Alternate Load-Path Analysis for Mid-Rise Mass-Timber Buildings

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ABSTRACT

This paper presents an investigation of possible disproportionate collapse for a nine-storey flat-plate timber building, designed for gravity and lateral loads. The alternate load-path analysis method is used to understand the structural response under various removal speeds. The loss of the corner and penultimate ground floor columns are the two cases selected to investigate the contribution of the cross-laminated timber (CLT) panels and their connections, towards disproportionate collapse prevention. The results show that the proposed building is safe for both cases, if the structural elements are removed at a speed slower than 1 sec. Disproportionate collapse is observed for sudden element loss, as quicker removal speed require higher moments resistance, especially at the longitudinal and transverse CLT floor-to-floor connections. The investigation also emphasises the need for strong and stiff column-to-column structural detailing as the magnitude of the vertical downward forces, at the location of the removed columns, increases for quicker removal.

KEYWORDS: Disproportionate Collapse, Progressive Collapse, Robustness, Structural Integrity, Tall Timber Buildings.

INTRODUCTION

The occupancy level and the intended use of the structure are baselines for collapse tolerances. For multi-storey buildings or high importance structures, such as military and federal buildings, the social and economic impacts of structural damages are of increased concerns. Here, the need for structural performance that avoids progressive collapses, also described as disproportionate collapse (CEN, 2006), is imminent.

The incidents of the World Trade Centre in 2001, and the Ronan Point building in 1968 are typical examples for disproportionate collapse. Although these extreme events have a low rate of occurrence, they do imply serious risks for human lives. The magnitude and complexities of the applied abnormal loads make design for absolute safety, with zero probability of failure, unrealistic and uneconomic (Ellingwood and Dusenberry, 2005). Since all buildings are liable to initial failures after extreme loads, building regulations, design guidelines, and current research, all focus on disproportionate collapse preventions. Most often, structural robustness is suggested as preferred method; it enables the structure to bridge over the initial damage, hence stops collapse propagation.

For timber buildings, the only available guidance is the Timber-Frame research project (TF2000), an experimental light-frame wood building tested to assess its structural performance after removal of wall sections (Milner et al., 2003). This study resulted to engineering bulletins which describe the tie force requirements, and propose structural detailing for structural integrity, hence robustness in light-frame wood buildings. The tie forces ensure continuity and redundancy in the building, which are characterised by the ability to develop resistance mechanisms for disproportionate collapse prevention. Nevertheless, the TF2000 guidelines become unrealistic for mid-rise timber buildings made of mass-timber panels such as Cross-Laminated Timber (CLT). Consequently, disproportionate collapse design preventions for mid-rise timber building, such as the UBC Tall Wood Building (Fast and Jackson, 2017), are left solely to engineering judgements and best practices.

ALTERNATE LOAD-PATH ANALYSIS

In EN1991-1-7 (CEN, 2006), abnormal loads are described as malicious actions of an unspecified source, to account for a broader and unpredictable scope. As a result, it is neither practical nor economic to design for all possible events that might occur during the life-span of a building. A threat-independent approach is recommended so the designer can focus on the structural performance rather than the extreme event itself. This method, in combination with the Alternate Load-Path Analysis (ALPA), avoids the complexities and limitations in modelling the abnormal loads. The ALPA, described in the American guidelines for disproportionate collapse Unified Facilities Criteria (UFC 4-023-03) (Stevens and Crowder, 2011), is utilised as a performance-based analysis approach for disproportionate collapse mitigations.

The ability to develop new load-paths to avoid collapse propagation is assessed after an initial damage is assumed, such as removal of loadbearing elements. ALPA helps obtaining the structural damage-to-performance correlation. In other words, it quantifies the forces and deformations required for disproportionate collapse preventions after the loss of critical loadbearing elements, assuming a threat of an unpredicted source and magnitude (Driver, 2014).

ALPA is a numerical method using Finite Element Analyses (FEA). UFC 4-023-03 (Stevens and Crowder, 2011) suggests different approaches from linear static to nonlinear dynamic analysis, with the latter being recommended as the most appropriate approach for more accurate results. Here, the model needs to account for geometric nonlinearities to capture large deformations from possible resistance mechanisms (Mpidi Bita et al., 2016). In addition, material nonlinearities are also required to mimic post-yielding behaviour, therefore giving realistic and economic approximations of the load distribution, and structural performance after removal (CPNI, 2011).

A sudden key element loss results in a dynamic behaviour; this does capture the influence of fast element failure relative to the response time of the structure (Fu, 2009). A nonlinear dynamic analysis, as a method of performing ALPA, allows applying significant deformations and forces on the members of the affected part of the building, as well as their connections, and hence allows estimating the demand in terms of strength, stiffness and ductility. For this reason, this paper performs a nonlinear dynamic analysis to investigate the structural robustness of a mid-rise timber building.

CASE STUDY BUILDING

Description

The case study was a nine-storey residential timber building with flat-plate structural system. The gravity system was composed of CLT floor panels resting on Glue-Laminated Timber (GLT) columns. The CLT panel behaved as a two-way system, taking advantage of its strength in the transverse direction. Figure 1 shows the building floor plan with the column locations identified from 1 to 19. As the floors were point-supported, a grid of 2.2 × 4.0m was chosen for adequate bearing, considering the current manufacturing limits with respect to CLT width.

Panels were 8.0m double-span, continuous over the middle supports. The core, designed as a lateral load resisting system, was composed of CLT walls where individual panels spanned 10.5m, assuming 3.5m per storey. In addition, the CLT floors acted as diaphragm for horizontal load transfer to the core.

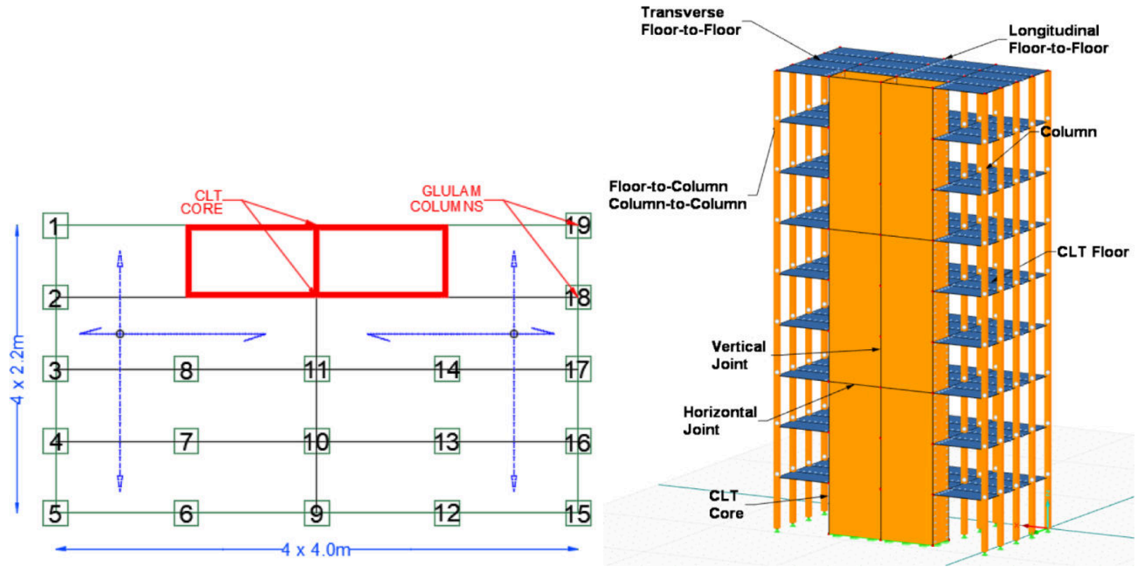

Figure 1: Building floor plan (left) and isometric view (right)

Mpidi Bita et al. (2017) considered the same case study, with less emphasis on the connection detailing, to investigate the structural behaviour after loss of key elements. The preceding study focused on possible resistance mechanisms following nonlinear static analyses, where ground floor columns were deleted without triggering a dynamic response on the structure. The results of the investigation identified the loss of corner columns as worst-case scenarios. It was also concluded that the floor connections helped towards collapse prevention. However, no investigations were performed to understand whether CLT panels themselves could contribute towards disproportionate collapse preventions.

For this reason, the present paper considered a nonlinear dynamic analysis; the model has hysteretic behaviours assigned at the connection level, for a realistic performance. The worst-case scenarios for the case study building, where the collapse resistance mechanisms would depend on the strength and stiffness of CLT panels, were defined as the damage of ground level columns. These were: i) single removal of column-5, and ii) single removal of column-6, as per Figure 1-left. The former required the panel to cantilever to prevent disproportionate collapse, whereas catenary action was the resistance mechanism for the latter.

This paper considered different removal speeds *(t)*, ranging between 5 seconds *(sec)* and 10^{-5}*sec*, to highlight the impact of *t* on the overall structural behaviour. Slower removal speeds would account for the static behaviour alone, which mimicked scenarios where the structural elements were simply deleted without inducing impact loads on the building. This was identical to the TF2000 experimental investigation, and the previous numerical study (Mpidi Bita et al. 2017). To capture the dynamic behaviours, the General Service Administration (GSA, 2013) recommends *t* faster than 1/10 of the period associated with the structural response mode for the vertical element removal. For the considered case study building, a *t* shorter than 0.01*sec* would induce dynamic behaviours.

Initial Gravity and Lateral Load Designs

The nine-storey case study building was designed as per the National Building Code of Canada NBCC-2015 (National Research Council Canada, 2015). It was assumed to be of normal importance with the 2015 Vancouver design spectrum. The superimposed dead *(SDL)*, live *(LL)* and snow *(SL)* loads were 0.7kPa, 1.9kPa and 1.82kPa, respectively. A 5-ply CLT floor panel, composed of 35mm thick layers was adequate to meet the ultimate and serviceability limit state requirements. The CLT stress grade was E3. A CLT bearing area of 300 × 300mm was required to avoid rolling shear failure. The GLT columns were 350 × 350mm 20f-EX. The core was 7-ply CLT of the same stress grade as the floor panels. The total dead load *(DL)* accounted for both the weight of all timber elements, assumed as 4.2kN/m^3, and the *SDL*.

Equivalent static force procedure, as per NBCC-2015, was performed for lateral design. The peak ground acceleration for 2% in 50 years probability was 0.366g and the soil type was class-c. The critical Rayleigh damping ratio was assumed to be 3%, typical for timber structures (Fallis et al., 2011). The first period of vibration was 1.12*sec*, corresponding to an acceleration of 0.40g, a mass participation of 67%, resulting to a base-shear of 400kN. The resulting inter-storey drifts were below 2.5% of the storey height, with a torsional sensitivity close to 1.0 in both directions.

Numerical Model

The nine-storey building was modelled in ANSYS (ANSYS, 2011), a commercial FEA software, to perform disproportionate collapse analysis. CLT walls and floor panels were modelled using 2D shell elements (Shell-181) with both membrane and bending stiffness. The idealised panels had sections defined by different layers; the top running longitudinal to the span and the remaining oriented crosswise. The panels were continuous over the middle support, and uniform in the transverse direction. The columns were represented by beam elements (Beam-188), with solid cross-sections.

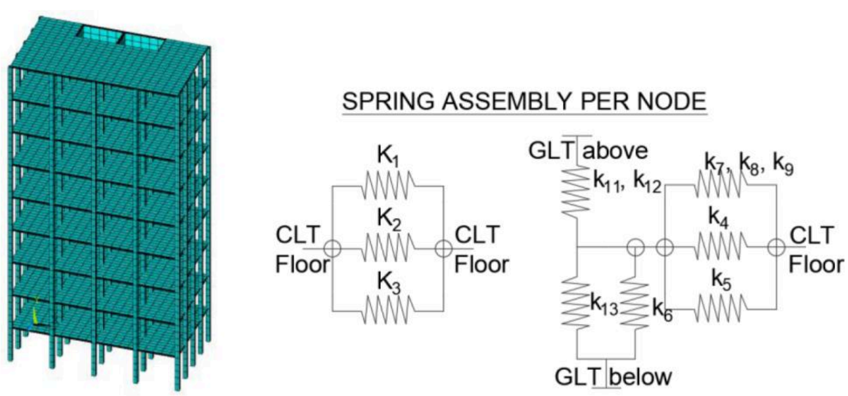

Figure 2: FE model of the 12-storey building (left), floor-to-floor connection (Centre), and column-to-column and floor to column connection (right)

The connections between all structural components were idealised by an assembly of uniaxial nonlinear/linear spring elements (Combin-39 and Combin-14) that could independently contribute to the overall behaviour of the connection through their assigned individual properties. This is the most complete and realistic approach of obtaining accurate connection behaviour representing each deformable region separately (Byfield et al., 2014). Furthermore, this approach drastically reduced computational times, and enabled to obtain an accurate representation of the building performance following the removal of key elements. Figure 2- left shows the FEA model of the 12-storey building.

Figure 1-right shows the main structural elements as well as their connections. To meet the capacity-based design approach, the horizontal joints of the CLT core were coupled using rigid constraints to transfer both forces and moments, as an idealisation of a moment connection. The vertical joints forming the corner connection were constraint for translation only. The floor-to-floor joints representing the surface plywood spline connection were idealised by 3 independent springs for each node: i) 2 nonlinear springs for horizontal shear in the two horizontal directions *(k_1 and k_2)*, and ii) 1 nonlinear spring for withdrawal resistance *(k_3)*. Figure 2-centre shows the corresponding spring assembly per node. The floor-to-column connections were represented by iii) 2 nonlinear springs *(k_4 and k_5)* for the horizontal shear in both directions, iv) 1 nonlinear spring for bearing and uplift *(k_6)*, v) 3 rotational springs *(k_7, k_8 and k_9)* for moments about all three orthogonal axes. The same spring assembly was used for floor-to-wall joints.

Linear springs *(k_{10})* were utilised to represent drag straps in their corresponding directions and locations. The column-to-columns were idealised by: 6) 2 linear springs *(k_{11} and k_{12})* to restrain the horizontal movements, and 7) 1 nonlinear spring *(k_{13})* for bearing and uplift. Figure 2-right shows the corresponding spring assembly per node. Therefore, there was a total of 9 springs per nodes to represent the column-to-column and floor-to-column detailing. With the mesh size of the floor, which generated 6 nodes per panel, 18 springs were assigned for the floor-to-floor connection. In the same manner, the floor-to-core connection required 36 springs with the same mesh size. It should be noted that the decision behind the use of nonlinear or linear springs was based on possible resistance mechanisms triggered after the removal of key elements. The behaviours helping the resistance were assigned nonlinear material properties, calibrated against available experimental test results.

Connection and material properties

ANSYS Combin-39 element type was used for all uniaxial nonlinear springs, with the option allowing unloading along the line parallel to the slope at the origin of the curve, to capture hysteretic behaviours. The properties for STS in shear and withdrawal were calibrated using the Equivalent Energy Elastic Plastic (EEEP) curves of the experimental results (Hossain et al., 2016). The values in Table 1 represent the properties of a single STS under the considered loading. Here, F_y and Δ_y are the yield force and displacement, respectively, whereas F_u is the ultimate force and Δ_u is the ultimate displacement obtained from testing. The rotational stiffness of the floor-to-column and floor-to-core were assigned 1,000kNm/rad as recommended by Mpidi Bita et al. (2016). For bearing and uplift resistance, the linear springs Combin-14 were given a stiffness of 10^9kN/m.

Table 1: Properties for single STS testing (Hossain et al., 2017)

Components	F_y [kN]	Δ_y [mm]	F_u [kN]	Δ_u [mm]
STS loaded in shear	5.9	10.5	5.9	54
STS in withdrawal	12.7	1.8	12.7	17

Quasi-static tests were conducted at FPInnovations to estimate the stiffness, strength, deformed shape, and failure mode of point-supported CLT floors in bending (Popovski et al., 2016). A FEA model representing to the tested specimen was constructed for model calibration. The material properties were assumed linear elastic transverse isotropic, see Table 2. Here, *(E)* is the Young's moduli, *(G)* is the shear moduli, and *(U)* is the poison ratio for the longitudinal *(L)* and transverse *(T)* direction of timber.

Table 2: Elastic material properties of the CLT panels using in ANSYS

Layer Direction	E_L [MPa]	E_T [MPa]	G_L [MPa]	G_T [MPa]	U_L [~]	U_T [~]
Longitudinal	8,300	E_L /30	E_L /16	G_{LT}/10	0.35	0.07
Transverse	6,500	E_L/30	E_L /16	G_{LT}/10	0.35	0.07

Disproportionate collapse thresholds

The thresholds defining disproportionate collapse, also referred to as Limit States Functions (LSFs), were the maximum allowable deformations of the CLT panels (δ_{max}) at the location of the removed elements, associated with vertical downward point loads *(P)* that would lead to element failure. δ_{max} and P were defined in such a way that stress and strain on the structural elements always remained within their respective thresholds, according to the Canadian Standard for engineering design in wood CSA-O86 (CSA, 2016). In other words, the resistance mechanisms for disproportionate collapse prevention should avoid any sort of failure within the CLT panels. These thresholds were defined as a series-system; violation of a single LSF resulted to the failure of the entire system.

Figure 3 illustrates the idealisations of corner (top) and middle (bottom) columns removal scenarios, corresponding to loss of column-5 and column-6 (as illustrated in Figure 1-left), respectively. For the proposed floor plan configurations and dimensions of the case study building, it was found that no failure would occur in both removal cases considering the NBCC-2015 extreme load combination (1.0*DL* + 0.5*LL*) alone. To prevent disproportionate collapse, the floor systems triggered cantilever or catenary action for corner or penultimate support removal, respectively.

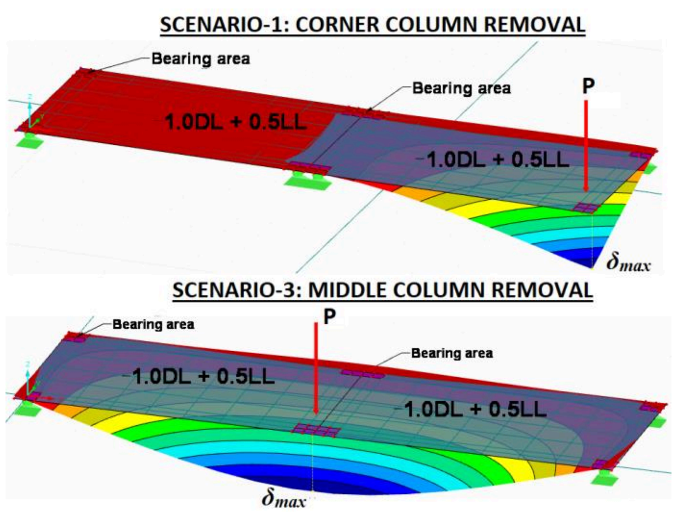

Figure 3: Element removal scenarios: corner column (top) and middle column (bottom)

An additional 2kN as *P* resulted to failure dominated by a combination of rolling shear and tension stress perpendicular to the grain of the transverse layers, for the case of corner support (column-5) removal, at a deformation of 22mm. After penultimate support removal (column-6), applying 2kN as *P* in addition 1.0*DL* + 0.5*LL*, caused the collapse of the floor system at a deformation of 30mm. Although the CLT was continuous over the internal support, rolling shear failure was the dominant failure. For both removal scenarios, failure in bending only occurred for *P* greater than 10kN. Therefore, the deformation thresholds or LSFs defining a disproportionate were: i) 22mm as δ_{max} for column-5 removal, and ii) 30mm for column-6 removal.

RESULTS AND DISCUSSION

Column-5 Removal

Figure 4-left shows the deformed shape of the building after removal of column-5. For the static case, without considering any dynamic effects, a deformation (δ_{max}) of 19mm was recorded at the top of the removed element. For this scenario, the *P* dropped to 10N, from 54kN noted before removal. The loss of the structural component at time *(t)* shorter than 5*sec* triggered a dynamic response, which resulted to an increase of $\delta_{max,}$ as well as the forces at connection level. However, the structural response up to t = 1*sec* showed negligible difference to the static case. For removal speeds slower than 1*sec*, no LSF was violated; and it was concluded that structural safety was provided. For *t* faster than 1*sec*, e.g. t = 0.1*sec*, the recorded δ_{max} was greater than 25mm. The imposed deflection highlighted failure in rolling shear of the transverse layers of the CLT panels, which was noted by the observed stresses at the nearest supports.

For *t* beyond 0.05*sec*, the analyses exhibited convergence problems; no results were obtained for *t* quicker than 10^{-2}*sec*. For the considered removal scenario, both longitudinal and transverse CLT floor-to-floor connections were critical for disproportionate collapse prevention. It was found that the lack of moment resistance from the proposed plywood spline connections between CLT floor panels, especially in the transverse direction, was the main reason behind the observed issues. Magnitudes of around 4kNm and 2kNm per meter width of the panel were recorded as applied moments in the transverse and longitudinal directions, respectively. However, the axial and shear forces supplied by the proposed connections were sufficient to carry their respective demands. Therefore, for the considered case study building, a disproportionate collapse occurred if extreme event resulted to the loss of column-5 at a speed quicker than 10^{-2}*sec*. This would be observed by the failure of CLT panels, due to the imposed deformations, as well as connections failure noted by the lack of moment resistance.

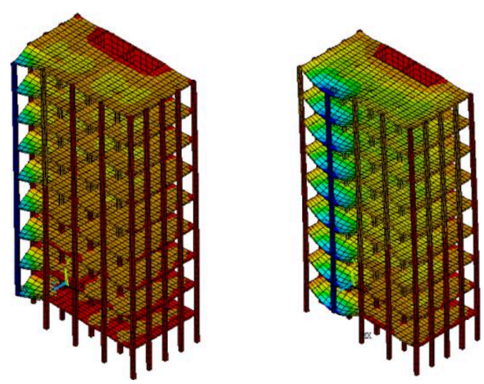

Figure 4: Deformed shape: Column-5 removal (left) Column-6 removal (right)

As an approach for disproportionate collapse mitigation, keeping the same model with respect to the size of the structural components, an upper bound for design purposes assumed that all connections were rigid, with full force and moment transfer. It was observed that with the joints constrained for all movements, δ_{max} was reduced to 12mm for the static case. Figure 5-left shows the change of the deflection with respect to time, after removal. It was noted that 10^{-1}*sec* would result to a deflection of 15mm; further 3mm increase was observed for 10^{-4}*sec* as removal time. δ_{max} had a constant value of 18mm for quicker removal, as changes in t led to negligible increase in the applied formation at the location of the removed element. Therefore, with respect to δ_{max}, the building remained safe, regardless the selected removal speed. It could then be concluded that stiffer moment connections helped satisfying the deformation LSF following the loss of column-5. Improved CLT floor-to-floor connections in both directions would help the individual panel to cantilever over the damage; here the stiffness of the CLT panels was less significant for failure resistance.

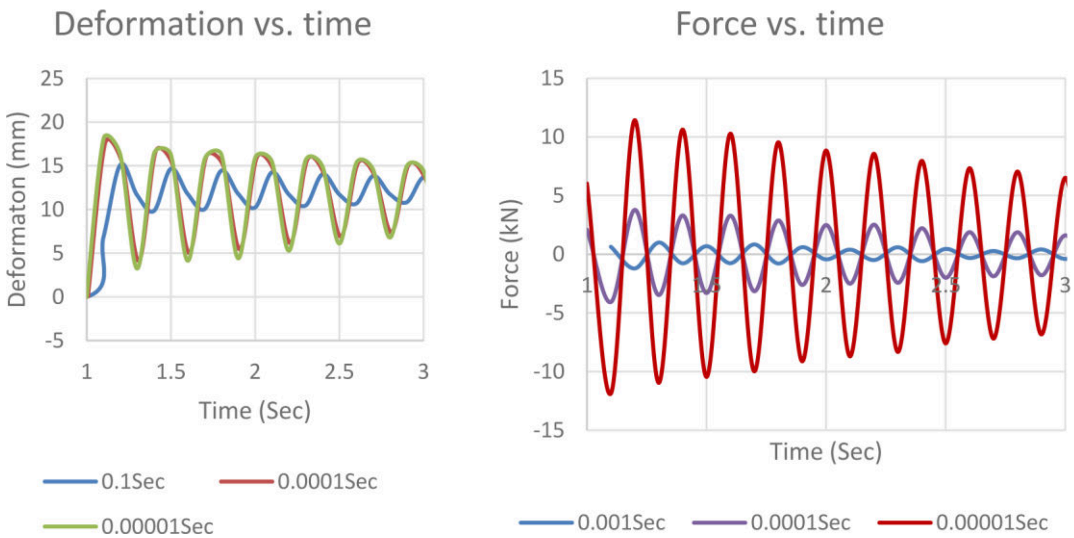

Figure 5: Column-5 removal: Deformation vs. time (left) and Force vs. time (right)

For the improved model with rigid connections, Figure 5-right shows that the resulting forces *(P)*, on top of the removed element, was also highly affected by the removal time. The reduction of the applied vertical deformation (δ_{max}) in the improved model, observed in Figure 5-left, resulted to an increase of forces at the connections. However, for removal times slower than 10^{-2}*sec*, changes of P could be neglected as its magnitude was less 1kN. For removal quicker than 10^{-4}*sec*, the column-to-column connection should be designed to carry a P of 4kN in tension in order to prevent collapse. This is also known as hanging action. An extreme event that could cause the loss of column-5 in 10^{-5}*sec* would trigger a tensile force of 12kN at the column-to-column connections. The structural detailing should also account for anticipated tensile forces to prevent collapse propagation.

Table 3: Forces and Moments at top floor for different models

Model	Longitudinal joints		Transverse joints		δ_{max}	Failure
	F [kN]	M [kNm]	F [kN]	M [kNm]	[mm]	[~]
Original	0.2	1.1	1.1	4.4	26	YES
Improved	1.7	5.7	2.5	10.1	16	NO

Table 3 helps for comparison between the original and improved models. There was no difference between the two models in terms of the size of the structural elements; the only change came in the strength and stiffness of the connection detailing. For the improved model, the connections between the structural components were assumed fully rigid, whereas the original model used the material properties in Table 1. The values for forces *(F)* and moments *(M)* were recorded at the top floor longitudinal and transverse CLT floor-to-floor joints for 0.05*sec* as removal speed. The results confirmed that in order to prevent disproportionate collapse after the loss of column-5, by keeping δ_{max} below the set thresholds, connections would require moment resistance higher than the supply from plywood spline connections.

Column-6 Removal

Figure 4-right shows the deflected shape of the building after removal of column-6. A maximum deformation (δ_{max}) of 19mm was obtained from the static case. Here, the building could redistribute the forces to the undamaged part of the building as no LSF was violated. The catenary action of the floor system was the anticipated resistance mechanism against disproportionate collapse. For removal time t slower than 1*sec*, the vertical downward forces on top of the removed element dropped from 107kN to a value less than 1kN, although an increase in δ_{max} was observed. Analyses with quicker t exhibited convergence problems. No solutions were obtained beyond $t = 0.05sec$, though deflections of less than 30mm were found. The demands in terms of moments at the longitudinal and transverse CLT floor-to-floor connections were beyond the supply from plywood spline connections. Therefore, an extreme event that could cause the loss of column-6 in 0.1*sec* or quicker would lead to a disproportionate collapse due to insufficient moment capacity at connection level.

Just as done for column-5 removal, the connections between different structural elements were constrained for all movements to solve convergence issues. The static case of the improved model of the building resulted to δ_{max} of 11mm. Stronger and stiffer joints, with moments resistance, resulted to the reduction in the applied deformations, and therefore resistance mechanism. In other words, the catenary action of the CLT floor panel, although continuous over the internal support, would mainly depend on the floor connections. To prevent disproportionate collapse when $t = 10^{-2}$sec, the longitudinal and transverse floor-to-floor joints should resist moment higher than 14kNm and 8kNm per meter width of the CLT floor panel, respectively. The column-to-column connection would need to resist a tension force of 24kN to prevent collapse propagation.

CONCLUSION

This paper presented FEA to investigate the possibilities for disproportionate collapse for a nine-storey flat-plate timber building, following the sudden removal of corner and penultimate ground floor columns. This was a nonlinear dynamic analysis, performed to capture the structural response for different removal speeds. It was found that the building was safe if the selected columns were removed at a speed of 0.1*sec* or slower. The lack of moment resistance of the connections, especially for the CLT floor longitudinal and transverse joints, was the main problem highlighting disproportionate collapse possibilities for quicker removal. It was found that stiffer connections, with moment resistance would reduce the deflection at the location of the removed element, hence enable the catenary and cantilever actions as resistance mechanisms. Furthermore, this study also identified the need for column-to-column structural detailing to trigger hanging action as sudden key element removal also increased the vertical downward forces at the location of the removed element.

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