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EXPERIMENTAL STUDY ON THE DYNAMIC PROGRESSIVE COLLAPSE RESPONSE OF POST-AND-BEAM MASS TIMBER BUILDINGS

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ABSTRACT: Mid-rise to tall mass timber buildings are gaining growing attention due to their low carbon emissions and ease of construction. However, the risk associated with a progressive collapse event, leading to significant social and economic repercussions, becomes even more heightened with the increasing height of these buildings. Progressive collapses are dynamic and nonlinear events. Therefore, it is crucial to consider these dynamic effects in the robustness assessment of such buildings. Moreover, current progressive collapse design guidelines are principally based on studies performed on steel and concrete structures, with research on understanding the robustness of mass timber buildings remaining limited. This paper presents an experimental study on a scaled, 2×2-bay post-and-beam mass timber substructure subjected to a sudden interior column removal scenario. This paper introduces the experimental setup and discusses the preliminary findings, focusing on understanding the overall dynamic behaviour, failure modes, load redistribution mechanisms under large deformations and the contribution of different structural elements in resisting progressive collapse.

KEYWORDS: Mass timber buildings, Progressive collapse, Dynamic effects, Post-and-beam, Sudden interior column removal

1 – INTRODUCTION

Mass timber buildings refer to structures constructed using Engineered Wood Products (EWPs) such as Laminated Veneer Lumber (LVL), Glued Laminated Timber (Glulam) and Cross Laminated Timber (CLT). They are gaining popularity globally due to their lower carbon emissions, faster erection times, and being more aesthetically pleasing when compared to buildings constructed traditional materials, like concrete and steel. According to CTBUH [1], a total of 66 timber buildings, with eight stories or higher, were completed worldwide as of February 2022, such as the 10-storey “25 King St” in Australia (2018). One of the emblematic mass timber buildings currently under development is the 100-meter tall “Rocket & Tigerli” located in Switzerland and set to be completed by 2028.

Progressive collapse is defined as a disproportionate failure of a structure relative to an initial local failure of a critical structural element caused by an abnormal load (such as gas explosion, blast, fire, design error or vehicular collision). This local failure may propagate through the building and ultimately cause its partial or total collapse [2]. Such dynamic and nonlinear events can lead to significant social and economic implications. The increasing height of mass timber buildings intensifies the consequences of a progressive collapse event, which must be considered in the design of mid-rise to tall buildings. Tragic incidents, like the Ronan Point Apartment Tower collapse [3], underscore the importance of incorporating a robustness assessment in structural designs. To achieve this, one of the most commonly adopted procedures is the Alternative Load Path (ALP) method based on the notional member (e.g., column) removal concept [4]. The ALP method is both deterministic and independent of specific threats [5].

This study aims at investigating the dynamic progressive collapse behaviour of post-and-beam mass timber buildings. This paper first introduces the overall experimental setup of a 3D scaled substructure tested

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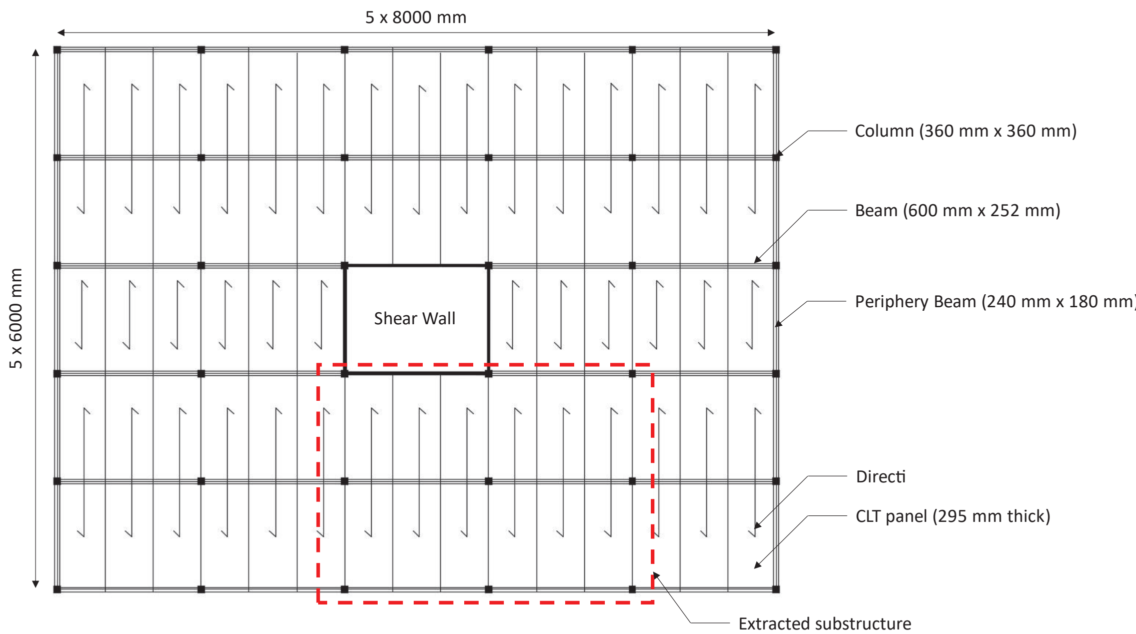

Figure 1. Plan view of prototype building and the extracted substructure (unit: mm)

under a sudden interior column removal scenario and then presents the preliminary results along with a discussion.

2 – RELEVANT EXISTING WORK

General guidelines to resist progressive collapse can be found in the Department of Defence (DoD) [4], Australian National Construction Code (NCC) [6] and the Institution of Structural Engineers (IStructE) [7]. These guidelines are principally based on studies performed on steel and concrete buildings. Very limited published articles have focused on the resistance of mass timber buildings to progressive collapse. In these limited studies, theoretical [8, 9], numerical [10, 11] and experimental [12-15] approaches have been employed.

In terms of experimental studies, Lyu et al. [13] performed 2D tests to investigate the behaviour of post-and-beam mass timber frames under a quasi-static column removal scenario. The tests were performed on a ¼ scaled structure and four different types of beam-to-column connectors were investigated. Furthermore, Lyu et al. expanded the research by quasi-statically testing 2×2-bay mass timber scaled down substructures, either under an edge [14] or corner [15] column removal scenario. Results showed that continuous CLT panels play a significant role in redistributing the load and that simplified design models adopted by industry to assess the robustness of post-and-beam mass timber buildings are conservative, as the full load redistribution mechanisms are ignored. However, these tests did not consider the dynamic effect associated with a progressive collapse event. Experimentally, only Cheng et. al [16] performed dynamic tests on mass timber substructures and presented a preliminary understanding of the dynamic behaviour of scaled-down, 2-bay post-and-beam mass timber frames under a sudden column removal scenario. Results showed that for some types of beam-to-column connectors, the Dynamic Amplification Factor (DIF) can be higher than the theoretical maximum value of 2.0 and that failure can develop prematurely dynamically at the connectors when compared to the static tests.

3 – EXPERIMENTAL SETUP

3.1 – OVERVIEW

A six-storey prototype post-and-beam mass timber office building, with an 8,000mm × 6,000mm grid, has been designed according to the Australian [17] and European [18] timber specifications. A plan view of the prototype building is shown in Figure 1. A ¼-scaled substructure (2×2-bay) was extracted from this prototype building (see Figure 1) and tested under a sudden interior column removal scenario.

LVL beams and columns with Megant-type beam-to-column connectors were used to manufacture the substructure as for the static tests performed in [14, 15]. The Megant type connector comprised of two aluminium brackets, and two clamping jaws which were CNC machined from AW 6060 aluminium alloy. One middle M8 socket headed cap screw (Grade 8.8) held the assembly together. This connector type is commonly used in Australia, as seen in mass timber buildings like 25 King Street (2018). The cross-sectional dimensions of the LVL beams and columns were 150 mm × 63 mm and 90 mm × 90 mm, respectively. The CLT panels, spanning

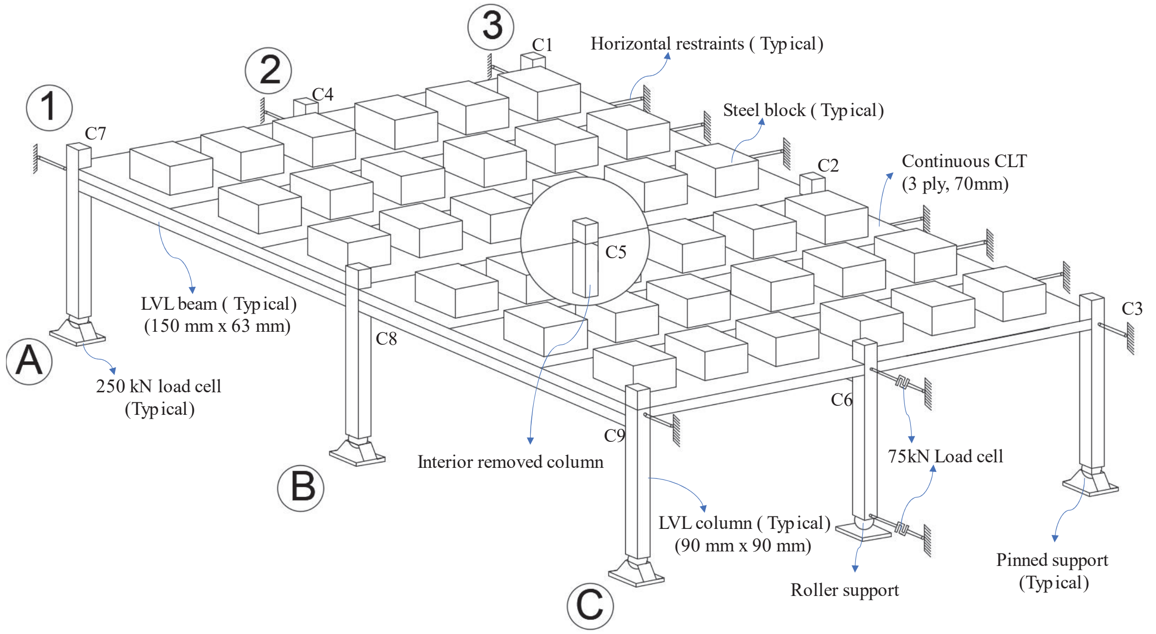

Figure 2. 3D perspective view of tested scaled-down substructure

2-bays, were 70 mm thick (3 layers) and manufactured from MGP10 [17]. HBS460 (4 mm × 60 mm), spaced every 136 mm, and HBS5120 (5 mm × 120 mm), spaced every 222 mm, self-tapping screws from Rothoblaas [19] were used to connect CLT-to-CLT and CLT-to-beam, respectively. A perspective view of the tested substructure is shown in Figure 2.

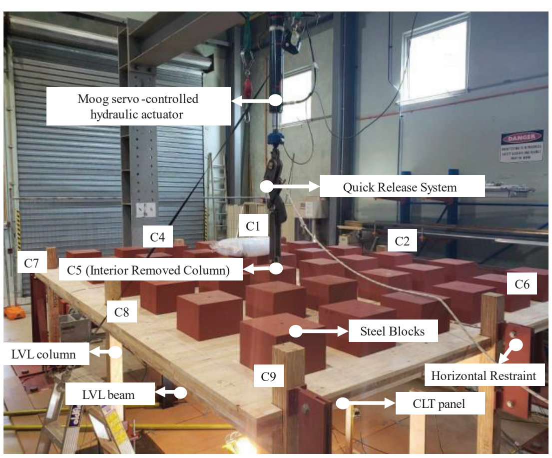

Figure 3. Photo of test setup shown with steel blocks only

Horizontal restraints (Figure 2) were applied around the perimeter of the substructure to simulate the restraints provided by the surrounding structural elements of the prototype building (Figure 1). Specifically, six columns (C1, C3, C4, C6, C7 and C9 in Figure 2) were horizontally restrained by pinned connections to rigid frames at the beam elevation, and the CLT panels were also horizontally restrained through pinned connections along their interior edge (axis 3-3 in Figure 2). All columns were pinned on the laboratory strong floor, except for (1) the removed interior Stub column C5 which was connected to a Moog servo-controlled hydraulic actuator through a quick-release system, as shown in Figure 3 and (2) Column C6 which was on a roller support but prevented from horizontal displacement at its base, effectively creating a pinned connection. The connection at Column C6 was designed to record the axial load and moment at the beam-to-column connection (see Section 3.2). A rope was attached to the quick-release system and yanked to trigger a sudden release. Note that the quick-release system plays an important role to trigger free fall of the structure. According to Biggs [20], the release time (tr) should be less than 1/10 of the natural period (T) of the structure to correctly induce and obtain the dynamic responses of the structure.

A uniformly distributed load (UDL) was incrementally applied to the CLT floor using steel blocks (each nominally weighing 125 kg) and sandbags. The loading sequence is outlined in Table 1. Five static and five dynamic experiments were conducted across 10 different loading sequences, ranging from the structure behaving elastically for Loading sequences 1 to 8 to the large deformation stage, with the structure behaving inelastically, for the last two loading sequences. The test IDs in Table 1 correspond to (1) the letter "S" for a static test, or "D" for a dynamic test, (2) the following number representing the equivalent number of steel blocks per bay, and (3) the word "Test X" indicating the specific test number when the tests were repeated. "D04-Test1" for example refers to the first dynamic test with four steel blocks per bay. Note that (1) tests were only repeated

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when the structure behaved elastically, (2) the design UDL (dead + live loads) of the structure corresponds to 9 steel blocks per bay, i.e., 4.0 kN/m^2, and (3) a UDL of 12.4 kPa was used to induce failure in the system.

A static test involved statically removing the interior column with the hydraulic actuator until all the load initially carried by the actuator (i.e., corresponding to the reaction of the interior column for an undamaged structure) was redistributed to the other columns. A dynamic test was conducted by suddenly removing the interior column using the quick-release system. After each static and dynamic test, the column was repositioned to its original position with the hydraulic actuator. When increasing the UDL, more steel blocks and sandbags were added to the substructure with the interior column attached to the hydraulic actuator in an undamaged scenario.

Table 1: Loading sequence

Loading sequence	Test ID	Load (kPa)	Behaviour
1	S04-TestX	2.0	Elastic
2	D04-TestX	2.0	
3	S06-TestX	2.8	
4	D06-TestX	2.8	
5[(1)]	S09-TestX	4.0	
6[(1)]	D09-TestX	4.0	
7	S16-TestX	6.8	
8	S20-TestX	8.4	
9	D16	6.8	Inelastic
10	D30	12.4	

[(1)] Corresponds to design UDL (dead + live load)

3.2 – INSTRUMENTATION

To quantify the static and dynamic behaviour, the load redistribution mechanisms and the contribution of different structural elements in resisting progressive collapse, the following data were recorded throughout the course of each experiment:

- Vertical displacement of the removed interior column stub using a laser displacement transducer.
- Bending moment and axial force at Column C6 beam-to-column connection using two horizontal 75 kN load cells connected to Column C6 and the rigid frame. The moment and axial force were calculated using the methodology outlined in [13, 16].
- Reaction forces at all columns using 250 kN load cells placed underneath each column except for the removed Column C5. For this column, the load cell of the hydraulic actuator was used to measure its reaction force.
- Strain development in beams and CLT at key locations using strain gauges.
- Overall vertical displacement profile of the substructure using laser displacement transducers.

During the dynamic tests, all sensors were recorded at a sampling rate of 1,400 Hz. A lowpass filter with a cut off frequency of 20Hz was applied to the signal to reduce noise [21].

4 – PRELIMINARY RESULTS

In this paper, the failure modes for the last two dynamic tests (D16 and D30), the vertical displacement at the removed column and the load redistribution to all columns after removing the interior column are presented.

During the tests, no noticeable failure was visually observed in the timber elements, with damage mainly

Figure 4. Deformation of the substructure after Test D30

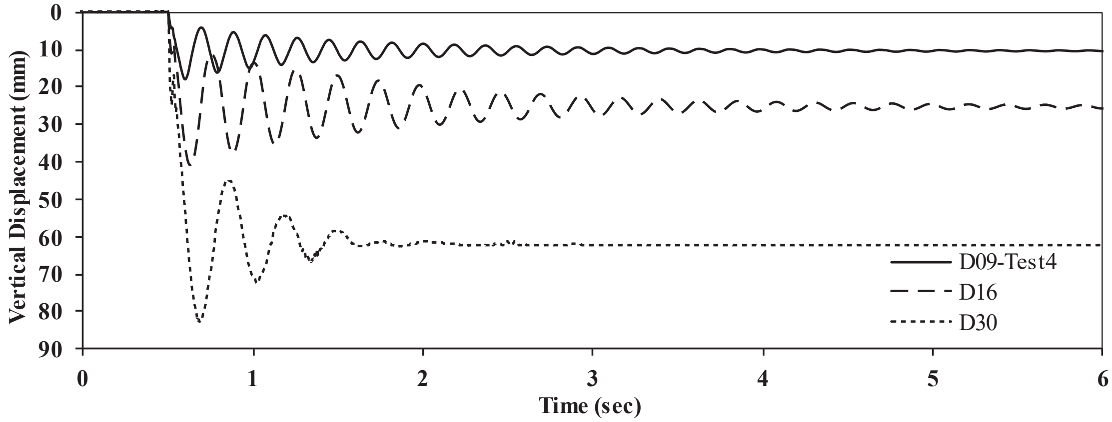

Figure 5. Vertical displacement-time history recorded at the removed interior Column C5 for representative dynamic tests

Table 2: Dynamic properties measured from representative dynamic tests

Test	Release time, t_r (ms)	Natural period, T (ms)	Maximum dynamic displacement (mm)	Displacement at steady state (mm)
D09-Test4	12	186	17.9	10.3
D16	11	256	40.8	25.4
D30	8	338	82.7	62.2

occurring at the beam-to-column connectors and the CLT-to-beam screwed connections. Permanent deformations were observed from Loading sequence 9 in Table 1 (D16). Figure 4 shows the pull-out of screws connecting the CLT to the beams near the removed Column C5 and the deformation in the beam-to-column connector after Loading sequence 10 (D30).

The vertical displacement-time histories for the removed interior Column C5 are shown in Figure 5 for representative dynamic tests. The maximum vertical displacement was found to be 17.9 mm, 40.8 mm and 82.7 mm for D09-Test4, D16 and D30, respectively, corresponding to 1/112, 1/49 and 1/24 of the beam span, respectively. The damage associated with the permanent deformation of the structure and the associated energy absorption for Loading sequence 10 can be observed in Figure 5 with a larger damping for D30 than the other two tests.

The measured release time (t_r), the natural period (T), the maximum dynamic displacement and the displacement at the steady state are summarised in Table 2 for the representative tests presented in Figure 5. The release times were found to be less than 1/10 of the corresponding natural periods of the tested substructures. Therefore, the dynamic response of the substructure, representing a free fall, was successfully triggered during the tests. The inelastic deformation of the substructure during Loading sequence 10 is further illustrated in the

Table 3: Redistribution of removed column reaction in undamaged columns

Test		Corner columns	Edge columns parallel to CLT	Edge columns parallel to beams	Total
D09-Test4	Reaction (kN)[(1)]	1.6	11.1	-0.6	12.7
	Percent (%)	13.0	91.6	-4.6	100
D16	Reaction (kN)[(1)]	3.2	22.3	-1.4	24.1
	Percent (%)	13.3	92.5	-5.8	100
D30	Reaction (kN)[(1)]	6.7	36.7	-5.0	38.4
	Percent (%)	17.4	95.6	-13.0	100

[(1)]A negative reaction value indicates a tensile reaction force

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table with the ratio of the maximum to steady state displacements decreasing from 1.7 for D09-Test4 to 1.3 for D30.

Table 3 presents the redistribution of the reaction force of the removed interior Column C5 to the remaining columns at the steady state, presented both in actual values and associated percentages. In the table, the columns were grouped into three groups: (1) Columns C1, C3, C7 and C9 representing the corner columns, (2) Columns C2 and C8 representing the edge columns along the direction parallel to the CLT panels (i.e., perpendicular to the beam direction), and (3) Columns C4 and C6 representing the edge columns along the direction parallel to the beams, (i.e., perpendicular to the CLT panel direction). For all loading sequences, the reaction force at the removed column is principally transferred, at more than 90%, to the edge columns in the direction of the CLT, indicating that the middle CLT panels spanning two bays are the main contributor in redistributing the load throughout the structure. While the load redistribution pattern is similar for tests D09-Test4 and D16, at the large deformation stage for D30, the effect of the CLT panels detaching from the beams (Figure 4) can be seen in Table 3 with less load being redistributed to the edge columns in the direction parallel to the beams and more load resisted by the CLT panels.

This result suggests that if a system is designed to resist the loss of an interior column through catenary action developing in the beams, such an alternative load path mechanism would be inefficient and would only potentially develop after the failure of the middle CLT panels spanning two bays, which initially provide the primary alternate load path.

5– CONCLUSION

This paper presented the preliminary results of an experimental investigation aiming at capturing the dynamic progressive collapse mechanisms of post-and-beam mass timber buildings under a sudden interior column removal scenario. Tests were performed in both the elastic and inelastic stages. Results showed that the CLT panels spanning two bays provided the primary alternative load path, redistributing more than 90% of the reaction of the removed column. However, this result suggests that if a system is designed to resist the loss of an interior column through catenary action developing in the beams, such an alternative load path mechanism would be inefficient and would only potentially develop after the failure of the middle CLT panels spanning two bays, which initially provide the primary alternate load path. Failure was observed in the beam-to-column connections and in the CLT panels detaching from the beams below, with the screws pulling out.

6– ACKNOWLEDGEMENT

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