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RELIABILITY ANALYSIS AND DISPROPORTIONATE COLLAPSE FOR MULTI-STOREY CROSS-LAMINATED TIMBER BUILDING

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Abstract

This paper focuses on developing finite element models and subsequent sensitivity and reliability analyses to evaluate the probability of disproportionate collapse of a twelve-storey case study Cross-Laminated Timber (CLT) building, following extreme loading events such as earthquakes. As an initial damage, the methodology mimics event-independent scenarios that include a sudden removal of a vertical loadbearing element. This investigation emphasises on the rotational stiffness (*k*) of the joints by considering nonlinear dynamic analyses of the structure at global, component and connection levels. The rotational stiffness-demand required to prevent disproportionate collapse is checked against the supply at all levels of structural idealisation. A sensitivity analysis at the sub-frame level shows that an improved structural detailing, to carry tie forces between structural members, as well as adequate thickness of structural elements, are important to prevent disproportionate collapse. The reliability analysis of the optimised structural model proves that the required *k*-value, to develop resistance mechanisms, is too large to be practically achieved by means of common CLT joint detailing, which is provided by angle brackets and screws. As a result, connection detailing and size of members of the case study building need to be reevaluated in order to avoid disproportionate collapse in the twelve-storey CLT building.

Keywords: Progressive collapse; sudden removal; redundancy; Robustness; structural integrity

1. Introduction

1.1 Disproportionate collapse

Collapse occurs when a building component can no longer fulfill its structural functions. In other words, the structural element loses its design capacities and cannot meet its principal purposes [1]. The National Building Code of Canada (NBCC) [2] uses the concept of importance factor when determining earthquake loads to reduce the probability of structural failure for a building. Collapse of a residential house does not have the same social and economic impacts when compared to the failure of a hospital, as the latter is essential to the provision of services in the event of a disaster. The same concept is also found in Europe where the EN1991 (Part 1-7 section A.1) [3] categorisations are with respect to building height and occupancy. With an increase in height, occupancy level and intended use of the building, safety concerns become more stringent.

The importance of these classifications, in the context of disproportionate collapse, was first required after the failure of the Ronan point flats in London, in 1968, where an explosion caused direct failure of a vertical loadbearing element, which in turn triggered the collapse of a major part of the building [4]. This incident highlights the danger of tall structures in situations when building residents do not have enough time for escape. Numerous failures of this kind have been recorded around the world; the one receiving most attention of this nature happened in New York City, in 2001. The World Trade Centre incident was characterised by a chain of failures, and ultimately a catastrophic collapse, even though the immediate damage was the loss of loadbearing elements located well above the ninetieth floor [5]. These events led to scepticism regarding the design and construction of tall buildings [6]. From there, attention was shifted to resilience after extreme loadings scenarios, defined as loads that occur infrequently but with a magnitude big enough to cause significant structural damages. In Part 4 of the NBCC, earthquake load is an example in this category [7].

The Commentary B of the NBCC advises the designer to consider severe accidents with a probability of occurrence estimated to 10^{-4} per year [7]. At the maximum considered earthquake (MCE) level, with 2% in 50-year occurrence, buildings and structural elements are designed for a failure probability of 10%. Therefore, it is acceptable when a component or joint loses its structural function following an earthquake. However, concerns arise when this ineffective element triggers the impairment of connected components and ultimately, a large part of the structure. Whether it is in a progressive manner or not, this type of collapse is classified as disproportionate if the final damage goes beyond acceptable thresholds. In this paper, these thresholds are also referred to as collapse tolerances, which are more or less judgements made after observing the final damage [8]. In EN1991, disproportionate collapse is defined when the initial failure affects more than 15% of the floor area of the damaged storey or $100 m^2$, whichever is less, and extends further than the immediate adjacent levels [3]. Identical thresholds were also found in NBCC 1977, Commentary C [9] but since then, they are no longer included in the NBCC.

The probability of disproportionate collapse, *P(F)*, in the event of extreme loadings can be estimated using Eq. (1). This depends on the probability of failure given that an extreme load occurred, *P(F|E)*, and the probability of occurrence of the load in question, *P(E)* [10]. The latter has a return period of 10^{-4} per year and the EN1991 [3] describes it as a malicious action of unspecified sources to account for a broader scope. This is an aleatory uncertainty, and as a consequence, reducing *P(E)* can be considered as an onerous, if not impossible, task for the designer. The NBCC, for example, has made it entirely up to the designer's decisions and engineering judgments. Therefore, the most realistic and only option is to reduce *P(F|E)*. Here initial failure, following *P(E)*, is accepted. However, the structure has to be designed to have a low probability for subsequent damages; with the final failure well within disproportionate collapse tolerances. This is a threat-independent method as there is no need to consider the extreme event itself nor its specific magnitude. EN1991 [3] and the U.S General Service Administration (GSA) in the United States [11] recommends the removal of structural loadbearing elements to check whether the resulting collapse is still within the set thresholds.

$$P(F) = P(F|E) \cdot P(E) \quad (1)$$

1.2 Robustness

The first stage towards reducing *P(F|E)*, as a design against disproportionate collapse, is to account for structural robustness [12]. Robustness is the ability of the structure to develop alternative load-paths and hence new equilibrium states, in situations when structural components become ineffective. This is done by triggering resistance mechanisms in order to bridge over the damaged element and consequently prevent it from spreading beyond the acceptable tolerances. To redistribute the loads to the undamaged parts, a building must have adequate continuity, ductility and stiffness at connection, component and global levels. For the structure to behave as a whole, the designer must account for the required structural integrity and redundancy; these are key ingredients in robustness strategies [13]. Both are influenced by the connection between different structural elements. Joints play an essential role in providing not only strength but also continuity, ductility and stiffness. Continuity is essential for force redistribution; ductility and stiffness become important when the structure is required to sustain large deformations, while maintaining loadbearing capabilities, without rupture at any level. When all three factors are satisfied, the structure is able to develop resistance mechanisms such as catenary or suspension action.

In event of loss of an interior vertical loadbearing element, provision of adequate longitudinal continuity between horizontal and vertical loadbearing elements is necessary in order to resist disproportionate collapse. The floor develops a suspension mechanism also known as catenary action; this prevents debris loadings on the floor below. When an internal structural element becomes ineffective, to resist the applied loads (W), the floor becomes a double-span ($2 \cdot L$) system only if sufficient tension forces (T) are provided at connection level. These are the wall-to-floor and floor-to-floor joints. Here the longitudinal continuity, supplied by the tie forces (T), can be calculated as shown in Eq. (2). In addition, the joints shall be able to carry the imposed floor rotation (Θ) in order to accommodate the required mid-span vertical deformation (δ). In this case, there shall be compatibility between the (T) and (Θ); which can be ensured by the provision of adequate embedment, cross-sectional area, spacing of ties.

$$T \cdot \delta = W \cdot (L^2 /2) \tag{2}$$

1.3 Design Standards

The requirements for structural robustness are embodied within design standards, in the form of guidance, to allow for adequate design against disproportionate collapse. In Europe, EN1991 [3] prescriptions are given with respect to notional tie forces. The guidance prescribes the minimum tension required at connection level for both frame and loadbearing wall systems, regardless of the material, to develop resistance mechanisms. This is an indirect approach because, if satisfied, for buildings less than four storeys for example, no further analyses are needed in order to design against disproportionate collapse. In cases when the notional tie forces are not sufficient, for buildings taller than four storeys for example, a direct approach is proposed. In this case, the removal of vertical elements, one at the time at every level, is advised to check whether the subsequent collapse goes beyond the set tolerances. In Canada, after 1980, the NBCC gives no direct reference to disproportionate collapse. The code only requires to account for sufficient capacity and structural integrity. Furthermore, the NBCC assumes that structures designed in accordance with the CSA material design standards have satisfactory degree of structural integrity to localise the damage and avoid widespread collapse. For concrete structures, the overall behaviour against disproportionate collapse is well understood and there is enough literature on analysis methods and design strategies to account for both direct and indirect approaches. GSA [11] and the Unified Facilities Criteria (UFC) [14] provide step-by-step worked examples as well as design strategies to meet the required tie forces using rebar detailing and curtailment [15].

While CLT, is described as a viable alternative for steel and concrete for the future of the world's skyline [16], little guidance on design of CLT structures against disproportionate collapse is available. Efficient and effective approaches to ensure structural integrity are yet to be implemented, tested and approved. In the United Kingdom, a six-storey timber frame experimental building was used to assess its performance in terms of disproportionate collapse. The project, known as Timber Frame 2000 (TF2000) [17], led to provisions based on tie force requirements for multi-storey timber buildings in general. However, these provisions are limited to six storeys and it is unclear whether extrapolation is possible to other structural systems of different heights and proportions [8]. In Canada, since structural integrity is provided using the CSA Standards, no information is available for the design of multi-storey CLT buildings for structural integrity. Design against disproportionate collapse for multi-storey CLT buildings is left solely to sound engineering judgement as the use of provisions such as TF2000 leads to unrealistic and expensive solutions. As a consequence, there is a need to implement practical guidance for adequate design methods and appropriate mitigation strategies against disproportionate collapse for this type of buildings.

CLT panels can be used as floor and wall elements for multi-storey timber building concepts. This system, either platform or balloon construction, can only be as strong as the connections between individual panels. Since interruption is caused by the presence of connection, structural failure often occurs due to inadequate design or improperly fabricated joints [18]. In this paper, an assumption is made that sufficient continuity is provided in the system, using the tie force (T) requirements and therefore, emphasis is on the rotational capabilities (Θ) of the connections in order to develop catenary action and subsequently design for robustness. The ability to achieve the required degree of rotation (Θ) is influenced by the rotational stiffness (k) of the joints. The hypothesis of this paper is that, to trigger resistance mechanisms against disproportionate collapse, the joints between structural elements must develop sufficient rotation at connection level, expressed in terms of (k), as compared to the deformation-demand, from the global level, after the loss of a structural element.

2. Numerical study of a twelve-storey CLT building

2.1 Overview of case study

The study considered a twelve-storey CLT building with 9m x 9m floor plan dimensions. This building was part of a TRADA example [19], designed to serviceability and ultimate limit states, all in accordance with EN1995 [20]. This was a platform construction with a 3m storey height, all walls loadbearing, and floors spanning 4.5m in one direction across an internal wall, as illustrate in Fig 1 . The external wall and floor panels were 125mm thick whereas the internal wall panels were 135mm; all were assumed to be solid with no openings. The structural design has proven that stability could be achieved using common joint detailing and no hold down straps would be required, although fixing to sliding would still be necessary. However, further checks for disproportionate collapse were still required [19].

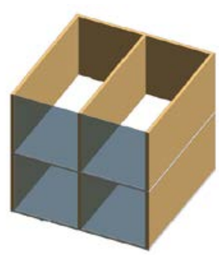


Fig 1: Case-study building (isometric view)

The aim of the study was to check whether the building would maintain its structural integrity after a vertical loadbearing wall was made ineffective, following high earthquake loadings. EN1991 and GSA [3, 11] gives locations of critical elements of the building to be removed. To narrow down the number of analyses, this study assumed that plastic hinges and accumulated damages during and after earthquake were concentrated on the ground floor. The magnitudes of forces and moments, from static analysis, were high on the ground floor external and internal walls.

In order to obtain the worst case scenario, defined as the element failure that would trigger a disproportionate collapse, a pushover analysis was required. The building was pushed in the direction of the three walls; a triangular displacement load was applied at every storey for this analysis. Although it was assumed that all three walls would be part of the lateral resisting system of the building, the internal wall was found to be the most critical due to its high stiffness as compared to the external ones. Fig 2 shows the magnitude of the base shear recorded for both walls, external and internal, with respect to the drift.

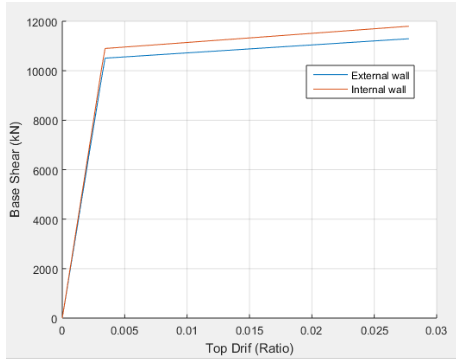


Fig 2: Pushover results

The pushover analysis confirmed that if a ground motion was applied on the building, the internal ground floor wall would yield first. Therefore, this would be considered as initial damage following the earthquake. Furthermore, this study followed the BS8110 [21] and BS5950 [22] in terms of load combinations and the EN1991 [3] for the alternate load-path design. The analysis was event-independent; where the critical element was removed instantaneously to trigger a dynamic response depending on the speed of removal. The resulting forces and deformations from this analysis were considered as demands on the structure. The study assumed that adequate continuity was provided and therefore focus was on the rotational stiffness (*k*) of the wall-to-floor and floor-to-floor joints, shown in Fig 3, to achieve the required structural robustness by developing catenary action. The structural details illustrated in Fig 3 are identical to the one used for the Stadthaus apartment, an eight-storey CLT building with platform construction located in London [23]. The connections consist of off-the-shelf brackets and self-taping screws. Since the joints would control the strength and stiffness of the building, the obtained *k*-values were considered as supply at connection level.

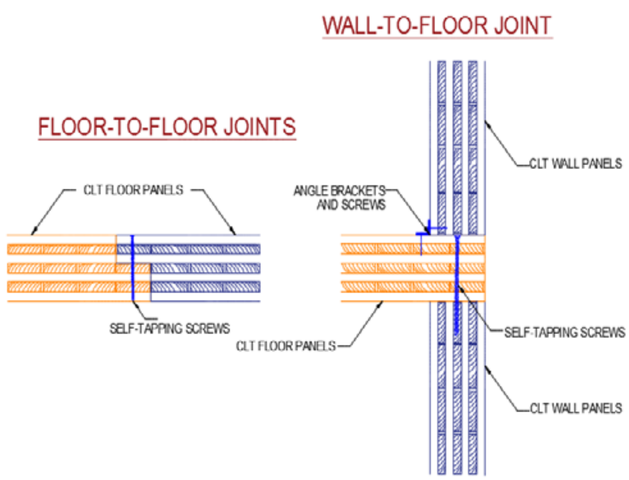


Fig 3: Common structural detailing for multi-storey CLT buildings

At sub-frame level, the model was built using the rotational stiffness values obtained from the analysis at connection level and the applied forces were from the dynamic analysis at global level. This model only accounted for the floor above the removed element, the ground floor and first floor external walls. It is at this level that comparison between demand and supply was done; the analysis was static. If the model could take the applied loads with the supplied *k*-values, then the building would be able to avoid disproportionate collapse after the loss of the internal loadbearing wall. Otherwise, the provided detailing needed to be improved, by modifying the *k*-values, until the subsequent collapse became acceptable. For this study, the thresholds were defined as 10% of the floor span; representing 900mm deformation at mid-span [15]. Although this deflection might not be realistic in timber structure, this would be the suitable gap to ensure that catenary action is effective in preventing debris from bearing on the level below [24].

2.2 Model development

There was a need to develop appropriate digital models for the investigation of the structural behaviour of the proposed twelve-storey building. This evaluation considered a sudden loss of the ground floor internal wall. Analyses incorporated the dynamic effects of the key element failure over a short duration, when compared to the response of the structure. In addition, nonlinearity was accounted in modelling the material properties for screws and angle bracket elements, as well as large deformations analysis. This was a nonlinear dynamic investigation as recommended by GSA [11] and UFC [14]. Finite element software, Ansys 12, was used for this investigation [25]. The study used a multi-level approach where analyses were done at three different structural idealisations according to the feasibility of the model reduction.

The Global model mimics the full building to understand the overall structural behaviour of the full twelve-storey CLT building following the loss of the internal ground floor wall at a speed (*t*). Here, focus was on load combinations, deflected shape, vibration frequencies and damping ratio. It is at this level that the force and deformation-demands were obtained. Since these demands defined the upper-bound of the scenario, the joints between structural elements were all assumed to be fully fixed, and the CLT material properties were idealised as linear orthotropic [18]. The permanent loads were estimated to 1.28kPa, 1.26kPa, 1.37kPa for external, internal walls and floor, respectively [19]. The imposed loads, for residential building, was given as 2.0kPa [26]. The dynamic analysis assumed a critical Rayleigh damping ratio of 5%, for timber buildings [18].

The second idealisation, the Micro model accounted for the behaviour at connection level. Here detailed models of joints, as shown in Fig 3, were built to capture appropriate behavior, as well as the contributions of elements present within the connection. This level considered nonlinear material properties, spacing and physical dimensions for connectors, and contributions of different layers of CLT panels in both directions. The Micro model provides the force and deformation-supply in terms of rotational stiffness (*k*). The CLT panels were 3-ply with the same material properties as the Global model. To avoid penetration between different elements, a friction of 0.3 was used. The angle brackets and screws were steel with strength of 240MPa [27] and 1,000MPa [28], respectively. The self-tapping screws were 300mm long with 8mm diameter. The angle brackets, fastened with regular timber screws, followed the minimum patterns of detailing provided in the specific products European Technical Approval [27]. A spacing of 500mm centre to centre was used for the installation of angle brackets, connecting the floor to the wall above, and self-tapping screws, inserted at 90 degrees, connecting the floor to the wall underneath.

The last idealisation, Macro model, accounted for the behaviour at sub-frame level. This was a simple 2D-model built from the *k*-values, obtained from the Micro model analysis, assigned to rotational spring that connect beam elements. This level mimics the force interaction between floor panels and possibilities to develop catenary action. It is worth mentioning that both second and third idealisation were modelled per metre width of the slab.

2.3 Results and discussion

The dynamic analysis at the global level showed that the magnitude of forces at the connection was subjected to an increase of about 200% when comparison was done against the static analysis of the building without the loadbearing wall. As shown in Fig 4, when the wall was removed in 0.001second (sec), the axial forces at the wall-to-floor joint increased from 181kN in the static to 487kN in the dynamic simulation, with a Rayleigh damping ratio of 3%. In the same context, the shear force values changed from 144kN to 321kN. With the new load-path, the total loads on the ground floor external walls increased from 1242kN, before removal, to 3481kN after removal. It was concluded that 5% damping ratio offered by timber structures would not be sufficient to provide acceptable resilience following extreme loading events as the structure needed more than 15 seconds to regain its static equilibrium.

The presented results were mainly influenced by the speed of removal (*t*); the shorter the time, the higher the resulting forces. Fig 4 shows how the rotation, at 0.9m away from the wall-to-floor joint, changed with respect to time. The graphs make a comparison between 0.001sec and 1sec *t*-values. It was observed that the joints would be subjected to high cyclic motions and therefore the design should account for high ductility expressed in terms of energy absorption.

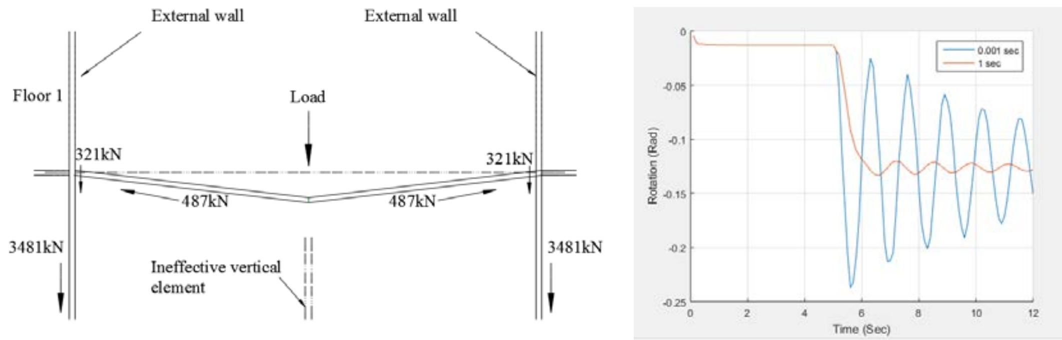


Fig 4: Results from Global model: Forces (left) and Rotation Vs Time (Right)

For 0.001sec, the resulting maximum vertical deformation, at the location of the removed element, of 0.91m. The obtained results have shown that, even with the upper bounds, the building is prone to disproportionate collapse after the removal of the internal ground floor wall. Therefore, regardless of the *k*-values, improvements needed to be considered in terms of CLT material properties and physical dimensions, such as material grade and panel thickness. Furthermore, the analysis showed that floor elements and all connections need to be redesigned to account for reversal or upward forces induced by the dynamic motion. At connection level, depending on (*t*), this could be as high as 350% of the original downward forces at static analysis, above the removed element.

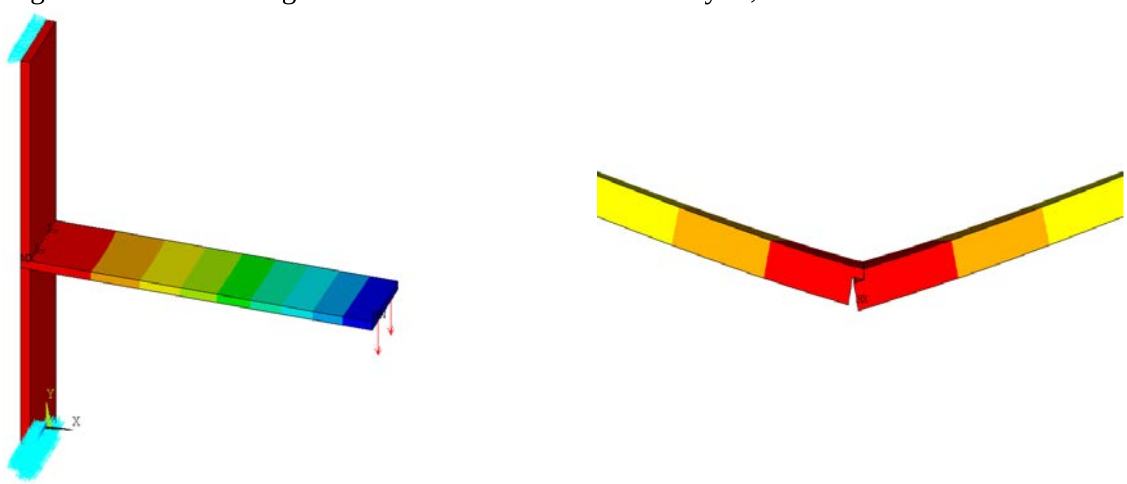


Fig 5: Deformed shape of Micro model: wall-to-floor (Left) and floor-to-floor (Right)

Fig 5 shows the deformed shape of Micro models, for wall-to-floor as well as floor-to-floor joints. At connection level, results of a static analysis showed that the joint could only carry 44% of the applied vertical dynamic loads from the Global model; giving a maximum rotation of 1.14rad. The *k*-values of the provided detailing were estimated to 500kNm/rad and 8kNm/rad for wall-to-floor and floor-to-floor joint, respectively. At this level, the outcomes confirmed that the strength and rotation-demands were higher than the supply, hence the building was prone to disproportionate collapse. To improve the results, using trial and error approach, it was found that the floor and wall thicknesses should be at least 300mm to keep the *k*-values practical and economic; which happens when the rotational stiffness (*k*) is less than 10^4kNm/rad. At sub-frame level, it was found that the system would not be able to develop catenary action with the provided structural detailing. The Global model has fully restrained connections; when modified using the k-values, this magnitude of the axial load was observed to be twenty times higher. This illustrated how the stiffness of the connection influenced the load distribution between the members. This brought concerns as common CLT joints are primarily designed for shear. Following these analyses, a better understanding of the chance of disproportionate collapse in this building was required. The Macro model was used to perform a sensitivity analysis, in order to understand how the thickness and *k*-values influences the response of the structure, and a reliability analysis, in order to quantify the probability of failure of the building following the sudden loss of the ground floor internal loadbearing wall.

3. Reliability analysis

3.1 Random variable description

The reliability analysis allows one to compute the probability of failure of the building in the presence of uncertainties either epistemic or aleatory [29]. For this case, parameters influencing the overall behaviour of the structures, at sub-frame level, should be considered as random variables as they could change depending on the applied loads, obtained from the dynamic analysis at Global level. The probability of failure would be affected by the speed of removal, which would be considered as an aleatory uncertainty since this is an event-independent scenario.

The first considered random variable was the *k*-value since a different structural detailing could have been used instead of the proposed one. Nonetheless, to remain within this paper's recommended practical range, the rotational stiffness was taken as a deterministic variable with continuous values estimated between 1kNm/rad and 10^3kNm/rad for both wall-to-floor (k_1) and floor-to-floor (k_2) connections. The second random variable was the thickness of floor and wall panels expressed in terms of number of ply (Num_{ply}). For this, discrete values were chosen to represent 3-ply, 5-ply, 7-ply and 9-ply CLT panels; these were 102, 170, 238 and 306mm thick, respectively. These values were chosen not only to simplify the Ansys scripts by having 34mm as single layer thickness (d) for all panels, but also to keep the overall depth within the range of CLT manufactured in Canada [30]. Since timber is a natural material with uncertain material properties, its young modulus (E), bending and rolling shear strengths would depend on the panel's stress grades given in the CSA-O86 [31]; these are E_1, E_2, E_3, V_1 *and* V_2. The Ansys scripts accounted for these parameters in Num_{ply}. For this study, it was assumed that all modification factors (k_i) are 1.0 and effective bending stiffness (EI_{eff}) was calculated as shown in Eq. (3). Here (n) is the number of layer in the panel, and (z) is the distance between the centre point of the i^{th} layer and the neutral axis of the panel. The results of the EI_{eff} would then be used to calculate the bending moment (M_r) and shear (V_r) resistance of the panels according to CSA-O86 [30]. The wall thickness (T_{wall}) was a separate random variable since it was assumed that EI_{eff} would not affect the vertical elements under the considered loadings.

Furthermore, as the dynamic loads at Global level was governed by the speed of removal (t), the magnitude of applied forces (*Force*) used for this analysis was considered as the last random variable. Here, ten different analyses were performed at Global level with (t) varying from 10^{-5} to 10^{-2} sec. Being an uncertainty, this variable was given a lognormal distribution with a mean of 295kN and 100% coefficient of variation. The optimum model for the analysis would try to minimise the thickness of the CLT panels, by controlling the overall volume (V) of the model, to have a light but efficient structure, and the tension force (F_x) at wall-to-floor joints, since common CLT joint detailing are primarily designed for shear.

$$EI_{eff} = \sum_{i=1}^{n} E_i \times b_i \times \frac{d_i^3}{12} + \sum_{i=1}^{n} E_i \times b_y \times d_i \times z_i^2 \tag{3}$$

This paper considered the bending moment (M_r) and shear forces (V_r) on the floor panels as well as the rotation (Θ) of the wall-to-floor joint as constraints or limit state functions, defining probability of disproportionate collapse. Resistance M_r and V_r values were measured against the applied maximum bending moment (M_z) and shear force (V_y) of the floor panel, obtained after analysis of the structure under the applied loads. The rotation limit was estimated to 0.20radians (rad) to match with the paper's collapse tolerances. OptiSlang, a software that provides a framework for robustness design optimisation and stochastic analysis [32], was used in this paper to evaluate the structural safety of the considered CLT Macro model.

3.2 Sensitivity analysis

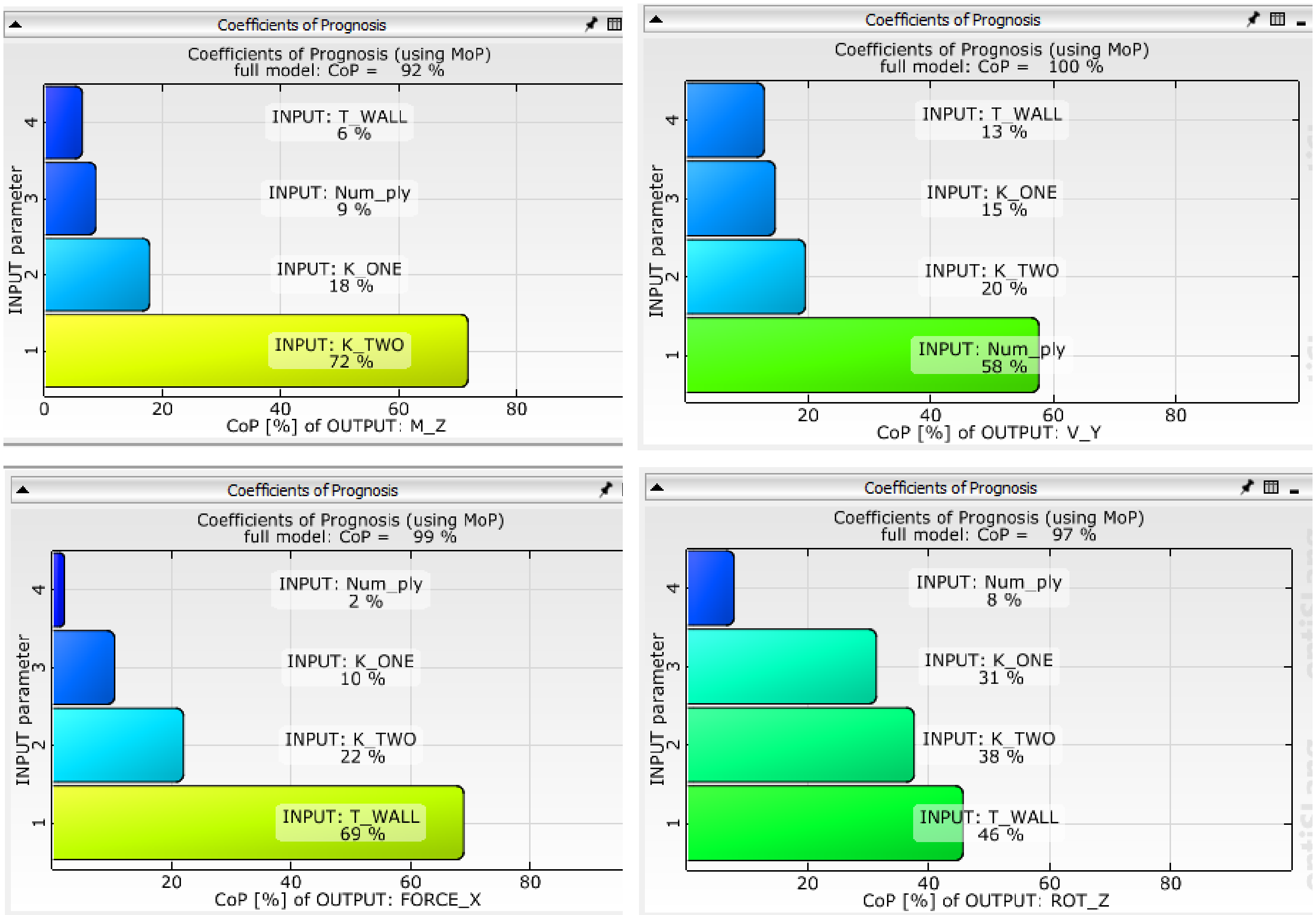


Fig 6: Sensitivity Analysis results: M_z (top left), V_y (top left), F_x (bottom right) and Rot_z (bottom left)

A global variance-based sensitivity analysis was performed using Advanced Latin Hypercube Sampling (ALHS) in order to scan the design space and estimate the sensitivity with the multivariate statistic based on surrogate models [32]. This was used to obtain the importance factors for all input random variables. At this level, it was assumed that applied force was a constant. This allowed for a better judgment on how much the CLT thickness and the connection rotational stiffness affect the structural response. The criteria was to maintain M_z and V_y below M_r and V_r, respectively, and have a rotation smaller than 0.20rad. In addition, the analysis had to keep F_x and V to a minimum. A total of 1000 samples were required to perform this analysis; the results are shown in Fig 6. Optislang uses the Metamodel of Optimal Prognosis (*MOP*) as an algorithm to obtain the results, expressed in terms of coefficient of prognosis (*CoP*). This was the accuracy of the approximation of the output variables with respect to the selected input random variables. From the results, the *CoP* for M_z, V_y, F_x and Rot_z were 92%, 100%,99% and 97%, respectively. A *CoP* of 95% or higher showed that the approximation of the sensitivity analysis was accurate and implication of considered input random variables are trustworthy.

The applied moment (M_z) was mainly influenced by the rotational stiffness (k_2) of the floor-to-floor joint since the latter dictates the moment redistribution between the two floor panels. For a bigger value of k_2 approaching infinity, a smaller M_z was obtained. The number of ply (Num_{ply}) was found to be the main parameter affecting the shear (V_y) as the resistance is directly proportional to the cross-sectional area of the panel. On the other hand, the thickness of the external walls (T_{wall}) affected both the tie forces (F_X) and the rotation at the joint (Rot_Z). This is mainly because the weight of the wall above acted as favourable loads, hence restricting the rotation of the floor panel by squashing it against the wall below. A lower rotation means smaller catenary action and hence, a lower axial force (F_X).

3.3 Optimisation

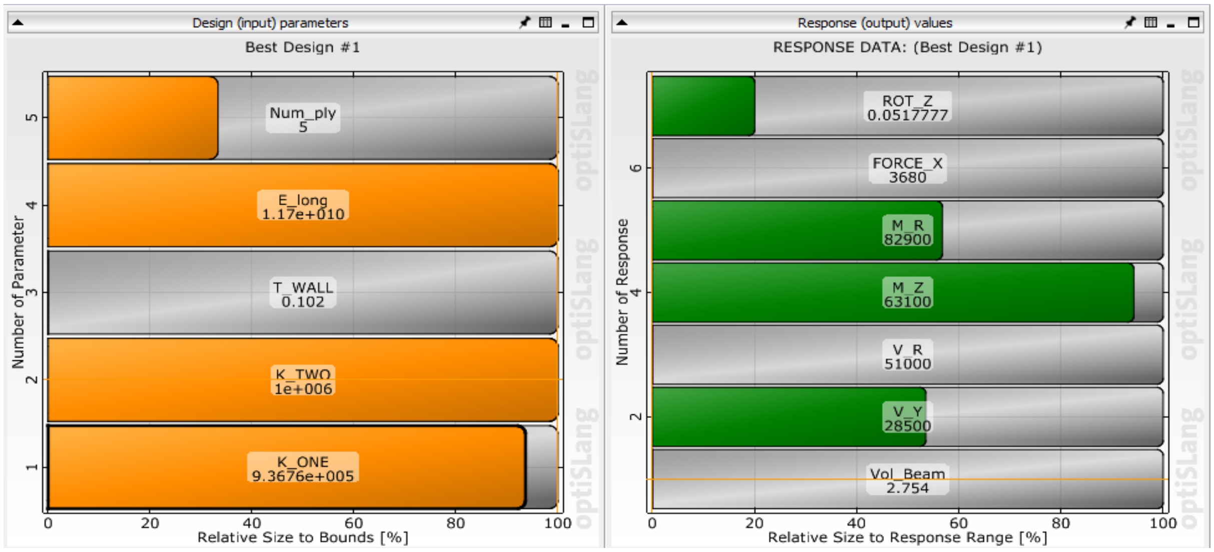


Fig 7: Optimised design: Input variables (left) and output response (right)

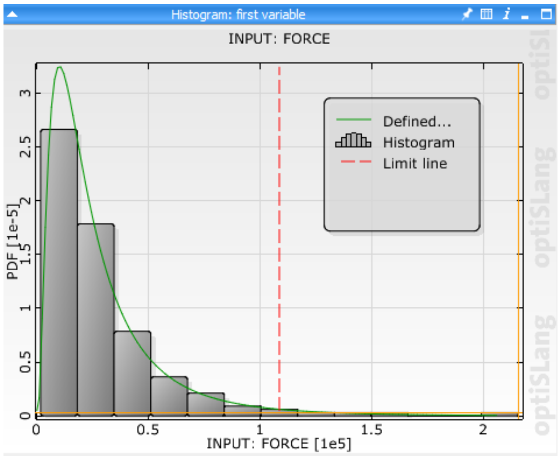
The coefficient of prognosis (*CoP*) obtained after sensitivity analysis was an important way to predict the influence of every single input variable with respect to the chosen output parameter. A new MOP was run to automatically find the optimum model. In other words, the best design that meets all the constraints (M_r, V_r and Rot_z) and objectives (minimum V and F_X). For the optimisation, analysis was performed using the Evolutionary Algorithm (EA-global) and 9900 was the proposed population size. Fig 7 shows the required input values to give the best possible or the optimum design of the structure. Here, 5-ply was found to be the ideal floor thickness, with a stress grade E_1 (associated with a young modulus of 11700MPa); and the thickness of the wall was at its minimum value of 102mm. The required rotational stiffness to obtain an optimum design were 0.937 and 1MPa for K_1 and K_2, respectively. Nevertheless, since a reliability analysis would be done afterwards, it is worth mentioning that the selection of the best design also depended on the feasibility of the structure. This was when the design targeted the smallest but realistic elements' thickness and axial forces that gave the smallest rotational stiffness in addition to keeping the moment and shear well below the resistances. This could also be considered for the Reliability Based Design Optimisation (RBDO) as here the objective was to minimise the total cost of the building (C_o), measured by its mass and affected by the volume, by varying the CLT thickness and joints rotational stiffness, affecting the probability of failure (P_f) times the cost of failure (C_f). This is illustrated in Eq. (4).

$$\mathrm{Min}\ (C_o + P_f \cdot C_f \mid \mathrm{Constraints}) \qquad (4)$$

If comparision has to be made against the original design, the floor thickness is the main revised parametre in order to keep the connections detailling practicable. The response of the optimised model shows that 76% and 55% of the 5-ply the bending and shear capabilities, respectively, would be used. The new model would have a rotation of 0.051rad. Furthermore, it is worth mentionning the magnitude of tie forces at wall-to-floor joint, required to obtain the estimated rotation, should have a minimum of 3.68kN per metre width of the slab.

3.4 Robustness and Reliability analysis

Before running a reliability analysis, a probability-based robustness evaluation was required. This helped quantifying the safety and reliability of the optimised structure in the presence of rare events uncertainties. In other words, how the uncertainties on the dynamic forces, which depends on (t), would influence the optimised model. This analysis calculated the probability of exceeding a certain response value, expressed in terms of sigma-level (σ). This evaluation required 202 samples. Fig 8 shows the probability density function (PDF) of the force having a log-normal distribution, as defined for the random variable *Force*. Setting a limit of 109kN to obtain the 95th percentile, a σ-level of about 2.8 was obtained. For a robust design in the presence of uncertainties with a lognormal distribution, the recommended σ-level is at least 4.0 in order to obtain a probability of failure of 10^{-3}or less [32]. The obtained σ-level showed that the optimised design would not be robust in presence of *force* uncertainties. This highlighted a high probability of failure. The σ-level deals with a single set limit value, while the probability of failure quantifies the event that any of the several limits is exceeded [32]. It is for this purpose that a reliability analysis was still required to prove the required safety level of the optimised building.



Statistic data			
Min:	2013	Max:	2.157e+005
Mean:	2.934e+004	Sigma:	2.811e+004
CV:	0.9581		
Skewness:	2.872	Kurtosis:	15
Defined PDF: Log-Normal			
Mean:	2.947e+004	Sigma:	2.947e+004
Limit x = 1.088e+005			
P_rel:	0.975124	1 - P_rel:	0.0248756
P_def:	0.976464	1 - P_def:	0.0235365
Sigma-Level:	2.82803		

Fig 8: Optimised design: Probability density (left) and Statistic data (right)

To compute the probability of failure of the optimised building, a First Order Reliability Method (FORM) was done in order to evaluate the reliability index. This was a series system reliability as failure occurred if any limit states functions, defined in terms of M_r, V_r and Rot_Z, were violated. From the FORM, a total of 84 designs was required to perform the analysis with the closest distance to the median, obtained from design point 82. A reliability index (β) of 0.16 was computed; giving a probability of failure of 0.44. The reliability analysis confirmed that the structure, even after optimisation, would still be vulnerable to disproportionate collapse when subjected to extreme load events. This highlighted safety concerns regarding the design and construction of multi-storey CLT structures.

3. Conclusion

This paper discusses the structural behaviour of the twelve-storey case study CLT building following a sudden removal of the ground floor internal loadbearing wall. The results confirm that design against disproportionate collapse is a major concern for multi-storey CLT buildings. Analyses from the three structural idealisations indicate that common off-the-shelf brackets and screws cannot supply the required strength and deformation-demands following the sudden removal of the internal ground floor wall of the twelve-storey CLT building. This investigation also confirms that the sizing of structural elements plays an important role in reducing the forces at connection level; hence a design for serviceability and ultimate limit states only is not sufficient. The sensitivity and optimisation analyses show that increasing the favourable loads by increasing the thickness of the wall is the ideal approach to reduce the force-demand on the connections. Furthermore, although it is possible to keep the rotational stiffness within practical and economic values, this study confirms angle brackets and screws are not strong enough to carry the imposed tie forces required to develop catenary action as a resistance mechanism against disproportionate collapse. This is a major concern as CLT connections are mainly designed to take shear loads. The low reliability index obtained from the reliability analysis shows that, in the presence of uncertainty with respect to the applied force, the proposed building is susceptible to disproportionate collapse. Considering the growing interest in the use of CLT in mid-and high-rise construction, some practical guidance should be provided to achieve structural robustness. These provisions should emphasise the tie force requirements to meet the demand for developing resistance mechanisms, hence avoid disproportionate collapse.

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