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NUMERICAL SIMULATIONS ON DISPROPORTIONATE COLLAPSE OF THE TALL GLULAM BUILDING UNDER FIRE CONDITIONS

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ABSTRACT

Perception of the public to structural fires is very important because there are only a number of tall timber buildings constructed in the world. People are hesitating to accept tall timber buildings, so it is essential to ensure the first generation of tall timber buildings to a very high standard, especially fire safety. Right now, there are no specific design standards or regulations for fire design of tall timber buildings in Europe. Even though heavy timber members have better fire resistance than steel components, many conditions still need to be verified before considering the use of timber materials, e.g. fire spread, post-fire collapse, etc. This research numerically explores the structural behaviours of a tall Glulam building when one of its internal Glulam (Glued laminated timber) columns fails after sustaining a full 120-min standard fire and is removed from the established finite element building model created in SAP2000. The numerical results demonstrate that the failure and removal of the selected internal Glulam column may lead to the local failure of the adjacent CLT (Cross laminated timber) floor slabs, but will not lead to large disproportionate damage and collapse of the whole building. Here, the building is assumed to be located in Glasgow, Scotland, UK.

Keywords: Tall timber building, Glulam, Fire, Eurocodes, FEM, Disproportionate collapse

1. INTRODUCTION

Timber elements used in civil engineering constructions are made from logs of exogenous trees. The general engineering timber used in daily life could be divided into softwood and hardwood according to its specifications and applications. Softwood is the wood from gymnosperm trees that have needles and exposed seeds without leaves and supply 80% of commercial timber globally, e.g. pine, Douglas fir and spruce (Wikipedia 2020). Timber construction is more environmentally friendly, and timber components could be assembled more quickly than other construction materials. Compared with traditional construction materials like steel and concrete, timber is a low-carbon, green, energy-saving renewable material. One cubic metre of harvested wood can store

0.9 tonnes of CO_2 and reduce 1.1 tonnes of CO_2 emissions when using wood as a construction material, saving a total of 2 tonnes of CO_2 emissions (Pei et al. 2011).

Timber members that are mechanically laminated were used in Europe in the early 19th century. Glulam is short for Glued laminated timber and is produced by jointing and laminating smaller pieces of timber with the grain of all the layers running parallel to the member length using durable, moisture-resistant structural adhesives. Small pieces of timber can be harvested from fast-growing trees with smaller log diameters (Ansell 2015). Nowadays, the modern construction industries can manufacture different lengths, depths and widths, and enable large single elements for different applications. The utilisation of Glulam has become an area associated with one of the construction material industries that have developed most rapidly and most successfully in the UK. The earliest Glulam structure in the UK was the hall of King Edward College, Southampton, built in 1860 (Lehringer and Gabriel 2014). Glulam was used as arches for the earliest railway bridges in England and Scotland between 1835 and 1855 (Riberholt 2007). Glulam had been produced and utilised particularly for only a few types of structures such as swimming pools, churches and pedestrian bridges before the 1970s, except for other professional purposes, e.g. aircraft and ship parts. As the upgraded and modern production plants with high capacities were built, they produced various types of standard Glulam elements, including straight, cone-shaped, bent or vaulted ones. As a result, the application scope of such components has been largely extended.

In 2010, Bergen og Omegn Boligbyggelag (BOB) planned to build a 14-storey timber frame building in Bergen, Norway, as shown in Figure 1. The design started in 2011, and the building was completed in 2015 at a total cost of 22 million Euros (Buildup 2020). The plan of this building was specially arranged, with the lift core to be independent and the Glulam frame to give the necessary stiffness to the building. The fifth and tenth storeys were set as the stiffened-strengthened storeys to increase the building's lateral stability. The maximum horizontal deflection of the building was determined as 71 mm, which is 1/740 of the building height. The Treet building was not intended to sustain seismic loading because seismic loading in that region would be very small and could be ignored. In this design, the timber elements in the load bearing system were thick enough to allow burning for 90 minutes without failing (Abrahamsen and Malo 2014). All steel connections consisting of 8 mm steel plates and 12 mm dowels were hidden inside the timber elements and would not fail within the required fire resistance time.

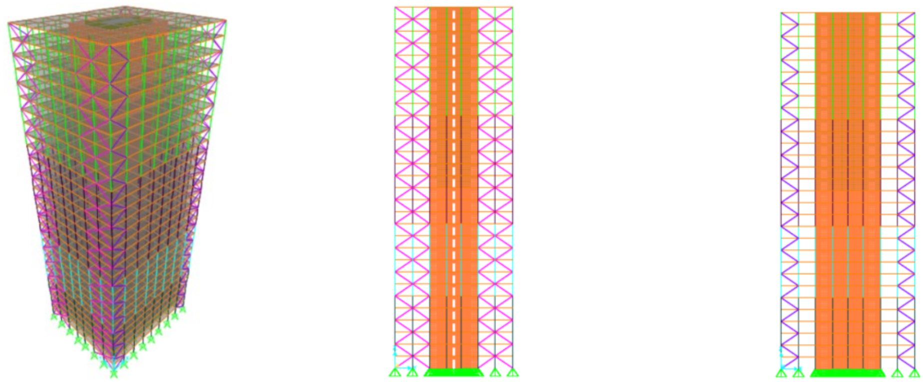
For a tall timber building under fire conditions, undesired disproportionate collapses may happen due to the failure of one or more of its key structural elements. This can be assessed on its robustness, i.e. its ability to withstand exceptional events like fire, explosions, impact or the consequences of human error, without being damaged to an extent disproportionate to the original cause. Accidental damage under fire conditions can be disastrous so it is essential for designers to make sure the designed tall timber buildings are robust enough. Although there are no specific design codes or guides for tall timer buildings under fire conditions, they could still be designed and constructed based on the current structural Eurocodes, together with the performance-based design approach, innovative construction technology and engineered timber products.

Figure 1: Treet in Bergen, Norway (Woodskyscrapers 2020)

This research aims to numerically investigate the structural behaviours of a 105 m high 30-storey tall Glulam building when one of its internal columns fails after sustaining a full 120-min standard fire and is removed from the established tall Glulam building model using commercial FEM software SAP2000 (CSI 2016a). For the building over 30 meters, the Building Regulations 2010 in England, UK (HM Government 2013) indicates that the fire resistance should be at least 120 minutes. Also, Building Standards Technical Handbook 2017: Domestic Buildings in Scotland, UK (Scottish Government 2017) indicates that for long fire resistance duration, the highest requirement is 120 min.

2. NUMERICAL BUILDING MODELS

In this study, the 30-storey tall Glulam building to be modelled is assumed to be located in Glasgow, Scotland, UK. The effects caused by the failure of an internal Glulam column in this building model after a 120-min standard fire are numerically simulated and analysed using the commercial finite element analysis software SAP2000. In this tall building model, the total building height is assumed to be 105 m, with a storey height of 3.5 m. The length of the building is 39 m, with the building width of 34 m. The CLT (Cross laminated timber) which is used for the lift core and floor slabs of the building is to be manufactured from C24 solid timber in accordance to BS EN 338 (BSI 2016). The Glulam which is used for constructing the frame structure of the building is manufactured from GL24c according to BS EN 14080 (BSI 2013). The effects of actual connections between structural timber elements are not considered yet and all CLT elements are assumed to be rigidly connected to Glulam beams, columns and bracings with the fixed boundary conditions. All Glulam beams and bracings are assumed to be pin-connected to the Glulam columns. The in-plane stiffnesses are applied to all the CLT wall elements with the out-of-plane stiffnesses applied to all the CLT slab elements. In this building, the infill panels are modelled as the linear loading on the simply supported beams, but the contributions of the infill panels to the stiffness are not considered for this building. As shown in Figure 2, it is clear to see those vertical elements are separated into different parts. Figure 2(a) gives the 3D view of the tall Glulam building, Figure 2(b) gives the side view in the vertical yz-plane, and Figure 2(c) gives the front view in the vertical xz-plane. The horizontal x-y plan of a typical CLT floor is illustrated in Figure 3, where x is the direction along the 39 m building length and y is the direction along the 34 m building width.



(a) 3D view **(b)** Side view (y-z plane) **(c)** Front view (x-z plane)

Figure 2: Different views of Model B for the tall Glulam building.

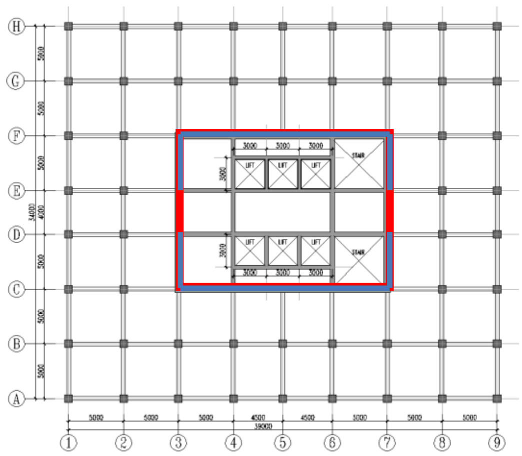


Figure 3: The floor plan of the tall Glulam building.

As shown in Figures 2 and 3, the internal core part mainly consists of CLT elements with the Glulam frame elements set to be around the core. In the recently completed tall buildings in the world, concrete shear walls are often used to provide the lateral stiffness and seismic resistance. To explore the possibility of high timber buildings, the CLT core is used instead of a concrete core. The red rectangular box indicates the 360 mm thick 9-layer uniform CLT walls for the inner core (Wall 1). The blue line on the red box indicates that there are no huge openings on the core wall. The grey lines within the core indicate the CLT walls for the building, with those 360 mm thick

9-layer walls for Storeys 1 to 15 (Wall 2) and those 280 mm thick 7-layer walls further above (Wall 3). All the floor slabs for this building are constructed from the 245 mm thick 7-layer CLT panels. Most wall elements have minimum openings of width × height = 1.0 m × 2.5 m for doors or windows. The details of door openings are listed in Table 1.

Table 1: Sizes of the wall and window openings.

Wall width	Sizes of wall and window openings (width × height)
5.0 m	2.0 m × 2.5 m
4.5 m	1.5 m × 2.5 m
4.0 m	1.0 m × 2.5 m

The dimensions of the Glulam members are listed in Table 2. The dimensions of the Glulam columns are 800 mm × 800 mm, 700 mm × 700 mm, 600 mm × 600 mm and 500 mm × 500 mm, respectively, pending the storey numbers. The dimensions of the Glulam beams are 350 mm × 500 mm. The dimensions of the bracing members in both y- and x-directions are 400 mm × 400 mm, and they are only located in the vertical plans along lines 1, 9, A and H, as shown Figure 3. Assume all the cross-sections of the Glulam elements in this building model are uniform.

Table 2: Sizes of Glulam elements.

Glulam elements	Size (mm)
Column 1 between Storeys 1 and 6	800 × 800
Column 2 between Storeys 7 and 12	700 × 700
Column 3 between Storeys 13 and 21	600 × 600
Column 4 between Storeys 22 and 30	500 × 500
Beams	350 × 500
Bracings	400 × 400

The class of Glulam elements is GL24c according to BS EN 10480 (BSI 2013). The bending elastic modulus parallel to the grain is 11000 N/mm^2, the bending elastic modulus perpendicular to the grain is 300 N/mm^2, the shear modulus parallel to the grain is 650 N/mm^2, and the shear modulus perpendicular to the grain is 65 N/mm^2. All the structural CLT panels are defined as orthotropic thin-shell elements based on the estimate method (Matsumoto et al. 2014; Yasumura et al. 2016). The in-plane stiffness is applied to all CLT elements. The detail of the equivalent stiffness parameters used in this research are listed in Table 3, where E_{rx} and E_{ry} are the equivalent bending elastic moduli about x- and y-axes for out-of-plane bending, E_x and E_y are the normal equivalent elastic moduli about x- and y-axes for in-plane bending, and G_{xy}, G_{yz} and G_{zx} are the equivalent shear moduli in the xy-, yz- and zx-planes.

Table 3: Equivalent stiffness parameters of the CLT elements.

Stiffness parameters	Wall 1	Walls 2 and 3	Floor slab
t	360.0 mm	280.0 mm	245.0 mm
E_{rx}	7441 N/mm^2	7933 N/mm^2	7929 N/mm^2
E_{ry}	3927 N/mm^2	3438 N/mm^2	3436 N/mm^2
E_x	5094 N/mm^2	4926 N/mm^2	4926 N/mm^2
E_y	6276 N/mm^2	6444 N/mm^2	6444 N/mm^2
G_{xy}	392.22 N/mm^2	402.77 N/mm^2	402.77 N/mm^2
G_{yz}	111.53 N/mm^2	107.54 N/mm^2	107.55 N/mm^2
G_{zx}	111.53 N/mm^2	107.54 N/mm^2	107.55 N/mm^2

Rigid diaphragms are assumed for all floor slabs. The wind load is defined as exposure from extents of rigid diaphragms in a wind direction of 270°. The wind velocity is 26.21 m/s, the terrain category is set as II, and the orography factor, turbulence factor and structural factor are all set as 1.0. The wind load is based on BS EN 1991-1-4 (BSI 2005a) and the corresponding UK NA (BSI 2005b). The dead load on the floor is 0.92 kN/m^2 excluding the self-weight of the structural elements, and the imposed load is 3.3 kN/m^2. The dead load from the plasterboards and the assembling and decorating parts on the beams is assumed to be 5.38 kN/m. All the loadings on the building are chosen or determined in accordance to BS EN 1991-1-1 (BSI 2002c) and the corresponding UK NA (BSI 2002d). The load combinations are in accordance to BS EN 1990 (BSI 2002a), the corresponding UK NA (BSI 2002b), BS EN 1995-1-2 (BSI 2004c) and the corresponding UK NA (BSI 2004d). As shown in Figure 3, there are some areas used as stairs within the core. To assemble these areas to the model, their thicknesses are adopted as 0 mm, with the assumed dead load of 4.0 kN/m^2 and the assumed imposed load of 3.0 kN/m^2. The units and grids of models, the adopted materials, sections, constraints, load patterns, load cases and load combinations for design purposes have all been defined when creating the building model according to the SAP2000 Manual (CSI 2016b).

3. DISPROPORTIONATE COLLAPSES

For tall timber buildings under fire conditions, undesired disproportionate collapses may happen due to the failure of one or more of their key structural elements. This section is to numerically explore the structural behaviours of the indicated tall Glulam building when one of its internal Glulam columns suddenly fails and then is removed from the established finite element building model after sustaining a full 120-min standard fire.

3.1 The reduced cross-section method

The reduced cross-section method is used for the structural analyses on the tall Glulam building under fire conditions incorporating with the reduced member thickness and the remaining equivalent mechanical properties. Using SAP2000, the residual axial and shear stresses for the fire resistance t_{fi} = 30, 60, 90 and 120 minutes can be accurately determined.

BS EN 1995-1-2 illustrates the reduced cross-section method where an effective cross-section is obtained by reducing the initial cross-section by the effective charring depth d_{ef} calculated from Equations (4.1) and (3.2) of BS EN 1995-1-2 as

$$d_{\mathrm{ef}} = d_{\mathrm{char,n}} + k_0\, d_0 \tag{1}$$

where $d_{char,n}$ is the notional design charring depth and $d_{char,n} = b_n t$, b_n is the notional design charring rate in mm/min, t is the charring time in min, k_0 is the charring depth factor, and t_0 is the zero strength and zero stiffness depth and is normally taken as 7 mm.

The reduced cross-section method could not be used automatic by SAP2000, and the new property of each element should be re-input. Because of the changes in the CLT element thicknesses, the new equivalent properties of different CLT members need to be recalculated based on the estimated method. The weights of timber elements are assumed not to change after fire, and the densities of relevant elements also need to be recalculated.

3.2 Disproportionate collapse analyses on the tall Glulam building

The selected internal column which is assumed to fail and be removed after sustaining a 120-min standard fire is Glulam Column B2 in blue dash line in Storey 1, as shown in Figure 4. It should be the worst situation for disproportionate collapse of this building. The fire is assumed to start in AC13 floor area, including four rooms BC12, AB12, BC23 and AB23. The fire conditions are the same as the four-side fire conditions for the middle supporting Glulam Column B2. It is assumed that the fire will char Glulam Beams BC2, AB2, B12 and B23, CLT floor Slabs BC12 (Slab 1), AB12 (Slab 2), BC23 (Slab 3) and AB23 (Slab 4) and Column B2 in Storey 1. The beams are assumed to be under three-side fire, and the CLT floor slabs are assumed to be under one-side fire and the indicated Glulam column is assumed to be under four-side fire. Figure 4 illustrates the seventeen selected node points on the CLT floor Slab 1 for further analyses. Because the CLT elements for Slabs 1, 2, 3 and 4 are symmetric about the two axes B12-B23 and AB2-BC2, a total of 17 node points on Slab 1 should be enough for exploring the typical stress developments on Slabs 1, 2, 3 and 4.

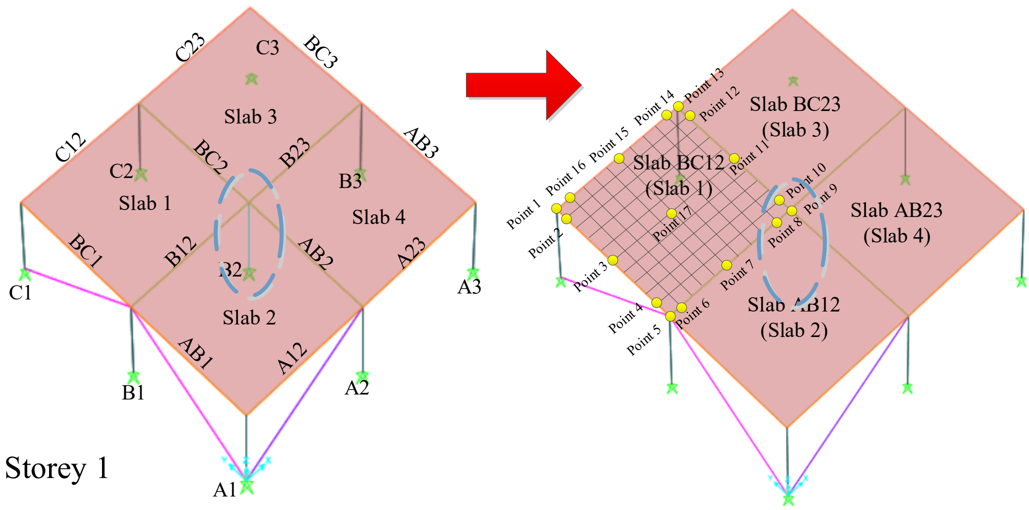


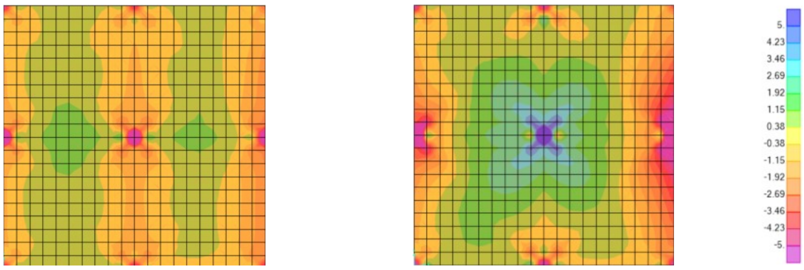
Figure 4: Removing the internal supporting Glulam Column B2 in Storey 1 in the tall Glulam building model after a 120-min standard fire

3.2.1 Stress developments in the relevant CLT slabs

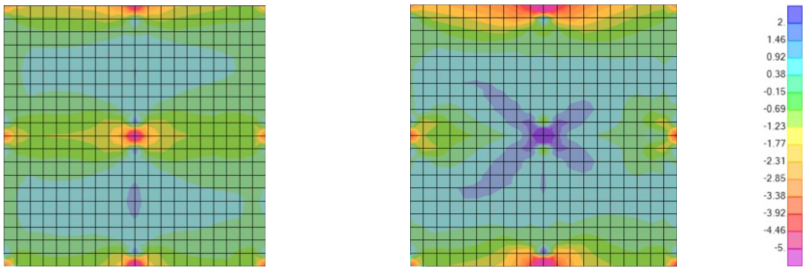
The contours of the principal stress S11 on the bottom surfaces of Slabs 1, 2, 3 and 4 at the top of Storey 1 before and after removing the selected fire damaged Glulam Column B2 below in the same storey once subjected to a 120-min fire are illustrated in Figures 5(a) and 5(b), with the corresponding contours of the principal stress S22 shown in Figures 5(c) and 5(d). Usually, the load paths for the selected Slab 1, 2, 3 and 4 in Figures 5(a) and 5(c) are just like those on the individual two-way spanning slabs. In Figures 5(b) and 5(d), because the selected Glulam column in Storey 1 fails after a 120-min standard fire and is removed from the building model, the load paths will be different and Slabs 1, 2, 3 and 4 will behave more like a whole two-way spanning slab with the doubled floor span in each direction over four rooms with most of the floor loadings transferred to the adjacent Glulam columns in the same storey.

The failure and removal of the fire-damaged vertical members will significantly affect the CLT floor slabs they support. As shown in the principal stress contours for all four indicated CLT floor slabs in the S11 and S22 directions, the stress developments cannot be completely symmetric about axes B and 2, but the stress development trends are similar about axes B and 2 for the tall Glulam building. All the principal stresses in the horizontal x- and y-directions at the 17 selected node points in the two complete building models for the tall Glulam building before and after removing the internal Glulam Column B2 once subjected to a 120-min standard fire are analysed.

For the principal stresses in the S11 direction, the compressive stresses should adopt the average values at four slab corners. In this case, the average values for the four corners are selected from those at node points 1 and 2, node points 4 and 5, node points 9 and 10 and node points 12 and 13, respectively.



(a) S11 before removing the column **(b)** S11 after removing the column



(c) S22 before removing the column **(d)** S22 after removing the column

Figure 5: Contours of the principal stresses on the bottom surface of the selected Slabs 1, 2, 3 and 4 at the top of Storey 1 before and after removing the Glulam Column B2 from the tall Glulam building once subjected to a full 120-min standard fire

Comparing the principal compressive stresses in the S11 direction in the building models before and after removing the internal Glulam Column B2 once subjected to a 120-min standard fire, it is observed that the maximum stress values at the corner points are based on those at node points 4 and 5. Before removing the fire-damaged Glulam Column B2, the average compressive stress in the S11 direction at node points 4 and 5 is -9.719 N/mm^2, while after removing Glulam Column B2, the average compressive stress sharply increases to -25.929 N/mm^2, up by 16.210 N/mm^2 or 166.8%. Also, the large average compressive stress of -8.923 N/mm^2 at node points 9 and 10 at the location of Glulam Column B2 sharply changes to the large average tensile stress of 11.499 N/mm^2 after removing Glulam Column B2, with a big change of 20.422 N/mm^2.

In the S22 direction, the average compressive stress values at four corners are based on those at node points 1 and 16, node points 5 and 6, node points 8 and 9 and node points 13 and 14, respectively. Comparing the principal stress developments in the S22 direction before or after removing the internal Glulam Column B2 once subjected to a full 120-min standard fire, it is observed that the maximum compressive stress for the slab corners comes from the average value of -17.388 N/mm^2 at node points 8 and 9 in the building model before removing the internal Glulam Column B2. After removing the internal Column B2, the stresses at the location sharply change to the tensile stresses, with an average of 13.418 N/mm^2, while the maximum compressive stresses happen at node points 13 and 14, with an average of -34.649 N/mm^2. Due to these large compressive stresses, the indicated CLT floor slabs may have been severely damaged after removing the internal fire damaged Glulam Column B2 in Storey 1.

3.2.2 Design loadings and deflections on the selected Glulam beams

For considering the effects of removing the internal Glulam Column B2 from the building model, not only the stress developments on the CLT floor slabs should be analysed, but also the loadings on the Glulam beams supported by Glulam Column B2 also need to be analysed. In this case, Glulam Beams BC2, AB2, B12 and B23 are all affected by removing the internal Glulam Column B2 supporting these Glulam beams. The design loadings are listed in Table 4, where M_{Ed} indicates the design bending moments on the beams, V_{Ed} indicates the design shear forces on the beams, N_{Ed} indicates the axial design loadings, and Δ indicates the mid-span deflections of the beams.

According to the results listed in Table 4, the design bending moments on the beams vary by -3.129 kNm to 12.428 kNm with the maximum value of 33.268 kNm on Glulam Beam B12. The design shear forces largely increase by 36.170 kN to 53.546 kN, with the maximum value of 67.765 kN on Beam B12. All the Glulam beams sustain axial tension when the indicated Glulam Column B2 in Storey 1 fails and is removed. The design tensile forces increase by 2.142 kN to 10.365 kN with the maximum value of 14.052 kN on Beam B12. The mid-span deflections increase largely by 33.019 mm to 35.554 mm, with the maximum value of 40.800 mm on Beam B12.

Table 4: Design loadings and deflections on the selected Glulam beams at the top of Storey 1 in the building model before and after removing the internal Glulam Column B2

Glulam beams at the top of Storey 1	M_{Ed} (kNm)	V_{Ed} (kN)	N_{Ed} (kN)	Δ (mm)
BC2 before removing column	28.411	15.621	1.953	6.621
BC2 after removing column	25.282	51.791	4.669	40.195
Difference	-3.129	36.170	2.716	33.574
AB2 before removing column	31.954	18.091	4.155	7.603
AB2 after removing column	29.074	49.286	11.803	40.622
Difference	-2.880	31.195	7.648	33.019
B12 before removing column	23.017	14.219	3.687	5.246
B12 after removing column	33.268	67.765	14.052	40.800
Difference	10.251	53.546	10.365	35.554
B23 before removing column	19.653	13.189	1.425	4.393
B23 after removing column	32.081	65.390	3.567	39.090
Difference	12.428	52.201	2.142	34.697

The detailed verifications on the design loadings and deflections are based on the design procedures presented in the structural Eurocodes. Using the maximum design bending moment listed in Table 4, the maximum design bending stress under fire conditions, $\sigma_{m,y,fi,d}$, is verified against its strength $f_{m,fi,d}$ as

$$\sigma_{m,y,fi,d} = \frac{M_{Ed,fi}}{W_{fi}} = \frac{33.268\times10^6}{4.684\times10^6} = 7.102\ \text{N/mm}^2 < f_{m,fi,d} = 27.6\ \text{N/mm}^2.$$

The maximum design shear stress under fire conditions, $\tau_{\mathrm{fi,d}}$, is verified against its strength $f_{\mathrm{v,fi,d}}$ as

$$\tau_{\mathrm{fi,d}} = \frac{1.5 \cdot V_{\mathrm{Ed,fi}}}{k_{\mathrm{cr}} \cdot b_{\mathrm{fi}} \cdot h_{\mathrm{fi}}} = \frac{1.5 \times 67.765 \times 10^3}{0.67 \times 168 \times 409} = 2.208 \text{ N/mm}^2 < f_{\mathrm{v,fi,d}} = 4.03 \text{ N/mm}^2 .$$

The maximum design tensile stress under fire conditions, $\sigma_{\mathrm{t,0,fi,d}}$, is verified against its strength $f_{\mathrm{t,0,fi,d}}$ as

$$\sigma_{\mathrm{t,0,fi,d}} = N_{\mathrm{Ed}} / A_{\mathrm{fi}} = 14.052 \times 10^3 / \left(168 \times 409\right) = 0.205 \text{ N/mm}^2 < f_{\mathrm{t,0,fi,d}} = 19.55 \text{ N/mm}^2 .$$

After the beams are subjected to the 120-min standard fire, the checks on the design bending, shear and axial tensile stresses all satisfy the requirements set in the structural Eurocodes. Also, after removing the internal Glulam Column B2, the spacing between two opposite Glulam columns is simply doubled from 5000 mm to 10000 mm in each direction. The allowance for the deflection of the beam after fire is listed in BS 476 (BSI 1987) as $L/20$, which is 10000/20 = 500 mm > 40.800 mm. The deflection of the Glulam beams is still satisfactory under the fire conditions, i.e. they are globally safe and will not cause proportionate collapse of the building after removing a main fire damaged internal Glulam column. Meanwhile, the allowance for the deflection of the beam under service load is given in the UK NA to BS EN 1995-1-1 (BSI 2004a, b) as $L/250$ = 40 mm < 40.800 mm, indicating that these Glulam beams have failed locally and are no longer serviceable. If the effects of connections between these Glulam members are considered, the situation could be worse. Nevertheless, these affected Glulam beams under fire conditions will no longer be serviceable but will not cause global proportionate collapse or failure.

As mentioned above, the CLT floor slab may have failed after removing the internal Glulam Column B2, and the loadings on the Glulam beams connected to the removed column are also affected. Here, assume another case where both the internal Glulam Column B2 and CLT floor Slab AC13 it supports fail under the fire conditions and are removed from the building model. The loadings on the CLT floor Slab AC13 are still assumed to be transferred to the supporting Glulam beams. The numerically calculated design loadings and deflections on the remaining Glulam beams at the top of Storey 1 after removing the fire-damaged Glulam Column B2 and CLT floor Slab AC13 in Storey 1 are listed in Table 5.

After removing the fire-damaged internal Glulam Column B2 and CLT floor Slab AC13 from the numerical building model for the tall Glulam building, the maximum design bending moment is 62.578 kNm, the maximum design shear force is 40.618 kN, the maximum design axial tensile loading is 18.866 kN and the maximum deflection is 42.288 mm. The detailed verifications on the design loadings and deflections in this case are based on the design procedure presented in the structural Eurocodes.

Table 5: Design loadings and deflections on the selected Glulam beams in Storey 1 after removing Glulam Column B2 and CLT floor Slab AC13

Glulam beams at the top of Storey 1	M_{Ed} (kNm)	V_{Ed} (kN)	N_{Ed} (kN)	Δ (mm)
Glulam Beam BC2	62.578	40.618	13.664	41.761
Glulam Beam AB2	62.578	40.618	14.866	42.122
Glulam Beam B12	62.578	40.618	18.811	42.288
Glulam Beam B23	62.578	40.618	18.046	41.428

The maximum design bending stress under fire conditions, $\sigma_{m,y,fi,d}$, is verified against its strength $f_{m,fi,d}$ as

$$\sigma_{m,y,fi,d} = \frac{M_{Ed,fi}}{W_{fi}} = \frac{62.578\times10^6}{4.684\times10^6} = 13.360\ \text{N/mm}^2 < f_{m,fi,d} = 27.6\ \text{N/mm}^2.$$

The maximum design shear stress under fire conditions, $\tau_{fi,d}$, is verified against its strength $f_{v,fi,d}$ as

$$\tau_{fi,d} = \frac{1.5\cdot V_{Ed,fi}}{k_{cr}\cdot b_{fi}\cdot h_{fi}} = \frac{1.5\times40.618\times10^3}{0.67\times168\times409} = 1.323\ \text{N/mm}^2 < f_{v,fi,d} = 4.03\ \text{N/mm}^2.$$

The maximum design axial tensile stress under fire conditions, $\sigma_{t,0,fi,d}$, is verified against its strength $f_{t,0,fi,d}$ as

$$\sigma_{t,0,fi,d} = N_{Ed} / A_{fi} = 18.866\times10^3 / \left(168\times409\right) = 0.275\ \text{N/mm}^2 < f_{t,0,fi,d} = 19.55\ \text{N/mm}^2.$$

The allowance for the deflection of the beam after fire is now $L/20$ = 10000/20 = 500 mm > 42.288 mm. The deflection of the Glulam beams is still satisfactory under the fire conditions, i.e. they are globally safe and will not cause proportionate collapse of the building after removing a main fire damaged internal Glulam column and the floor slabs above the Glulam beams. At the same time, the allowance for the deflection of the beam under service load is $L/250$ = 40 mm < 42.288 mm, indicating that these Glulam beams have failed locally and are no longer serviceable. Similarly, these affected Glulam beams under fire conditions will no longer be serviceable but will not cause global proportionate collapse or failure.

3.2.3 Design loadings on the adjacent Glulam columns

The design loadings on the adjacent Glulam columns in different design situations also need to be verified. The detailed numerical results of the design axial loadings and compressive stresses on Glulam Columns C2, A2, B1 and B3 before and after removing the fire damaged Glulam Column B2 and CLT Floor AC13 are illustrated in Figure 6 for the tall Glulam building model. Figure 6(a) illustrates the results on the four adjacent Glulam columns before removing the internal Glulam Column B2 in Storey 1 after subjected to the full 120-min standard fire, Figure 6(b) illustrates the results after removing the internal Glulam Column B2 in Storey 1 after subjected to the full 120-min standard fire and Figure 6(c) illustrates the results after removing the internal Glulam

Column B2 in Storey 1 and CLT floor Slab AC13 it supports at the top of Storey 1 after subjected to the full 120-min standard fire.

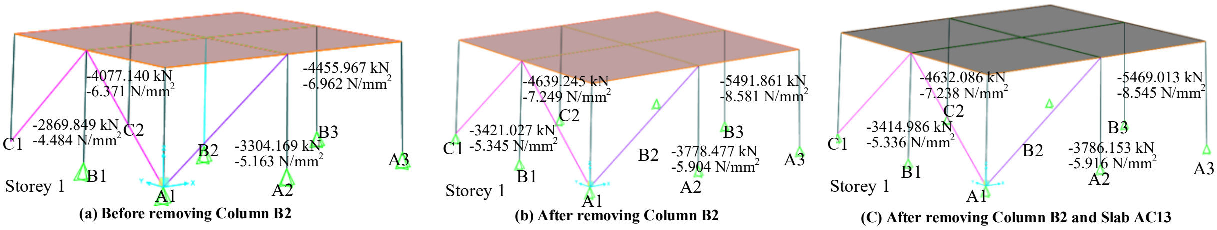


Figure 6: Design axial loadings and compressive stresses on the selected adjacent Glulam columns in Storey 1 before and after removing some fire damaged key structural members

The checks and verifications on the indicated Glulam columns confirm that these four adjacent Glulam columns under different disproportionate collapse situations all satisfy the requirements of the Structural Eurocodes. From the results illustrated in Figure 6, it can be seen that the design compressive loadings or stresses on any of these Glulam columns after removing Glulam Column B2 only or both Column B2 and CLT floor Slab AC13 are very close, but are largely different from those before removing any of those key structural members. The axial compressive loadings and compressive stresses on the indicated Glulam Columns C2, A2, B1 and B3 increase by 13.8%, 14.4%, 19.2% and 23.2%, respectively, after removing the fire-damaged Glulam Column B2. This indeed indicates that the failure and removal of the select fire damaged Glulam column will lead its loadings to be transferred to the adjacent Glulam columns.

The Glulam Columns B2 in Storeys 2, 3 and 4 are also explored and checked before and after removing the Glulam Column B2 in Storey 1. The numerically calculated design loadings and stresses on these Glulam columns are illustrated in Figure 7. The left figure illustrates the results before removing the internal Glulam Column B2 and the right figure illustrates the results after removing the internal Glulam Column B2. After removing the Glulam Column B2 in Storey 1, the Glulam Columns B2 in Storeys 3 and 4 sustain small compressions, while the Glulam Column B2 in Storey 2 sustains small tension. This also indicates that the loadings originally acting on the Glulam Column B2 in Storey 1 have been transferred to the adjacent Glulam columns after its removal due to severe fire damage. All the obtained numerical results demonstrate that the failure and removal of the selected internal Glulam column may lead to the local failure of the adjacent CLT floor slabs, but will not lead to large disproportionate damage and global collapse of the whole tall Glulam Building. Figure 8 illustrates the deformed floor shapes and possible collapse modes of the CLT floor Slabs AC13 in Storeys 1 to 4 of the Glulam building model.

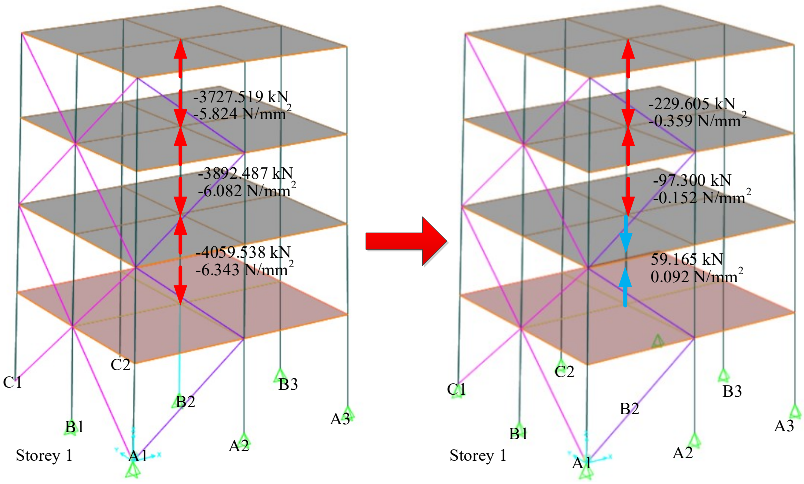


Figure 7: Design loadings and compressive stresses on the selected Glulam Columns B2 in Storeys 2, 3 and 4 before and after removing Column B2 in Storey 1

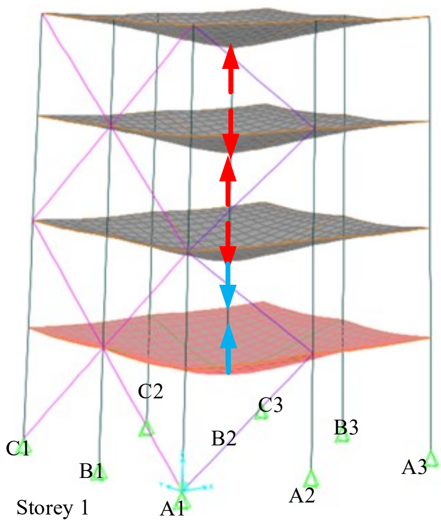


Figure 8: Deformed floor shapes and possible collapse modes of the CLT floor Slabs AC13 in Storeys 1 to 4 of the Glulam building model after removing Column B2 in Storey 1

As shown in Figure 8, Slab AC13 may fail after the failure and removal of a fire damaged internal Glulam Column B2 in Storey 1. In this Glulam building model, the associated beams connected to the Glulam Column B2 at the top of Storey 1 have not failed and they could support the Glulam Columns B2 in the upper storeys. In Section 3.2.2, The maximum design ratios, i.e. the ratios of the design stresses or deflection to their design strengths or deflection limit, are 48.41% for bending, 54.87% for shear, and only 1.41% for tension. The maximum vertical deflection of the Glulam beams is 42.288 mm and the actual floor span is doubled from 5000 mm to 10000 mm after removal of Glulam Column B2 in Storey 1. Based on BS 476, the deflections of Glulam beams still satisfy the safety requirement under fire conditions and will not cause global progressive collapse of the whole Glulam building. However, based on BS EN 1995-1-1 and the corresponding UK NA to the code, these Glulam beams have failed under service conditions and become unserviceable.

4. CONCLUSIONS

In this research, a 105 m high 30-storey tall Glulam building has been numerically simulated under fire conditions using commercial finite element software package SAP2000 and the results have been analysed, including the developments of the design loadings and stresses and disproportionate collapses due to the failure and removal internal column for this building. Most stresses are all smaller than the design strengths except the stresses of the CLT floor slabs. The failure and removal of a fire damaged internal Glulam column largely increase the principal stresses on the CLT floor slabs above in both directions. In the S11 direction, after removing the fire damaged Glulam Column B2 in Storey 1, the average compressive stress sharply increases to -25.929 N/mm^2 or 93.9% of the design compressive strength. In the S22 direction, however, the maximum compressive stresses happen at node points 13 and 14, with an average of -34.649 N/mm^2 or 125.5% of the design compressive strength. The indicated CLT floor slabs may have been severely damaged after removing the internal Glulam Column B2 in Storey 1. The largely increased stresses on the indicated CLT floor slabs may damage the slabs locally, but will not lead to the progressive collapse of the whole Glulam building. This is because the Glulam column in Storey 2 above will hold the Glulam beams and CLT floor slabs below at the top of Storey 1 after the removal of the fire damaged Glulam column in Storey 1.

The damage and removal of the internal Glulam Column B2 in Storey 1 also influence the design loadings and stresses. The maximum design ratios are 48.41% for bending, 54.87% for shear, and only 1.41% for tension. This indicates that these Glulam beams will not fail due to the increased design loadings under the fire conditions.

Large deflections occur at the location of the indicated Glulam column due to the big rise of the actual floor span from 5000 mm to 10000 mm. The maximum deflection of the Glulam beams connected to the fire damaged Glulam Column B2 is 42.288 mm, only 8.57% of the deflection limit according to BS 476 under the fire conditions. However, the maximum deflection is larger than the deflection limit according to BS EN 1995-1-1 and the corresponding UK NA under serviceability limit. This indicates that Glulam beams under fire conditions will no longer be serviceable but will still be safe and will not cause global progressive collapse or failure of the whole building.

All the loadings originally acting on the removed fire damaged Glulam column are transferred to the adjacent Glulam columns through the CLT floor slabs and Glulam beams it supports.

The numerical simulations on this tall Glulam building also confirm that the more vertical elements of structures are provided, the larger robustness of the building structures against progressive disproportionate collapses under fire conditions.

5. ACKNOWLEDGMENTS

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