ASSESSMENT OF DISPROPORTIONATE COLLAPSE FOR MULTI-STOREY CROSS-LAMINATED TIMBER BUILDINGS

Hercend Mpidi Bita[1], Neil Currie[2], Thomas Tannert[3]

ABSTRACT: This paper investigates the risk of disproportionate collapse following extreme loading events. The methodology mimics a sudden removal of a loadbearing wall of a twelve-storey CLT building. The ductility-demand from the dynamic simulation is checked against the ductility supplied by the structural components and their connections. The analyses focus on rotational stiffness (k) of the joints by considering three different sub-structural idealisations according to the required modelling details and the feasibility of model reductions. To resist the imposed dynamic forces, the required *k-values* may be too large to be practically achieved by means of off-the-shelf brackets and screw connections. Improved structural detailing as well as adequate thickness of structural elements need to be considered in order to reduce the probability of disproportionate collapse.

KEYWORDS: Progressive collapse, robustness, structural integrity, redundancy, rotational stiffness, sudden removal

1 INTRODUCTION

Failure of individual structural elements after extreme loading events, such as explosions, is understandable. However, concerns arise when this initial damage spreads in a successive manner over a major part of the building. This is known as progressive collapse. When the final damage goes beyond acceptable thresholds, whether it happens immediately or progressively after the initial damage, this type of failure is classified as disproportionate [1]. In EN1991 [2], such thresholds are that the final collapse affects less than 15% of the floor area of the affected storey or 100m^2, whichever is less, and does not extend further than the immediate adjacent storeys.

Collapse of a residential house does not have the same social and economic impacts if comparison is made against the failure of a multi-storey building. In EN1991 (Part 1-7 section A.1) classifications are done with respect to the height, occupancy level and intended use of the building [2]. The importance of these categorisations was first required after the 1968 failure of the Ronan Point apartment building in London [3]. An explosion caused the failure of the external loadbearing wall, which in turn triggered the collapse of a major part of the building. Identical failures during the past decades, most prominently the World Trade Centre incident, brought new attention regarding the consideration of disproportionate collapse in design and construction of tall buildings [4].

1.1 STRUCTURAL ROBUSTNESS

The first stage towards a design against disproportionate collapse is to account for structural robustness [5]. A building is robust if it is able to find new load-paths, after one or many structural members have become ineffective, and consequently remains stable as a whole. In other words, the structural system can develop a new equilibrium state to redistribute the loads to the undamaged parts, and ultimately stop the initial damage from spreading beyond the acceptable collapse thresholds. This can only be possible if redundancy and structural integrity are considered as key design factors.

The availability of alternative load-paths within a structural system results in a redundant building. This redundancy depends on the topology of structural elements, continuity between them and ductility of their connections [6]. Possible approaches are to place vertical loadbearing elements at closer spacing, to provide adequate ties between all structural members or to design the joints to sustain large deformations. Nonetheless, the building can only behave as a whole if adequate stiffness at connection level is provided in addition to continuity and ductility. With sufficient structural integrity, the building can develop resistance mechanisms that signal impending failure, and avoid disproportionate collapse.

[1] Hercend Mpidi Bita, Wood Sciences, The University of British Columbia, hercend_mb@alumni.ubc.ca

[2] Neil Currie, Computing, Science and Engineering, The University of Salford, n.g.r.currie@salford.ac.uk

[3] Thomas Tannert, Wood Science and Civil Engineering, The University of British Columbia: thomas.tannert@ubc.ca

Figure 1 shows catenary action as an example of resistance mechanism. It enables for suspension of the floor above the ineffective vertical loadbearing element, hence avoiding debris loading on the floor below. To be able to hang, the components shall carry tension forces. These forces have to be compatible with the amount of deformation, expressed in terms of rotation, required at connection level [7]. For adequate design, the requirements for robustness are embodied within design Standards, providing guidance against disproportionate collapse.

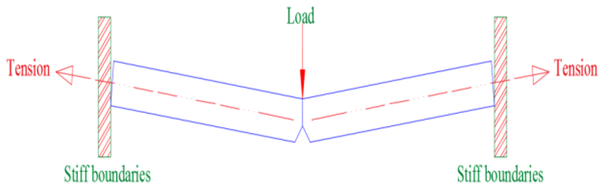

Figure 1*: Floor elevation showing catenary action*

1.2 DESIGN STANDARDS

Eurocode 1 (EN1991) is detailed and prescriptive [2]. To design for adequate tension forces, the notional tie force requirements is employed. As an example, for loadbearing walls, the minimum tie forces between internal floor components is estimated to the lesser of 60kN/m or $(20+4n_s)$kN/m, where n_s is the number of storeys. These prescriptions apply to buildings of any height and proportion. This Eurocode approach is considered as an indirect approach; in other words, if satisfied, no further analyses are required for disproportionate collapse preventions [8].

In the United States, for Minimum Antiterrorism Standards for Buildings, the Unified Facilities Criteria (UFC 4-023-03) gives guidance with more details on analytical methods and design requirements [9]. As an example, for laterally restrained reinforced concrete slabs, the recommended safe value of central deflection is estimated from 10% to 15% of the shortest span of the building; beyond this, the structure is unsafe [10]. UFC 4-023-03 also provides analysis techniques, depending on complexity, from linear static to nonlinear dynamic analysis. Clause 4.1.1.3 of the National Building Code of Canada requires structural integrity considerations in the design of buildings as an approach towards disproportionate collapse preventions [11]. The code relies on CSA material standards for structural detailing such as CSA-O86 [12] in the case of timber. Therefore, timber, as a structural material, has to be studied independently.

1.3 GUIDANCE FOR TIMBER STRUCTURES

In the United Kingdom, design guidance can be derived from the Timber Frame 2000 (TF 2000) project; a six-storey experimental building subjected to a rigorous test programme to assess its performance against disproportionate collapse [13]. Sections of a loadbearing wall were delicately removed to check the magnitude of forces at connection level. The test led to provisions based on tie force requirements. These prescriptions are limited to six storeys and it is unclear whether extrapolation is possible to other structural systems of different proportions and heights [14]. In Canada, since the NBCC relies on the detailing provided within the CSA-O86, to satisfy structural integrity, no information is available to confirm the behaviour against disproportionate collapse for multi-storey timber building taller than six storeys [12]. With these limitations, designers use different approaches to analyse and verify structures under extreme loading scenarios.

The compliance requirements for multi-storey timber structures are not explicit and left to sound engineering judgements. The Stadthaus apartment, an eight storey building in London, is a typical example of multi-storey CLT building using platform construction [15]. For this building, redundancy in addition to the alternative load-path method was the ideal design strategy. The designer had to proceed to the removal of single wall or floor panels, one at the time, in order to check whether the subsequent failure is beyond EN1991-1-7 collapse tolerances. For efficient redundancy of the considered structural system, floor panels were designed to span in two directions wherever possible otherwise, they should cantilever if the support underneath was to be removed [15]. This approach also accounted for the high in-plane stiffness of the CLT panels which helped wall elements to act as deep beam in event of support removal.

The alternative load-path method is an addition to the indirect approach and therefore requires additional analyses, which are only appropriate to the building in question, in order to understand the structural behaviour at global, components and connections levels. As a consequence, there is the need for immediate guidance for both general concepts to handle the question of disproportionate collapse and provisions for designing multi-storey timber structures in general, which perform well after an extreme loading event.

2 FINITE ELEMENT INVESTIGATION

2.1 MULTI-STOREY CLT STRUCTURES

The ability of CLT to be used as floor and wall elements opens up possibilities for them to be considered as main structural component for multi-storey timber building concepts. Albeit their positive performance with respect to fire resistance, strength, stiffness, and durability, investigations on structural robustness are required for resistance against disproportionate collapse. By their very nature, CLT wall systems are only as stiff and strong as the connections between the individual CLT elements [16]. This means that adequate joints detailing is required to develop resistance mechanisms. This paper assumes that sufficient continuity is provided between structural elements and therefore focus needs to be on the rotational capacities at connection level to maintain structural integrity. The rotation of joints directly influences the maximum vertical displacement at component level. This can be quantified in terms of rotational stiffness [17].

The hypothesis of this study is that the deformation capabilities are dictated by the rotational stiffness. The basic principle is that resistance mechanisms against disproportionate collapse are determined by rotational deformation-demand on the joints, following an extreme loading events, in relation to the rotation they can supply.

2.2 DESCRIPTION OF CASE STUDY BUILDING

This study considered a twelve-storey CLT building as a case study to understand its ability to resist disproportionate collapse after extreme loading events. The structure was a TRADA example, designed to serviceability and ultimate limit states in accordance with EN1995 [18].

The building, shown in Figure 2, was 9m x 9m, a cross-wall or platform construction, comprising two spans with one central loadbearing wall in one direction [19]. The floor slabs and the external walls cross-section depth was 125mm and the thickness of the internal walls was 135mm. The design had no provisions for openings hence all panels were assumed to be solid and continuous over the entire length. Only the floor was made of two different panels resting on the internal walls. The storey height was 3m and all walls are loadbearing.

It was proven that the building had the required strength to take the design loads such as permanent, variable, wind and snow loads. The stability checks achieved all relevant safety factors, holding down straps were not required and fixing to resist sliding were normal. The structural design and detailing have shown the feasibility of the building however, further checks for disproportionate collapse were still required [19].

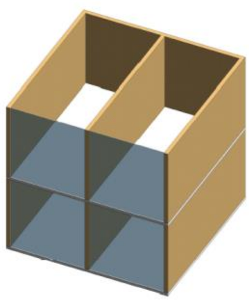

***Figure 2**: Isometric view of the building*

The aim of the study was to verify whether the twelve-storey building would be able to maintain its structural integrity after the loss of the internal ground floor loadbearing wall. Assuming adequate continuity is provided, as the structure did not have many vertical elements, focus was on rotation ductility in order to achieve a system with sufficient structural integrity. Here, this was expressed in terms of rotational stiffness of the connections. Figure 3 shows the chosen structural detailing used for the wall-to-floor and floor-to-floor joints. This was the same detailing provided for the Stadthaus apartment.

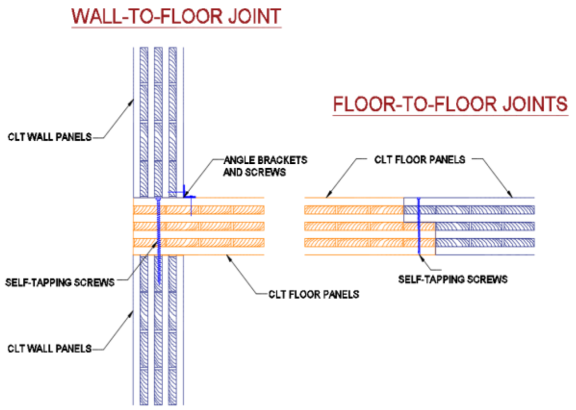

***Figure 3**: Structural detailing of joints*

It was assumed that the joints would control the overall strength and stiffness of the building; therefore the loads and deformation at joint level, following extreme loadings, would be taken as the demand. Comparison was made against strength and deformation supplied by the provided connections. Disproportionate collapse could only be avoided if the supply was higher than the demand and the resulting deformations was smaller than the collapse thresholds. These were assumed to be 10% of the floor span, representing 900mm as vertical deformation at mid-span.

For first detailing, off-the-shelf angle brackets with screws were used to connect the floor and the wall above. The minimum screws patterns, according to the European Technical Approval ETA-06/0106, were followed [20]. Angle brackets 90mm wide without rib were placed at 500mm centre to centre. Self-tapping screws, ASSY 3.0 [21], were used to connect the floor and the wall below. These screws had 8mm diameter, located at 500mm centre to centre, and inserted at 90-degree to the CLT panel to act in shear under the floor loadings. The same configuration, using self-tapping screws, was used for the second detailing to connect the two floor panels.

2.3 METHODOLOGY

The alternative load-path method, recommended in EN1991 [2], accounting for nonlinear and dynamic behaviour, was the preferred methodology. The loss of the internal ground floor loadbearing wall was considered as initial damage after an extreme loading event. It was assumed that the applied loads have the magnitude required to make the structural element completely ineffective. There is no need for further details on the type or source of the applied loads [6]. The idealised event-independent scenario was considered to be instantaneous; this created an impulsive load on the structure. The speed of removal would incorporate dynamic effects of key elements failure over a short duration when compared to the response period of the structure. In addition, nonlinearity in the analysis was accounted by large deformations and post-yielding behaviour on the material properties of certain elements such as self-tapping screws.

Finite Element Analyses (FEA) were performed in Ansys 12 [22]. As shown in Figure 4, the method was a multi-level assessment where investigations were done at three different sub-structural idealisations; which were global, macro and micro levels. The modelling details and complexities dictated the feasibility of the model reductions. At the Global level, focus was on the overall structural behaviour of the building in terms of load combinations, deflected shape, vibrations frequencies, mode shape and damping ratio. It is on this model that the internal wall was removed instantaneously to trigger the dynamic effect on the structure. This level of idealisation defined the magnitude of the forces transferred to the joints at every storey as well as the associated deformations. The FEA results obtained at this stage were considered as the demand on the structure. Since this was the upper bound, it was assumed that the joints were fully restrained, and CLT elements were taken as linear orthotropic.

With appropriate boundary conditions, provided that surrounding structural elements could redistribute the forces after the removal of the internal wall, a reduction to micro models was possible. Taking advantage of symmetry, the Micro-models 1 and 2 were idealised to understand the contribution of the detailed joints. First, 3D-models were constructed, with all screws and angle brackets, to capture a realistic behaviour of the connection. This was important as the failure of the floor is largely influenced by the maximum deformations as well as forces that the joint could carry. Results from these models were considered as the supply; which needs to be compared against the demand from the global model. For this reason, the 3D-models, the lower bound, considered the contribution of different layers within the CLT panels, assuming they would always be bounded to reduce nonlinearity. Linear orthotropic material properties were used for CLT elements whereas the screws and angle brackets used a bi-linear backbone curve. Also, with the high forces from the global-model, the analysis accounted for large deformations.

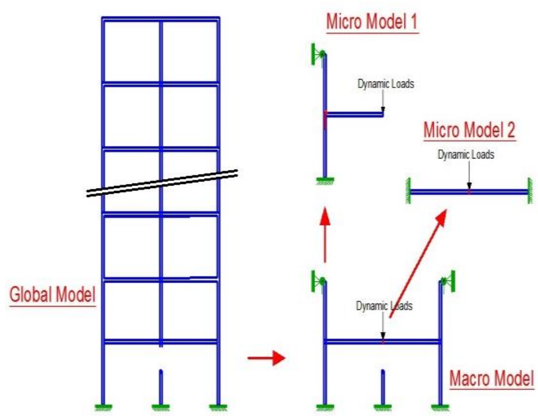

Figure 4*: Multi-level assessment approach*

The 3D models were highly nonlinear with numerous convergence problems due to their size and complexity. It was anticipated that no solutions would be obtained if the full dynamic loads from the global-model were to be applied. To go about the problem, 2D-models were constructed and calibrated to mimic the same behaviour as the 3D Micro models. Emphasis was given on the rotational stiffness of the connections to quantify their rotational capabilities. Joints were idealised by rotational spring elements.

A solid relationship between 3D and 2D Micro-models was necessary as the latter needed to be modified in order to carry the dynamic loads. For the 2D Micro-models, as illustrated in Figure 5, it was assumed that k_1, k_2 *and* k_3 would represent the rotational stiffness of the floor-to-wall above, floor-to-wall below and floor-to-floor, respectively.

The Macro-model was required to understand the structural behaviour at sub-frame level. Here, only 2D-model was constructed since idealisation was based on the properties from micro-models. For it conception, as shown in Figure 5, the values of the rotational stiffness k_1, k_2 *and* k_3 were used. The macro-models simulated the interaction between the two floor segments, right above the removed internal wall, and captured possible resistance mechanisms against disproportionate collapse. It was possible to mimic the catenary action and load transfer which helped reducing the rotational stiffness values required at the wall-to-floor connections.

2.4 INPUT DATA FOR NUMERICAL MODELS

The accidental load combination, giving a total load (*W*), was taken from the British Standards BS 8110 [22] and BS 5950 [23]. Equation 1 accounts for a reduction of the imposed load in case of extreme loading events. The characteristic permanent and imposed loads were G_k and Q_k, respectively. All floors would carry imposed loads for residential buildings, 1.5kPa, as described in EN1991 [2]; and the snow load would be ignored in this load combination. The permanent load on floors was estimated to 1.4kPa, including density, the latter estimated to 480kg/m^3.

$$W = 1.05 \times \left(1.0G_k + 0.33Q_k\right) \tag{1}$$

Self-tapping screws and angle brackets were made of steel with characteristic yield strength of 1000MPa and 240MPa, respectively [20, 21]. It was assumed that all CLT panels are 3-ply with the material properties [16] given in Table 1. Friction between CLT panels, modelled as contact elements, was assumed to be 0.3. This was important to account for proper bearing between panels hence reduce penetration between elements during analysis.

Table 1*: CLT orthotropic material properties*

Elastic Modulus [MPa]	Poisson Ratio [~]	Shear Modulus [MPa]
$E_x = 8{,}000$	$v_{xy} = 0.35$	$G_{xy} = 700$
$E_y = 500$	$v_{xy} = 0.35$	$G_{xz} = 700$
$E_z = 500$	$v_{xy} = 0.04$	$G_{yz} = 70$

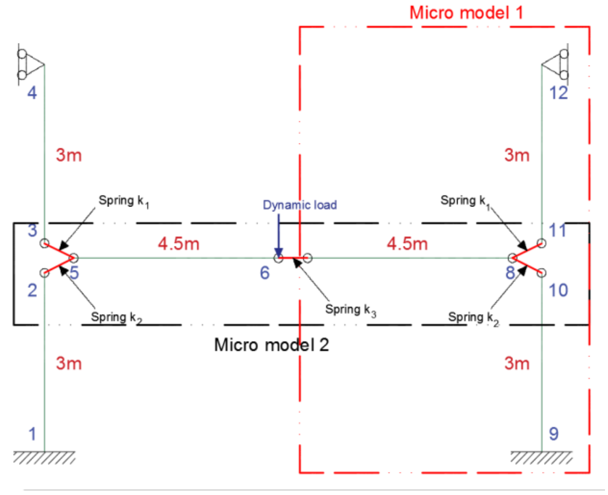

Figure 5*: Boundary conditions for Macro model*

Figure 5 illustrates the boundary conditions used for modelling at the 2D Macro-model. The locations of nodes were numbered from 1 to 12. Nodes 2, 3 and 5 (similarly 8, 10 and 11) had the same coordinate and were constrained in the two orthogonal directions, x and y. This enabled them to have the same deformations. The same constraints were used for nodes 6 and 7. In addition, only the permanent loads from the floors above acted as favourable load on node 4 and 12 to improve the resistance of the sub-frame. From the Macro-model shown in Figure 5, both Micro-models 1 and 2 were extracted. However, for the latter, fully fixed boundaries were assigned at node 5 and 8 to emphasise on the rotational stiffness between the two panels.

2.5 SUDDEN ELEMENT REMOVAL ANALYSIS

A static analysis was first required in order to estimate the total forces on the building and the magnitude of loads on top of the removed element. This estimate was used to replace the ground floor internal wall to mimic the normal condition, which is before extreme loading, in the dynamic analysis. Thereafter, using the total mass of the building, a modal analysis was run to evaluate the natural frequencies of the building. Here, focus was on mode shapes with vertical motions near the removed element. The modal analysis was performed without the ground floor internal wall in order to capture the frequencies of vertical motions. Furthermore, the building frequencies was important to account for Rayleigh damping in the analysis. This is calculated as ($\alpha M + \beta K$) where M and K are the mass and stiffness of the building, respectively; and α and β are coefficients. For the dynamic analysis, it was assumed that the critical damping ratio would range from 3% to 5%.

Four different load-steps were applied to perform the dynamic analysis which mimics the sudden loss of the ground floor internal wall. In the first load-step, all the loads, including gravity and nodal force replacing the removed element, were applied in a single step size. For load-step 2, since this step represents the normal condition of the building before extreme loadings, a longer time step size was assigned with 100% as critical damping. This was done in order to give sufficient time for the structure to regain a static equilibrium similar to the normal condition. The nodal force was deleted at load-step 3 and the time step size corresponded to the speed of element removal. For the last load-step, the value of critical damping was brought back between 3% and 5%, to observe the time required for the building to damp away the dynamic forces and therefore regain static equilibrium. It is at this level that the maximum dynamic load and deformation, corresponding to the demand on the structure, were extracted.

2.6 RESULTS GLOBAL MODEL

2.6.1 Static and modal analysis

Static analysis of the building gave a total vertical load of 1863kN on top of the internal ground floor wall. For the dynamic analysis, this load was distributed at the location of the removed element to mimic the normal condition, before extreme loadings. For static analysis performed without the ground floor internal wall, the horizontal and shear loads on the floor, at wall-to-floor joint, are 180kN and 144kN, respectively. At 0.9m away from the joint, the rotation was 0.13rad with a vertical deflection of 0.064m. This analysis gave a maximum deformation of 0.494m at the location of the removed element. Thereafter, a vibration analysis was run to obtain the natural frequency of the building.

The modal analysis estimated the total mass of the building to 456tonnes, with 0.77Hz has natural frequency and 4.71Hz the frequency with more than 99% mass participation in all three orthogonal directions. The mode shapes captured between those two frequencies were dominated by vertical motions of the floors near the removed element. Figure 6 shows the mode shape of the building corresponding to its fundamental frequency.

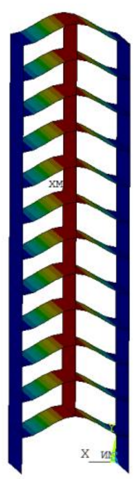

Figure 6*: Mode shape at fundamental frequency*

2.6.2 Dynamic analysis

The first analysis considered 0.001sec as speed of removal (t) and 3% critical damping ratio. Figure 7 shows how the floor oscillate after the sudden removal of the internal wall. Illustrating the rotation, 0.90m away from wall-to-floor connection. Here, the maximum rotation was found to be 0.24rad, giving a vertical deformation of 0.91m at the location of the removed wall. These values were obtained 0.7sec after removal. Furthermore, the difference between 3% and 5% critical damping ratio, with respect to forces and deformations, was small hence could be neglected. For both 3% and 5%, 7sec were not sufficient for the structure to regain a static equilibrium. This was only possible with a critical damping ratio above 30%. Since the analysis considered linear material properties, the equilibrium geometry from dynamic solution would be the same as the static analysis of the structure without the internal wall.

Figure 7 also emphasises the difference of the structural response with respect to the speed of removal of the internal wall. Here, comparison was made between 0.001sec and 1sec, a bigger rotation, associated with high dynamic motions, was obtained for quicker removal. The graphs also confirm the fundamental period of the structure, 1.3sec, which correlated well with to the results obtained from the modal analysis.

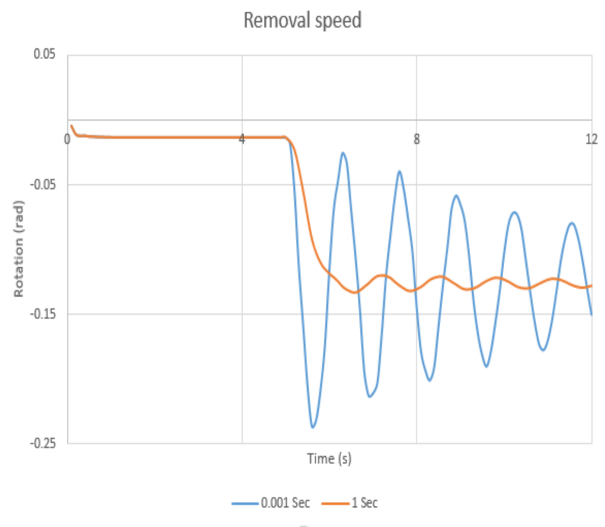

Figure 7*: Joint rotation and speed of removal*

Figure 8 shows the behaviour of forces at wall-to-floor connection for 0.001 sec as speed of removal. The maximum shear (F_y) and axial (F_x) forces were 321kN and 487kN, respectively. Both were obtained 0.7sec after removal. Furthermore at the location above the removed element, on one side of the span only, the shear forces on the floor changed from downward 51kN to upward 181kN. With sufficient time for damping, at the equilibrium geometry of the structure, the shear converged to downward 50kN. With the new load-path, the floor loads was transferred to the external walls. The maximum compression load on those elements increased from 1242kN, before removal, to 3481kN after removal.

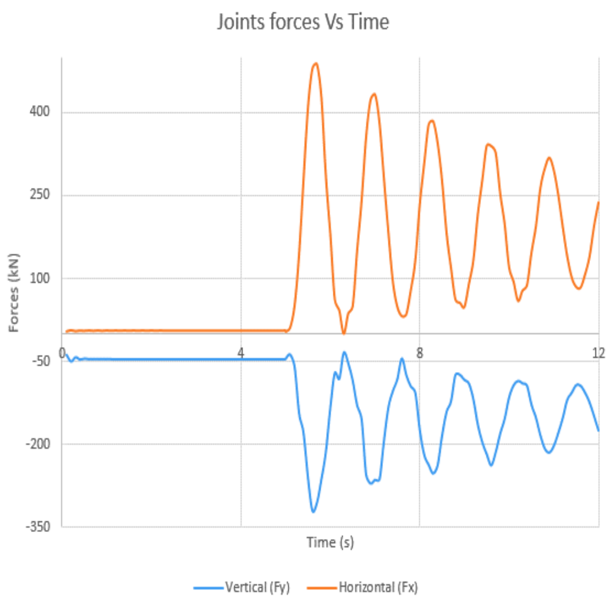

Figure 8*: Forces at joint level*

2.7 RESULTS MICRO MODELS

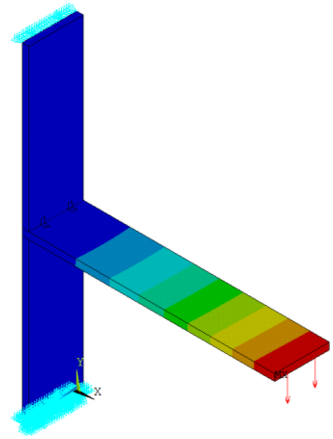

Figure 9*: Vertical deformed shape of 3D micro model 1*

The 3D model shown in Figure 9 had convergence problems; only 44% of the vertical dynamic loads from the global model could be applied. From this model, the rotational stiffness for k_1 and k_2 were estimated to 211kNm/rad and 359kNm/rad, respectively. The deformed shape of the model gave a maximum joint rotation of 1.14rad. The 2D-model was calibrated from the 3D-model in order to estimate the rotational stiffness demand. It was found that a rotational stiffness of 10^6kNm/rad, for both $k_{1,}$ and k_2, would be required in order to take the full dynamic loads. Nevertheless, as shown in Figure 10, the rotation at 0.9m from the wall-to-floor could not go beyond 1.14rad, taken as the maximum rotation. Hence the considered changes had no influences on the outcome.

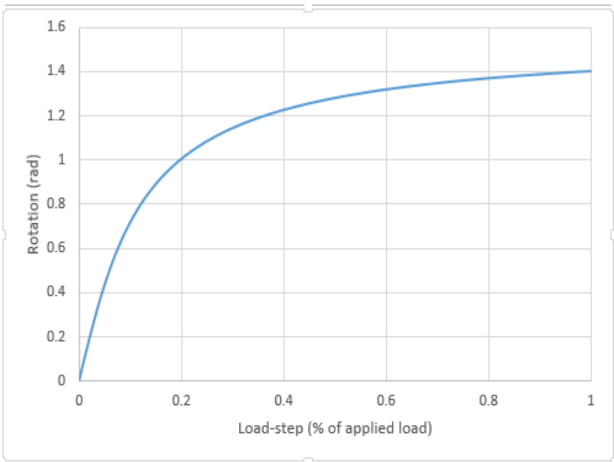

Figure 10*: Rotation vs applied load for 2D Micro-model*

The rotational stiffness (k_3) of the micro model 2 was estimated to 8kNm/rad from the 3D-model. Like for the previous joint, the analyses for this model was not able to converge with the applied dynamic loads, and increasing the value of k_3 did not improve the outcomes.

Figure 11 illustrates the deformed shape of the floor-to-floor joint under the applied loads. It captures the membrane action between the two components and shows that the connection was inadequate; highlighting possible failure before full application of the dynamic load. For both micro models, it was found that to stay within collapse thresholds the CLT elements' thickness needed to be increased to at least 300mm. It is only then that the rotational-supply could increase with an increase in the rotational stiffness values. A rotational stiffness of 10^5kNm/rad, for all joints, would be adequate to avoid disproportionate collapse, considering the updated floor thicknesses.

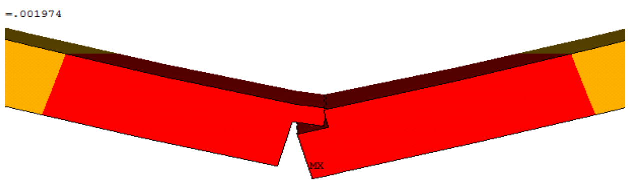

Figure 11*: Deformed Micro model 2*

2.8 MACRO MODEL

For the macro model, the walls and floors thicknesses were updated to 300 and 350mm, respectively. Here, all joints were assumed to have the same k-values to emphasise on the forces at the connection. Table 2 shows the tension and shear forces at wall-to-floor joint, for the given values of rotational stiffness. It was noticed that beyond 10^3kNm/rad, the k-values would have a little effect on the rotation. Furthermore, beyond 10^6kNm/rad, decrease in the tensile force at the joint would become insignificant. The results also showed k_1, and k_2 control the failure mechanisms or deformed shape at macro-level. It was found that if k_1 is bigger than k_2, the sub-frame would bend inward under the applied loads and therefore increase the maximum floor deflection.

Table 2: Macro model results

k-values (kNm/rad)	*Fx* (kN)	*Fy* (kN)	*Rotation* (rad)
10^3	2800	931	0.24
10^6	156	931	0.04

2.9 DISCUSSION

2.9.1 GLOBAL LEVEL

The scenario of slowly removing a loadbearing component, just as done for the TF2000, is identical to a vertical settlement of the building at the location of the removed element. The results of this scenario have shown that the building would not be prone to disproportionate collapse as the obtained maximum deflection of the floor was smaller than the set collapse thresholds. In addition, this highlights the need to design the floor to span twice its original length. Results from the dynamic analysis shows how the scenario changes when the wall is removed at a speed (t).

Table 3 shows that, from the dynamic analysis, the magnitude of the forces at the wall-to-floor joints could be subjected to an increase of more than 200% depending on the speed of removal. This led to an increase of about 280% compression loads at the external walls. The obtained results shows that the resistance of the structural elements, both floors and walls, needed to be revised in order to avoid failure. With respect to deformations, the analysis showed that the building was prone to disproportionate collapse as the vertical deflection at mid-span was 0.91m. Also, physically, it would be impossible to keep a safe structure with such a deformation.

Table 3: Forces at wall-to-floor joint

Forces	Static (kN)	Dynamic (kN)	Increase %
Axial (F_x)	181	487	270
Shear (F_y)	144	321	223

Results of the modal analysis have shown that the structure was prone to vibration. It is for this reason that 5% damping ratio was not sufficient for the structure to regain its static equilibrium geometry for the given time. This highlights resilience problems, hence safety questions, that most multi-storey timber structures would have in event of extreme loadings. Keeping in mind that the magnitude and deformation-demand would depend on the speed of removal, the joints would have to supply high ductility in order to dissipate the imposed energy. Furthermore, the dynamic analysis proves the necessity of designing the floors panels and connections to take reversal loads. These are upwards forces, estimated to be as high as 350% of the original downward forces at the joint, at the normal states, depending on the speed of removal.

2.9.2 MICRO LEVEL

In EN1995-1 [18], for joints made with dowel-type of fasteners, the axial stiffness of the connection under the service loads, also described as the slip modulus (K_{ser}), can be calculated as in equation 2.

$$K_{ser} = \sum_{i=1}^{s} \sum_{j=1}^{n} \rho_m^{1.5} \times \left(\frac{d}{23}\right) \tag{2}$$

Where (ρ_m) is the mean density of timber, (d) is the screw diameter and (s) and (n) are the number of shear plane and screws, respectively. Considering the connection between floor and wall above, provided by off-the-shelf brackets and self-tapping screws, K_{ser} per fasteners was estimated to 7kN/mm. With the rotational stiffness updated to 10^5kNm/rad and 500mm distance from a fastener to the centre of the group of fastener, the required number of screws was calculated to be 81. These calculations show that it would not be practical to achieve the required rotational stiffness. To be within reasonable and economic solutions, which are when the k-values need to be between 1kNm/rad and 10^4kNm/rad.

2.9.3 MACRO MODEL

The macro model showed the importance of continuity in the floor elements. A higher value of k_3 reduces the floor deflection and the required tensile forces at the wall-to-floor connections. With the initial structural detailing, which provides a rotational stiffness of around 10^3kNm/rad for all joints, the required tension at the wall-to-floor joint was estimated to be twenty times higher than the scenario with fully fixed joints. Even if the entire floor was assumed to be continuous (e.g. k_3 tends to infinity), the tension forces at wall-to-floor connections would still be seven times higher, approximately. This highlights design issues as common CLT connections are often designed for shear resistance. As a consequence, new structural detailing has to be proposed in order to prevent disproportionate collapse.

Furthermore, to develop the necessary catenary, at reasonable forces, the floor element must undergo large deformations (both in terms for central deflection at mid-span and rotational stiffness at connection) which might cause high internal stresses within the CLT panels. In such situations, even though the adequate connection detailing has been provided, brittle failure on the timber element could still happen.

3 CONCLUSIONS

The focus of this study has been devoted to numerical modelling approaches to investigate the possibility of disproportionate collapse on a twelve-storey CLT building subjected to the sudden removal of the internal ground floor loadbearing wall. The evaluation required nonlinear dynamic analyses. This study considered a multi-level investigation, done at three different sub-structural idealisations according to modelling details, feasibility and complexities. The Global-model accounted for the overall behaviour of the whole structure when subjected to dynamic loadings triggered by the speed of removal of the structural element. The resulting deformations were checked against the results from Micro- and Macro-models where the former accounted for the performance at connection level and the latter considered the behaviour at sub-frame level. In this study, focus was given on the forces and rotational stiffness at the joints, necessary to develop resistance mechanisms against disproportionate collapse. The following were the main findings:

- The investigations at global level showed that a static removal of loadbearing element is not sufficient for design against disproportionate collapse. Dynamic analysis, which considers a sudden removal of the structural components at speed (t), would be required as this could lead to an increase of about 200% of force and deformation-demands from the static case.
- The sudden removal of vertical key elements results in high downward and reversal loads requiring attention in joint detailing, in terms of number of screws, embedment length and, sizes of angle brackets; as well as the selection of structural members thickness.
- Based on the presented structural idealisations, including the chosen floor plan, this study indicates that normal off-the-shelf brackets and screws would be insufficient to supply enough rotational ductility and tension resistance to develop catenary action. Therefore this detailing cannot be used for design against disproportionate collapse of multi-storey CLT buildings.
- The study also confirmed that sizing of structural elements plays an important role in keeping the final failure within set collapse thresholds; serviceability and ultimate limit states designs and checks would not be sufficient.
- There is a need for further analysis to check the stresses within the CLT panels, necessary to develop catenary action. Even though adequate connection detailing is provided, it is still essential to check whether the CLT panels themselves would be able to physically accommodate the anticipated deformations.
- This study also recommends, for better estimations on the probability of failure, a reliability analysis of the proposed buildings in order to account for uncertainties on the magnitude of extreme loadings, element and connection stiffness.

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