 250 kN capacity Moog
servo-controlled hydraulic jack was pin connected to the top of the removed column through a swivel,
and a quasi-static load was applied at a stroke rate of 10 mm/min. To prevent in-plane and out-of-
plane rotations of the middle column, a telescopic tube system [37] which could guide the removed
middle column vertically was used (Figure 9). It consisted of three different-sized steel tubes, in
which the smallest one was rigidly connected to the bottom of the removed middle column, and the
largest one was connected to the strong floor. Ultrahigh-molecular-weight polyethylene (UHMWPE) smooth plates were glued to the sliding contact surfaces and greased to minimise friction.

To prevent shear force to develop in the side columns between the points of action of $LC_2$ and $LC_4$, and the strong floor, the bottom end of each side column was connected to a roller support manufactured from a steel cylinder and roller bearings. In the preparation phase, the removed middle column alone was connected to the swivel and to the extended telescopic tube. The load in the hydraulic jack was then zeroed, and the whole specimen was assembled.

To check repeatability, tests on the T, DC1 and M connectors were performed twice and as the results between two tests were similar (see Section 3.2), no further repeated tests were required. Furthermore, as the two DC1 test results were similar, the continuous D connector (DC2) was only tested once. Similarly, (1) based on the repeatability of the tests performed on the above connectors and (2) as failure primarily occurred in the steel or aluminium for the DP connector, a material with a lower variability in its material properties than timber, each design modification of the DP connector was only tested once.

2.5.2. Measurements

Six linear variable displacement transducers ($DT_1$ to $DT_6$) were placed at various locations on the specimens to measure the vertical displacements of the middle column and of the two beams. The vertical displacement of the removed middle column was measured by two transducers ($DT_5$ and $DT_6$), symmetrically positioned on each side of the removed column (Figure 9), and the average of the two readings was used to measure the vertical displacement of the column. The other four displacement transducers were positioned along the beams as shown in Figure 9.

The rotation of the beams at the removed column location was measured by two linear variable rotation transducers ($RT_1$ and $RT_2$), with the location of the transducers given in Figure 9. The rotation of the beams relative to the middle column was calculated as the average of the two transducer readings. Additionally, two strain gauges (30-mm gauge length, $SG_1$ and $SG_2$) were glued on the top and bottom surfaces at the mid-span of one beam to record the strain profile in the beam.
In this paper, only selected measurement readings are reported and discussed.

3. TEST RESULTS

3.1. Material Testing Results and Moisture Content

The average density, average dynamic longitudinal and edge bending MOE of all beams used, and average dynamic longitudinal MOE of all columns used, are summarised in Table 3 with Coefficients of Variation (COV) on measurements given. The average moisture content of the timber at the time of the MOE measurements is also given in Table 4. The COV values on the MOE measurements are small (less than 4.5%) showing the homogeneity of the LVL products and as such the influence of the variation in timber material properties on the results are minimal. Table 4 also shows that the average MC increased by 0.6% in average within the seven months needed between the MOE measurements (acoustic tests) and testing of the frames.

The tensile test results for all bolts and dowels are summarised in Table 5 with COV on measurements. The yield stress of M4 bolts was higher when compared to 830 MPa of the design grade 8.8. Table 6 presents all steel and aluminium coupon tensile test results with COV on measurements.

3.2. Test Results

3.2.1. Applied load-displacement curves and failure modes

The applied load \( P \) to the frame versus the measured vertical displacement of the removed column is shown in Figure 10 for all tests performed.

For the T-section connector (T) (Figure 10 (a)), the load increased linearly, with an average stiffness of 0.068 kN/mm, until a middle column displacement of about 40 mm. The applied load then reached a plateau at a displacement of about 75-80 mm (marked as A in Figure 10 (a)). Failure fully developed at a middle column displacement of about 130-140 mm where the load dropped rapidly (marked as B in Figure 10 (a)). The average maximum applied load was 4.1 kN. Failure occurred upon yielding and fracture of the aluminium bracket at the web-flange intersection. Bending tests performed on full-scale connectors in [46] showed that aluminium fracture occurred at an early stage in the \( \frac{1}{4} \)-scale
connectors, suggesting that more ductility but same failure mode would have been expected if full-scale frames were tested. The typical observed damages for each key stage and the final failure modes are shown in Figure 11.

The double beam connectors showed a different response to those of the T-section connectors. For DC1, the load increased almost linearly, at a stiffness 4.3 times lower than that of the T-section connectors, up to a middle column displacement of about 170-180 mm, where tensile failure perpendicular to the grain developed in the column. This was characterised in Figure 10 (b) by a sudden drop of the applied load in the first test (marked as A in Figure 10 (b)) and a plateau in the second test. At this stage, the catenary action was fully developed (as explained in Section 3.2.2), and the load further increased to an average applied load of 5.3 kN (marked as B in Figure 10 (b)). Failure eventually occurred when tension perpendicular to the grain was fully developed in the columns, as shown in Figure 12. The full-scale bending tests of connections in [46] showed a relatively lower moment capacity when compared to the ¼-scale connectors due to the tension perpendicular to grain failure developing at an early stage. This suggests that a more brittle behaviour would be expected if full-scale frames were tested.

On the other hand, DC2 showed a different behaviour when compared to DC1. The load increased linearly, with a stiffness 4.9 times higher than that of DC1, until the middle column displacement reached 92 mm (marked as D in Figure 10). At this stage, the load suddenly dropped from 9.1 kN to 3.0 kN due to brittle bending failure occurring in the continuous beam at the middle column location (Figure 12). Catenary action developed after this stage, and the applied load further increased up to 8.7 kN (marked as E in Figure 10 (b)). The applied load then dropped again and remained constant at about 7.2 kN (mark as F in Figure 10 (b)) until the test was ended. The ultimate failure was due to a tensile failure perpendicular to the grain in the middle column (Figure 12).

For the Megant type connector (M), the initial stiffness was similar to that of the double beam connectors tested in DC1, and the load increased linearly until a removed column displacement of about 50 mm. The applied load then reached a first peak of 1.8 kN (marked as A in Figure 10 (c)), due to the bending of the aluminium plates. After a middle column displacement reached about 200
mm, the load increased again due to the development of the catenary action (marked as B in Figure 10 (c)) described in next sub-section. Finally, the applied load reached a second peak at an average of 2.4 kN (marked as C in Figure 10 (c)), which is 33% higher than the first peak. At this stage, both the catenary action and the moment resisted by the connector were exhausted. The ultimate failure (marked as D in Figure 10 (c)) was induced at a middle column displacement of about 300-330 mm by the bending of the aluminium plates screwed to the beam in one of the four connectors, eventually detaching from the two clamping jaws, as shown in Figure 13. Note that while the tests were performed until all connectors failed, the curves are only plotted herein until complete failure occurred first in one connector. Indeed, due to friction developing at that stage in the telescopic tube forced to keep the middle column vertical from unbalanced moments, the applied force increased unrealistically and is omitted. This phenomenon was only observed for Megant type and DPC3 connectors.

The proposed double plate connector was successful in enabling catenary action (see next sub-section) and providing higher capacities when compared to the other connection types investigated. For DPC1, the load increased almost linearly, at a stiffness of 0.062 kN/mm, until a vertical displacement of 83.7 mm (marked as A in Figure 10 (d)), at which the load reached a plateau of about 4.7 kN due to the bolts overcoming the friction forces and sliding into the slotted holes. From a displacement of 150 mm, the applied load increased again due to bolt bearing and the subsequent development of catenary action. Successive failure of the bolts in bearing then occurred up to a removed column displacement of 401.4 mm when the maximum load of 10.3 kN was reached (marked as B in Figure 10 (d)). Then the applied load dropped and finally, the load reached 8.3 kN (marked as C in Figure 10 (d)). DPC2 showed a similar behaviour to the first test, however the maximum load of 6.6 kN reached was 36% lower. For DPC3, while the initial stiffness was similar to the previous two tests, the lower Young’s modulus of the aluminium, when compared to steel, allowed the bolts not to fail successively as in the first two tests. Indeed, the bearing forces of the bolts were more uniformly distributed due to the deformation of the aluminium and more tension force was able to be mobilised in the catenary stage. The applied load reached the first peak of 14.4 kN which corresponding the removed column
displacement of 242 mm (marked as D in Figure 10 (d)). A maximum applied load of 15.7 kN was then reached (marked as E in Figure 10 (d)). In the final stage before the test was ended, complete failure occurred in one of the four connectors (marked F in Figure 10 (d)), completely disconnecting the beam from its column. The deformations of the specimens at the key stages in Figure 10 (d) are shown in Figure 14.

For ease of comparison, all applied load-removed column displacement curves are plotted together in Figure 10 (e). Table 7 provides the maximum load reached for each test.

3.2.2. Axial load-displacement curves

Figure 15 shows the measured axial force in the beams (Equation (1)) versus the measured vertical displacement of the removed column for all tests. For the T-section connector (T) shown in Figure 15 (a), the axial force was almost zero up until a middle column displacement of about 110-130 mm. At this stage, the axial force started to develop in the beam until premature failure occurred in the aluminium brackets (Figure 11 (c)), preventing the catenary action to fully develop. The average maximum recorded axial force was 3.2 kN.

The DC1 double beam connectors (Figure 15 (b)) presented a different behaviour to the T-section connectors. The catenary action was recorded to develop at a slightly higher middle column displacement of about 140 mm, and the average maximum recorded axial force was 11.2 kN. The double beam connectors tested in DC2 showed that the catenary action developed at a middle column displacement of about 130 mm, and the maximum axial force of 14.9 kN was 32% higher than the DC1 tests. The continuous beam through the column significantly influenced the overall capacity of the system to enable catenary action.

For the Megant type connector (M) shown in Figure 15 (c), up to a middle column displacement of 200 mm, a compressive axial force was recorded due to a rocking mechanism of the beams allowing a compressive arch action (CAA) to develop, a similar phenomenon encountered in concrete structures [16]. This phenomenon was mainly encountered for the Megant type connector. The
catenary action then developed, and the axial force in the beams increased up to an average recorded load of 7.2 kN at which point, the connectors reached their maximum capacity.

The axial force behaviour was similar between double steel plate DPC1 and DPC2 connectors. The catenary action started to develop around a vertical displacement at 150 mm, which indicated that the bolts have completely slid into the slots and to bear the metal plates. From then, the increasing of the axial force was not smooth due to the successive failure of the bolts, as mentioned in Section 3.2.1. The maximum axial loads reached for DPC1 and DPC2 were 27.2 kN and 15.7 kN, respectively, corresponding to a removed column displacement of 337 mm and 333 mm, respectively. DPC3 showed a slightly different behaviour when compared to the previous two tests. The catenary action started to develop earlier and at a removed column displacement of about 90 mm. The axial force increased linearly from a removed column displacement of 160 mm until a maximum axial load of 37.8 kN was reached, at a column displacement of 245 mm. The maximum axial force reached for DPC3 was 38% higher than that for DPC1, reconfirming the efficiency of the aluminium plates in mobilising more bolts to resist the axial force than the steel plates. After a removed column displacement of 300 mm, the axial force dropped quickly, at about 14 kN.

Again for the purpose of comparison, all axial force-displacement curves are plotted together in Figure 15(e).

3.2.3. Moment-rotation curves

Figure 16 plots the moment (Equation (2)) versus beam rotation for all tests. The marked letters in Figure 10 are reproduced in Figure 16 to highlight the key stages mentioned in Section 3.2.1 on the figure.

The maximum average moment resisted by the T-section connector is 1.95 kNm (marked as B in Figure 16 (a)), corresponding to a beam rotation at about 4°. Although the T-section connector reached a moment higher than the DC1- and M-connectors, the early fracture of the aluminium did not allow the overall structural system to resist a higher applied load than the DC1 connector, principally due to different development patterns of the catenary action.
Two peaks were observed in the moment-rotation curves of the DC1 connectors, as shown in Figure 16 (b). The first peak was encountered at a beam rotation of $4^\circ$ and averaged at 0.73 kNm, which is 62% lower than the maximum moment resisted by T-section connectors. The second peak averaged at 0.8 kNm and a beam rotation of $7.7^\circ$. When the tests were ended at a removed column displacement of about one-fifth of the span, the connectors were still able to sustain a moment of the same order of magnitude as the second peak moment. Moreover, due to the connections being different between the side columns (no continuous beams) and the removed column (one continuous beam), the moment-rotation curve of DC2, calculated from Equation (2), is not plotted in Figure 16 (b) as it does not reflect the actual moment experienced by the connector at the removed column.

The moment-rotation curve of the Megant type connector (M) increased almost linearly until an average moment peak of 1.1 kNm (marked as A in Figure 16 (c)), corresponding to a beam rotation of about $3^\circ$. Then the moment resistance decreased to almost 0 kNm at a beam rotation of $8^\circ$ (marked as C in Figure 16 (c)), indicating that the applied force was mainly resisted by catenary action from this stage, up until ultimate failure at a beam rotation of about $11^\circ$ (marked as D in Figure 16 (c)).

The moment-rotation curves of the proposed double plate connectors are plotted in Figure 16 (d). DCP1 and DCP2, manufactured from steel plates, reached a maximum moment of 2.2 kNm (marked as A in Figure 16 (d)) at a beam rotation of $2.4^\circ$, and of 2.1 kNm at a beam rotation of $4^\circ$, respectively. After reaching the maximum moment, the moment progressively dropped to almost zero at a beam rotation of $8^\circ$ for the two connectors. DPC3, manufactured from aluminium plates, exhibited a different behaviour compared to the other two configurations. The moment resistance increased smoothly to a first peak of 3.2 kNm, corresponding to a beam rotating of about $4^\circ$. After a plateau stage, the moment resistance increased to 3.6 kNm at a beam rotated of $9.1^\circ$ at which the maximum applied load (marked as E in Figure 16 (d)) has already been reached.

The moment-rotation curves obtained from all the tests are plotted together in Figure 16 (e) to facilitate comparisons between different connectors.
The Department of Defence (DoD) UFC 4-023-03 [10] required rotation of 11.4° (0.2 rad) to be achieved by the connections for the beams “to provide some or all of the required tie forces”. Such a requirement was only met by both the double beam (D) and the proposed double plate (DP) connectors.

4. DISCUSSION ON SYSTEM CAPACITY

This section estimates the UDP applied to the floors that the post-and-beam system alone would be able to resist in case of a column removal scenario. As the experimental results presented in this paper were obtained from a concentrated point load applied at the removed column, the UDP that would generate the same moment at the connections, under the same removed column displacement, is calculated herein based on the complete derivation given in Appendix A. The maximum UDP ($UDP_{max}$) (in kPa) that the system would withstand is then given as:

$$UDP_{max} = \frac{F_{max}}{L_1L_2}$$  \hspace{1cm} (3)

where $F_{max}$ (in kN) is the maximum applied load (given in Section 3.2.1), $L_1 = 2$ m is the beam span and $L_2 = 1.5$ m is the floor span, based on the 8 m×6 m grid of the representative building detailed in Section 2.1. $UDP_{max}$ for all studied specimens are summarised in Table 7. The maximum UDP of all tested connectors was for the frame assembled by double plate connector (DPC3) and equal to 5.24 kPa and the maximum UDP of frames assembled with the commercially available connectors ranged from 0.76 kPa (Megant type) to 1.79 kPa (Double beam tested in Configuration 1), which is lower than the design dead load alone of 2.48 kPa.

$UDP_{max}$ is compared herein to the design pressures for office buildings $UDP_{design\_DoD}$, $UDP_{design\_IStructE}$ and $UDP_{design\_AS/NZS}$ in the DoD [10], IStructE [12] and Australian/New-Zealand [33] recommendations, respectively, given as:

$$UDP_{design\_DoD} = \Omega (1.2\times \text{Dead Load} + 0.5\times \text{Live Load})$$  \hspace{1cm} (4)

$$UDP_{design\_IStructE} = 1.0 \times \text{Dead Load} + 0.5 \times \text{Live Load}$$  \hspace{1cm} (5)
where $\Omega$ is the dynamic increase factor, equal to 2.0 for timber structures under nonlinear static analyses. Note that (1) out of the three recommendations, the DoD is the only one considering a dynamic increase factor and (2) $UDP_{\text{design, DoD}}$ is only applied to the bays above the removed column, the design pressure with no dynamic increase factor is applied to the remaining bays. In reference to the values in Section 2.1, $UDP_{\text{design, DoD}} = 8.95$ kPa, $UDP_{\text{design, IStructE}} = 3.98$ kPa and $UDP_{\text{design, AS/NZS}} = 3.68$ kPa for the representative building. None of the investigated connectors was able to sustain the design pressure of 8.95 kPa in the DoD. Only the double plate connector (DPC3) could achieve the design pressure in the IStructE and AS/NZS recommendations, with a $UDP_{\text{max}}$ 1.32 and 1.42 times higher than $UDP_{\text{design, IStructE}}$ and $UDP_{\text{design, AS/NZS}}$, respectively. The ratios between the maximum recorded UDP and the design ones are given in Table 7.

In view of the above, progressive collapse of post-and-beam mass timber buildings cannot be resisted by the frame alone using connectors currently used. Alternate load paths must be found, such as through the CLT floors to transfer the loads to adjacent frames and/or connectors especially designed to resist the large deformations associated with the loss of a column must be used [23].

5. CONCLUSION

The behaviour of mass timber frames, extracted from a post-and-beam building, under a middle column removal scenario was investigated. Large deformation tests were performed on $\frac{1}{4}$-scale, 2-bay substructures assembled from three types of commercial beam-to-column connectors and a proposed connector. The behaviour of the frames and the ability of the beam-to-column connectors to develop catenary action under large deformations were reported. The main findings are summarised below:

1. All commercial connectors provided enough rotation for the catenary action to either develop or start developing under large deformations. Megant type connector developed compressive arch action at the initial stage of loading. However, the amplitude of these mechanisms was not enough to be taken advantage of for progressive collapse design.
2. Overall when compared to currently used connectors (T-section connector, double beam connector, Megant type connector), the proposed novel double plate connector provided higher capacities and allowed catenary action to be taken advantage of. It therefore represents a potential solution to improve the robustness of post-and-beam mass timber buildings and shows that structural robustness can be achieved with correct design approaches.

3. Results showed that frames manufactured from the commercially used connectors investigated as part of this study and from the proposed double plate connector cannot resist by themselves the design UDP applied to the floor under a column removal scenario set in the DoD guideline. However, the proposed double aluminium plate connector (DPC3) allowed the frame to resist the design UDP set in the IStructE and Australian/New-Zealand recommendations, which are contrary to the DoD in that a dynamic increase factor is not considered.

4. In post-and-beam mass timber buildings, the loss of a column must be resisted either by ductile connectors specially designed for the large deformations associated with the loss of a column, or through alternate load paths (such as CLT floors), or a combination of both.

While this study investigated various beam-to-column connectors to provide a comprehensive picture of the behaviour of post-and-beam mass timber frames, conclusions from this study are only valid for the investigated connectors. More investigations would be needed to cover the full range of beam-to-column connectors currently used in post-and-beam mass timber buildings. Additionally, as progressive collapse is essentially a dynamic event, the study provides the test data to eventually derive dynamic amplification factors, either by comparing static and future dynamic test results or from the energy conservation principle.
ACKNOWLEDGEMENTS

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REFERENCES


FIGURES

Figure 1 Plan view of prototype building and the extracted substructure (unit: mm)

(a) Dimensions of T (unit: mm)  (b) Connector bolted to column

Figure 2. Details of Aluminium alloy T-section connector (T) used in experiments
Figure 3. Tested configurations (plan views) for double beam connectors (unit: mm)

(a) Configuration 1 (DC1)  (b) Configuration 2 (DC2)

Figure 4. Details of double beam connector (D) used in experiments

(a) Dimensions of DC1 (unit: mm)  (b) Connector bolted to column (DC1)

Figure 5. Megant type connector (M) used in experiments

(a) Dimensions of M (unit: mm)  (b) Construction of connector
(a) Breakdown components

(b) Beam assembly off-site    (c) Column assembly off-site    (d) Beam & column assembly on-site

Figure 6. Proposed double plate connector assembly instruction (DP)
(*: only for DPC1 and DPC3)

(a) Beam plates    (b) Column plates

Figure 7. Dimensions of DP used in experiments (unit: mm)
Figure 8. Test set-up for dynamic MOE determination

(a) Overall view
P: Applied load/Reaction force

DT: Vertical LVDT

LT: Horizontal load cell

RT: Rotational transducer

SG: Strain gauge

(b) Dimension and measurement details (side view, unit: mm)

Figure 9. 2D frame test set-up

(a) T-section connector

- T-Test 1
- T-Test 2
(b) Double beam connector

(c) Megant type connector
Figure 10. Applied load-displacement curves
Figure 11. Damage patterns at different stages and typical failure mode for T-section connector (A~C in Figure 10)
Figure 12. Damage patterns at different stages and typical failure modes for double beam connector (A~C: DC1; D~F: DC2 in Figure 10)
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Figure 14. Damage patterns at different stages and typical failure modes for double plate connector (A–C: DPC1; D–F: DPC3 in Figure 10)
(b) Double beam connector

(c) Megant type connector
Figure 15. Beam axial force-displacement curves

(d) Double plate connector

(e) All connectors
(a) T-section connector

(b) Double beam connector
(c) Megant type connector

(d) Double plate connector
Figure 16. Moment-rotation curves

(e) All connectors
### TABLES

#### Table 1 Summary of tensile tests of bolts and dowels

<table>
<thead>
<tr>
<th>Sample</th>
<th>Number of tests</th>
<th>Strain rate (mm/mm/min)</th>
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<tr>
<td>M4 dowel</td>
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<td>M4 bolt</td>
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<tr>
<td>M8 bolt</td>
<td>5</td>
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#### Table 2 Summary of tensile tests of steel or aluminium connector

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<td>T-section connector</td>
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<td>9.4×10⁻⁴</td>
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<td>3.4×10⁻³</td>
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#### Table 3 Non-destructive testing results

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<th>Element</th>
<th>Density (kg/m³)</th>
<th>Dynamic MOE (MPa)</th>
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<td></td>
<td></td>
<td>Longitudinal</td>
<td>Edge bending</td>
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<tr>
<td>Beam</td>
<td>583.4 (1) (2.2)</td>
<td>15,428 (1) (4.5)</td>
<td>13,681 (1) (3.7)</td>
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<tr>
<td>Column</td>
<td>598.3 (2) (7.2)</td>
<td>15,552 (2) (3.4)</td>
<td>-</td>
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Note: COV measurements given in ( ) in %

(1) Average on 24 tests
(2) Average on 30 tests

#### Table 4 MC test results

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<tr>
<td>BING Test</td>
<td>8.5% (1) (5.8)</td>
<td>9.5% (2) (13.7)</td>
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<tr>
<td>Experimental Test</td>
<td>9.1% (1) (5.9)</td>
<td>10.1% (2) (7.2)</td>
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</table>

Note: COV measurements given in ( ) in %

(1) Average on 24 tests
(2) Average on 30 tests

#### Table 5 Tensile test results of bolts and dowels

<table>
<thead>
<tr>
<th>Sample</th>
<th>Elastic modulus (MPa)</th>
<th>Yield stress (MPa)</th>
<th>Ultimate strength (MPa)</th>
<th>Reduction of area (%)</th>
</tr>
</thead>
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<tr>
<td>M4 dowel</td>
<td>204,084 (14.6)</td>
<td>538 (3.5)</td>
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<td>64 (3.5)</td>
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<td>M4 bolt</td>
<td>204,644 (6.7)</td>
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<td>67 (2.2)</td>
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<td>M8 bolt</td>
<td>214,865 (7.5)</td>
<td>838 (1.2)</td>
<td>892 (1.5)</td>
<td>71 (1.2)</td>
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</table>

Note: COV measurements given in ( ) in %

#### Table 6 Tensile test results of steel or aluminium plates

<table>
<thead>
<tr>
<th>Sample</th>
<th>Material</th>
<th>Elastic modulus (MPa)</th>
<th>Yield stress (MPa)</th>
<th>Ultimate strength (MPa)</th>
<th>Reduction of area (%)</th>
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<td>194,918 (1.9)</td>
<td>264 (1.2)</td>
<td>383 (0.6)</td>
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<td>T-section connector</td>
<td>Aluminium</td>
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<td>314 (0.6)</td>
<td>345 (1.1)</td>
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<td>Megant type connector</td>
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<td>64,460 (2.2)</td>
<td>214 (5.1)</td>
<td>237 (4.4)</td>
<td>53 (4.7)</td>
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<td>63,662 (3.4)</td>
<td>240 (9.6)</td>
<td>270 (2.0)</td>
<td>24 (2.9)</td>
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Note: COV measurements given in ( ) in %
Table 7 Summary of loading methods

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<th>Megant type connector</th>
<th>Double plate connector</th>
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<td></td>
<td>P&lt;sub&gt;max&lt;/sub&gt; (kN)</td>
<td>UDP&lt;sub&gt;max&lt;/sub&gt; (kPa)</td>
<td>UDP&lt;sub&gt;max&lt;/sub&gt;/UDP&lt;sub&gt;design DoD&lt;/sub&gt;</td>
<td>UDP&lt;sub&gt;max&lt;/sub&gt;/UDP&lt;sub&gt;design IStructE&lt;/sub&gt;</td>
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<tr>
<td>T-Test 1</td>
<td>4.14</td>
<td>1.38</td>
<td>0.15</td>
<td>0.35</td>
</tr>
<tr>
<td>T-Test 2</td>
<td>4.26</td>
<td>1.42</td>
<td>0.16</td>
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<tr>
<td>DC1-Test 1</td>
<td>5.36</td>
<td>1.79</td>
<td>0.20</td>
<td>0.45</td>
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<td>DC1-Test 2</td>
<td>5.26</td>
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<td>0.21</td>
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<tr>
<td>M-Test 2</td>
<td>2.27</td>
<td>0.76</td>
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<td>DPC 1</td>
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<td>DPC2</td>
<td>6.62</td>
<td>2.21</td>
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<td>15.72</td>
<td>5.24</td>
<td>0.59</td>
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Note: Calculations given in Appendix A are not applicable to DC2 which is a continuous beam
APPENDIX A MOMENT EQUATIONS

A.1. Point Load

Figure A.1 shows the free body diagram of half the test specimen, in which each connector is replaced by a spring and is assumed to respond the same under positive and negative moment. As the deformation principally occurred in the connectors, the bending stiffness of the beam is ignored herein. The free body diagram represents the frame under large deformations in which the applied force is resisted either by catenary action, bending of the connector or a combination of both. Based on statics principles, the moment $M$ at the connectors can be calculated by Equation (A.1) as:

\[
M = \frac{PL}{4} - \frac{F_1 \delta}{2}
\]  

where $P$ is the applied force, $F_1$ represents the horizontal reaction force originating from the axial force in the beams, $\delta$ is the displacement of the removed column and $L_1$ is the span of the beam.

A.2. Uniformly Distributed Load (UDL)

Figure A.2 shows the floor layout of the building. Considering the tributary area of the frames with the removed column, the $UDL \ \omega$ (in $kN/m$) applied to the frame is:

\[
UDL \ \omega \ (in \ kN/m) = Design \ UDP \ (in \ kPa) \times L_2
\]  

(A.2)
where \( L_2 \) is given in Figure A.2.

Similar to Figure A.1, the free body diagram of the half system when a UDL is applied to the beams is given in Figure A.3. Equilibrium gives

\[
M = \frac{\omega L_1^2}{4} - \frac{F_1 \delta}{2}
\]  

(A.3)

By combining Equation (A.1) to Equation (A.3), the design UDP (in kPa) which generates the same deformation of the system as a concentrated load \( P \) is:

\[
Design \ UDP \ (in \ kPa) = \frac{P}{L_1 L_2}
\]  

(A.4)