ROBUSTNESS OF TIMBER STRUCTURE AFTER COLUMN REMOVAL

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PROJECT REPORT SUMMARY

ROBUSTNESS OF TIMBER STRUCTURE AFTER COLUMN REMOVAL

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Timber is increasing in popularity as a construction material for structures around the world given its sustainability. Yet, little is known about the long-term behaviour of timber structures. Moreover, few studies have focused on the robustness of timber structures.

Robustness in timber structures refers to the capacity of the building to avoid disproportionate collapse due to an abnormal event. The structure is analysed with a holistic framework system which encompasses the ability of connected members and surfaces to withstand deformation, redistribute loads, and preserve the overall structural performance of the system. The initial damage may induce deformation propagation to a section and/or to the entire structure causing it to collapse progressively. The study numerically investigates this behaviour of timber structures by considering column removal of a case-study building. Column dimensions are varied in the analyses to investigate the effect that the column size has on the resistance to progressive collapse of the structure. The failure of the column is instantaneous, and the structure is analysed with the removal of preceding members and surfaces that collapsed.

The relationship between various column cross sections and the ability of the structure to withstand progressive collapse shows that with increasing column size the deformation decreases. It is seen that increasing the column section dimensions improves the resistance to progressive collapse of the structure however, this relationship is not definite.

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LIST OF SYMBOLS

°C Degrees Celsius

kN Kilonewton

kPa Kilopascal

m metre

mm milimetre

Mpa Megapascal

$P[C]$ probability of disproportionate collapse as a result of an abnormal event.

$P\{C|D\}$ probability of a disproportionate spreading of structural failure, C, due to the initial damage D.

$P\{D|E\}$ probability of initial damage, D, in consequence of an abnormal event, E.

$P\{E\}$ probability of occurrence of an abnormal event.

LIST OF ABBREVIATIONS

3D Three-dimensional

ALPA Alternative Load Path Analysis

CLT Cross-Laminated Timber

EU European Union

EWPs Engineered Wood Products

FEA Finite Element Analysis

FEM Finite Element Method

Glulam Glue-laminated timber

RFEM *Räumlich* Finite Element Method

SLS Serviceability Limit State

ULS Ultimate Limit State

v. versus

1 INTRODUCTION

1.1 BACKGROUND

The use of timber as a building material dates back throughout various civilisations including the Romans, Egyptians, and Anglo-Saxons where timber was widely employed in the construction of houses, temples and other structures. As time progressed, new construction techniques and methodologies emerged that led to the development of Engineered Wood Products (EWPs) like Cross-Laminated Timber (CLT) and Glue-laminated timber (Glulam) during the 19th and 20th centuries.

In contrast to reinforced concrete and structural steel buildings, timber structures offer several advantages. Firstly, timber has the ability to store carbon by sequestering it and preventing its release as carbon dioxide into the atmosphere. Additionally, timber construction can be cost-effective and more efficient in terms of construction time. However, the construction of multi-storey timber structures is a relatively new phenomenon that requires better understanding of their behaviour (Lyu et al, 2021).

The construction industry can benefit from the use of timber by providing a renewable building material. The research contributes to the application of timber buildings, encouraging their wider adoption as a sustainable and resilient alternative to traditional construction materials.

1.2 OBJECTIVES OF THE STUDY

The objective of this research is to determine the effect of column size on the robustness of a timber structure in the event of an interior column removal. The focus is on exploring the phenomenon of progressive collapse, which refers to the structural behaviour and stability of a building when subjected to an abnormal or extreme event.

By removing an interior column, the research aims to simulate a scenario where a critical element fails in the timber structure. The investigation seeks to analyse how the structure responds to such an event and to evaluate its ability to maintain structural integrity. The specific emphasis is on studying the progressive collapse behaviour of the timber structure which involves the analysis of subsequent member and surface failures that may occur as a result of the initial element loss.

1.3 SCOPE OF THE STUDY

The study investigates the global deformation of the structure when an interior column is removed in the design model. The behaviour of members and surfaces are linear elastic and materials are isotropic. It is not within the scope of the study to determine the cause of an element loss; it is assumed that it fails instantaneously due to accidental loading. A numerical model representing a three-story timber office building was designed in RFEM 6.0. A physical experiment was not feasible to conduct in this study.

1.4 METHODOLOGY

The robustness of the timber structure is accessed using Dlubal RFEM 6.02, a comprehensive software for structural analysis and design. Initially, the original structure is analysed to ensure its stability and compliance with design check ratios for members and surfaces. This serves as a baseline assessment of the structure's integrity. Then, an interior column is selectively removed, and the analysis is repeated to investigate the effects of the element loss. Any members or surfaces that fail to meet the design check ratios due to the element removal are subsequently removed from the structure.

The process is repeated for the three different column sizes whilst keeping all other dimensions of the structure constant. By comparing the global deformation in each iteration scenario for various column cross sections, the influence of column size on the structure's robustness can be evaluated.

1.5 ORGANISATION OF THE REPORT

Chapter one acts as an introduction to the report, outlining its main objective and relevance. It provides the research's purpose and objective. The chapter puts into context the study within existing literature, highlighting its potential contributions and implications for the field of study.

Chapter two is a literature review on Robustness of Timber Structures. The chapter outlines the benefits of Timber as a construction material, and the standards that govern them. It discusses the different analysis approaches and design methods carried out for Robustness.

Chapter three describes the methodology of the study, which entails the calibration of the numerical model. The process is motivated and validated for credible results.

In Chapter four, the case-study's results are discussed in terms of the effect of column size on Robustness. Importantly, the chapter presents the limitations of the system and specifies the assumptions made during the model's analysis.

Chapter five concludes the case-study by answering the objective of the research and recommendations on future research paths.

Lastly, Chapter six acknowledges and lists the references used in the research.

2 LITERATURE REVIEW

2.1 TIMBER STRUCTURES

Timber structures have a historical precedent, with ancient wooden structures in Asia and Europe still standing today. However, concerns about fire safety and building codes hindered their construction until recent advancements in fabrication techniques and environmental considerations which brought about the resurgence of mass timber buildings. Modern timber buildings have emerged, showcasing the potential of timber as a construction material. Despite the successes in the field, there is a need to address misconceptions surrounding long-term performance and strength, as public perception often underestimates the safety and durability of tall timber structures compared to reinforced concrete buildings (The Economist, 2018 and CNN, 2018).

Figure 2-1: 'El Amor de Chile' at Expo Milan 2015 (Roland Halbe, 2015)

The concern with timber buildings is not related to fire or structural weakness but rather the potential for significant errors due to the complex nature of timber as a building material. As timber structures scale up to new heights, there is a risk of encountering unexpected challenges (Köhler, 2006, Madsen, 1995).

Timber is a naturally grown material that degrades when exposed to prolonged moisture or wet-dry cycles. Designing timber structures to withstand changes in moisture levels is crucial especially with high-rise timber structures (Glass *et al*., 2010). Over time, this can reduce the load bearing capacity of the section, and the management of differential settlement becomes challenging in hybrid buildings with load-bearing timber elements (Jockwer *et al*., 2018). Moreover, the lightweight nature of timber in comparison to reinforced concrete and structural steel introduces a unique vulnerability. Consequently, as altitude increases, timber buildings become more susceptible to critical wind loads, necessitating meticulous engineering solutions (Glass et al., 2010).

Due to the orthotropic nature of timber, it possesses different sectional properties along different directions. It is stronger along the fibres of the timber but weaker in the transverse direction. This material property needs to be taken into account in the design process (Glass *et al*., 2010). Furthermore, constructing and forming rigid connections between timber members is challenging due to the orthotropic behaviour of timber; its mechanical behaviour is not isotropic, but varies depending on the direction of the applied forces (Tulebekova *et al*., 2023).

The stiffness of timber is much lower compared to reinforced concrete. This has a significant impact on the section resistances of members. When subjected to tension, bending, and shear forces, timber undergoes brittle behaviour A brittle material is defined as a material that fails when a single particle in the material fails. Brittleness can hinder load redistribution in structures, which is one of the design methods for robustness (Bolotin, 1969).

The complexities involved in designing timber buildings can pose significant challenges. Limited practical knowledge and the difficulty of testing large assemblies further complicate the understanding of these structures. However, these complications should not undermine the environmental and economic advantages that mass timber buildings offer (Voulpiotis *et al*., 2022).

2.2 THE CASE FOR TIMBER

Timber refers to processed wood that has been converted into sections intended for use as beams, columns, and various structural components. One of the advantages of timber structures is their environmental sustainability. Wood is a renewable resource that can be harvested from responsibly managed forests, promoting sustainable forestry practices (Green and Karsh, 2012). Its inherent strength and adaptability establish timber as a construction material with additional attributes throughout the construction process (Harte, 2009).

The utilisation of timber as a building material offers notable advantages in terms of project delivery speed and cost reduction compared to traditional materials. The construction process is faster and more streamlined when working with timber due to its higher potential for prefabrication. The connections and detailing for timber are simpler than steel connections. The construction of such buildings allows the building to be put in use, thereby expediting return on investment. Furthermore, overall material costs are reduced because there is no need for secondary steelwork (Ramage *et al.,* 2017). Furthermore, Engineered Wood Products (EWPs), such as Glue-laminated Timber (Glulam), Laminated Veneer Lumber (LVL) and Cross-Laminated Timber (CLT) are prefabricated offsite. They offer enhanced strength and dimensional stability, allowing for the construction of larger and more complex timber structures that result in fewer deliveries to site (Schickhofer *et al*., 2016). In timber construction, onsite labour like welding and plastering is not as common, which can result in fewer personnel required on site. The self-weight of timber is significantly lower than concrete hence, shallower foundations are required which can further reduce capex (Ramage *et al.,* 2017).

Structures built from timber encompass a wide range of applications from residential housing to bridges, and high-rise structures. As the use of Timber expands in the Built Environment, it can add to the existing century old structures as well as the mass timber structures today. Lastly, timber can be a step towards sustainability and to preserve natural resources for future generations (Igwe, 2021).

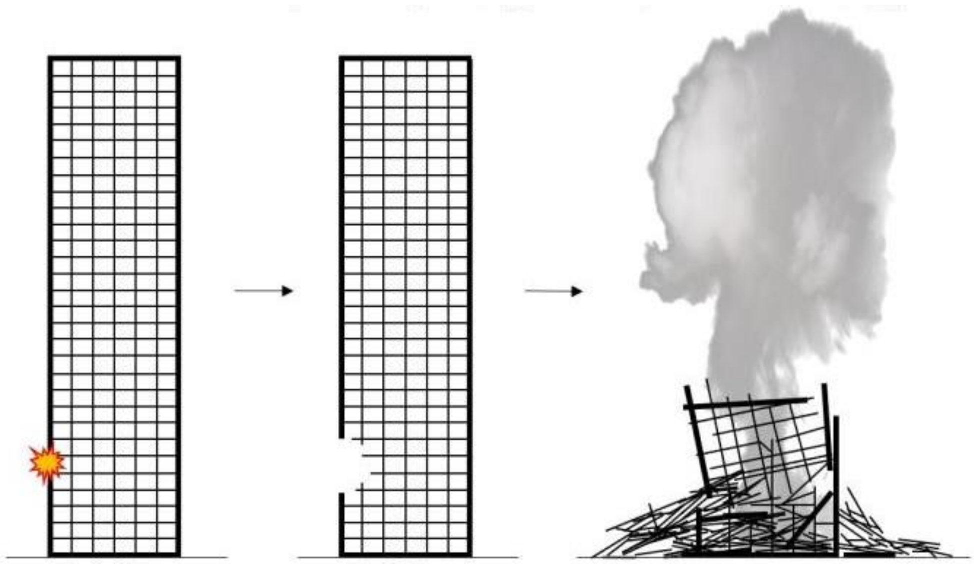
2.3 ROBUSTNESS

The partial collapse of the Ronan Point Tower in London 1968, shown in Figure 2-2, triggered discussion around the disproportionate collapse in buildings and its prevention. The 22-storey precast concrete large-panel tower collapsed from the corner bay due to the failure of a loadbearing wall panel (Griffiths *et al*., 1968). There was no provision of an alternative load path (Russel *et al*., 2019). The consequences of the event deemed unacceptable relative to the initial damage which brought light to the importance of structural robustness, a phenomenon that cannot be implicitly assumed to be inherent to all structures (Bussell and Jones, 2010).

Figure 2-2: Partial collapse of Ronan Point Tower (left) London, 1968 (Daily Mail)

The Bad Reichenhall Ice Arena in Germany 2006 was a long-span timber roof. One roof girder failure resulted in the collapse of the entire roof (Winter and Kreuzinger, 2008). The cognizance is that moisture-sensitive glues should not be used in permanent timber structures (Munch-Andersen and Dietsch, 2009).

A disproportionate collapse can occur in a progressive manner, but this is not always the case. This implies that the initial failure of a structural member causes a few structural components to fail which then triggers a cascading failure of other structural members that were not directly impacted by the initial event. The sequence of these events is referred to as progressive collapse (Starossek and Haberland, 2010). The development of disproportionate collapse is illustrated in Figure 2-3.



2-3.

$\boldsymbol{P\{E\}}$ $\boldsymbol{P\{D|E\}}$ $\boldsymbol{P\{C|D\}}$

Figure 2-3: Development of progressive collapse (Voulpiotis, 2021)

With reference to Figure 2-3, a disproportionate collapse can develop in three stages. First, an unforeseen or abnormal event, which is not considered in the conventional design of structures due to its low probability of occurrence $P\{E\}$, acts on the structure. Second, the extended loading causes an initial damage, $P\{D|E\}$. The initial loss is due to the initial cause, and it can be locally limited and does not include the response of the whole structure. Possible effects are weakening and/or failure of the member as it may experience a total loss or reduction in the load-carrying capability of the section. Third, the initial damage causes failure to spread throughout the structure resulting in a disproportionate collapse, $P\{C|D\}$. The response of the structure to the damage is an inherent structural characteristic assessed through scenario analyses where the initial damage is assumed and independent of specific abnormal events, $P\{E\}$, referred to as notional damage in Equation 2.1(Starossek and Haberland, 2010).

$$P\{C\} = P\{C|D\} \cdot P\{D|E\} \cdot P\{E\} \qquad \text{(Equation 2.1)}$$

$P[C]$: probability of disproportionate collapse as a result of an abnormal event.

$P\{C|D\}$: probability of a disproportionate spreading of structural failure, C, due to the initial damage D.

$P\{D|E\}$: probability of initial damage, D, in consequence of an abnormal event, E.

$P\{E\}$: probability of occurrence of an abnormal event.

2.4 EFFECT OF COLUMN SIZE

An experiment to investigate the size effect of timber columns was conducted by Fryer *et al., (2018).* The columns were glulam with four sets of samples for three column sizes namely: 120 mm x 360 mm, 240 mm x 720 mm, and 360 mm x 1080 mm. The timber was graded with a characteristic strength of 24 MPa in compression and tested with an Amsler rig with a maximum capacity of 4900 kN. The column dimensions were chosen such that it would be able to fail in the machine. The compressive rig applies the load under displacement-controlled conditions.

The column strength reduced from 40.3 MPa to 39.5 MPa between the smallest and largest column sizes. Three of the four 360 mm x 1080 mm samples did not fail hence, their failure was estimated from analysis of their stress-strain curves. The results of the compression tests are shown in Table 2-1 and Figure 2-4, which illustrates the trends that can be extrapolated from the data set with a Weibull analysis and an Energetic Statistical analysis.

Table 2-1: Results from compression test (Fryer *et al.*, 2018)

Column Size	**120 mm x 360 mm**	**240 mm x 720 mm**	**360 mm x 1080 mm**
Average Force (kN)	580	2311	5114
Average Failure Stress (MPa)	40.3	40.1	39.5
Standard deviation (%)	5.3	4.4	2.1

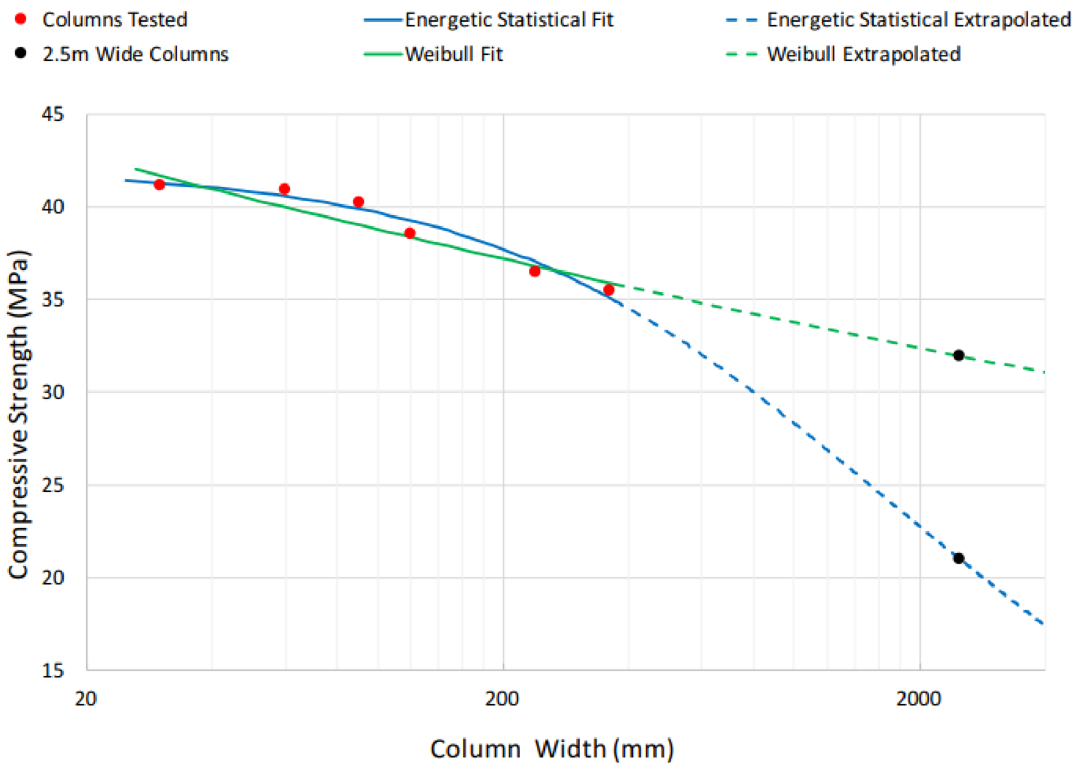


Figure 2-4: Graph illustrating the size effect of glulam columns (Fryer *et al*., 2018)

The tests show a reduction in compressive strength, parallel to the grain, with increasing column dimensions. Although there are relatively small changes in the compressive strengths across the samples, the load-bearing capacity of columns in mass timber structures can be reduced to size effect (Fryer *et al*., 2018).

2.5 DESIGN STANDARDS AND GUIDELINES

The consideration of robustness in structural design is a requirement in most major codes and standards. However, the specific methodologies and approaches to address robustness may vary across different countries and codes.

In South Africa, the South African National Standards (SANS) 10160-1 (2019) addresses Robustness in Clause 4.4.1, which states that a structure shall be designed for accidental loading situations and actions to provide compliance with the basic requirements. Additionally, the structure must have the ability to prevent widespread failure and not be damaged to an extent disproportionate to the initial cause.

In Eurocode 1 (CEN, 2002), robustness is mentioned in the context of accidental loads. The code provides guidelines and requirements for structural robustness of buildings under accidental events, such as explosions or seismic loads.

In the United States, the ASCE-7 standard (American Society of Civil Engineers, 2016) adopts a similar approach to the Eurocodes regarding robustness. The standard includes provisions for the design of structures to withstand accidental events and mitigate the consequences of unforeseen circumstances.

In Denmark, the design rules related to robustness are specifically applied to structures where the consequences of failure are considered serious. The Danish Code of Practice for the Safety of Structures outlines an analysis framework for assessing and addressing robustness, considering the potential consequences of structural failures (DK EN, 1990).

As the understanding of robustness in structural design continues to evolve, there is ongoing research and development aimed at harmonising and improving the guidelines, and methodologies related to robustness across different codes and standards. This can warrant consistent and effective approaches to address robustness requirements in structural design.

2.6 ANALYSIS METHODS

The analysis and quantification of robustness can be approached through three different analyses namely a risk analysis, reliability analysis, or deterministic analysis. Risk and reliability analyses are probabilistic methods that consider probability distributions of building exposure and material parameters. On the other hand, deterministic analysis can be conducted in a pragmatic manner and serves as a complementary approach to a probabilistic analysis. Both probabilistic and deterministic analyses yield measures to quantify robustness (Adam *et al*., 2018).

In risk analysis, the total risk is accessed by modelling different paths in decision trees. These paths consider various exposures and potential damage states. Each damage state may lead to system failure with a certain probability, and consequences thereof (Adam *et al*., 2018).

In contrast, reliability analysis quantifies the probability of a structural system's performance over its designed service life. In deterministic analysis, the structural response to an initial damage is evaluated. Notional damages considered in scenario-independent approaches. The analysis focuses on the building's ability to sustain damage, often by removing a load-bearing element. Scenario-dependent approaches consider specific exposures such as explosions, earthquakes, or fires (Adam *et al*., 2018).

These different analysis approaches provide insights into the robustness of a structure and help in quantifying its ability to withstand various disturbances and exposures (Adam *et al*., 2018). The analysis methods, in order of increasing complexity, are listed in Table 2-3.

Table 2-3: Robustness analysis approaches (adapted Huber *et al*., 2018)

Method	Advantages	Disadvantages
Deterministic approach	Simple and efficient. Stable model is only analysed.	Oversimplification of reality which may be misleading.
Probabilistic approach	Imperfections and uncertainties are calibrated in the model.	The propagation of uncertainties in the model can be timely.
Risk based approach	The most comprehensive approach capable to quantify robustness.	Challenges in quantifying consequences. Not well known in industry.

Irrespective of selected approach, a deterministic structural model is required in robustness analyses. The typical form of damage applied in these analyses involves the removal of the critical column (Adam *et al*., 2018).

2.7 DESIGN METHODS

The two common design methods are either direct, focusing on specific damage scenarios, or indirect, which are scenario-independent approaches (Huber *et al*., 2018).

The Alternative Load Path Analysis (ALPA) is the primary direct approach that involves designing a structure explicitly to withstand a certain type of damage, such as the removal of a column. Alternative load paths are provided from the point where the load is applied to a point in the structure where the resistance is provided. This approach transfers the load away from the failed components to prevent the damage from spreading. ALPA can be provided by direct design only if the structure is deemed insensitive to abnormal events or notional damage. This method is therefore applied in a threat-specific or threat-unspecific manner (Huber *et al*., 2018).

The indirect method involves providing minimum tie forces and to introduce redundancies on critical elements. Both design methods increase the structure's resistance to progressive collapse (Huber *et al*., 2018). Lastly, the structure can be compartmentalised by dividing the structure into segments such that a collapse is prevented from propagating to the entire structure.

Segmentation offers an alternative approach to improve the robustness of a structure by preventing or limiting the propagation of failure after an abnormal event. This method entails establishing segment borders, which isolate the failing part of the structure from the rest. Segment borders can be formed by making use of components that are able to resist collapse, weak components that allow safe disconnection, or elements with a high ductility (Starossek, 2018). The design methods are depicted in Figure 2-5.

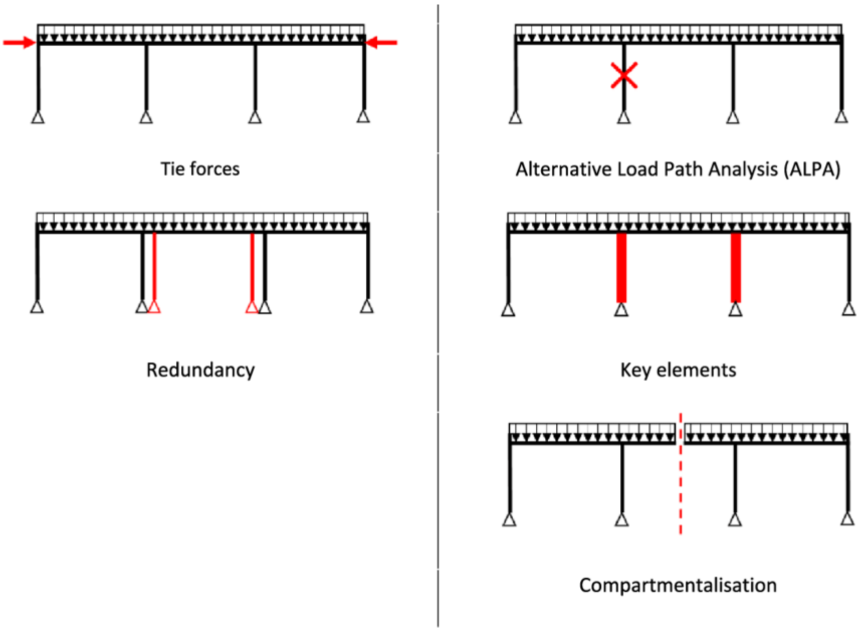


Figure 2-5: Design Methods (Voulpiotis, 2021)

2.8 SUMMARY

Timber structures, with their historical significance and recent resurgence driven by advanced fabrication techniques and environmental considerations, face a need to understand their long-term performance and strength, particularly in tall constructions. Ensuring robustness is crucial, as historical incidents like the Ronan Point Tower and Bad Reichenhall Ice Arena emphasise. To combat this, various codes accentuate preventing widespread failure and disproportional damage in structural design. In the research, understanding the influence of column sizes on robustness contributes to designing buildings that can withstand unforeseen events without compromising their stability.

3 NUMERICAL MODEL

3.1 INTRODUCTION

Numerical models use Finite Element Analysis (FEA). It is a computational technique that breaks down a complex problem into smaller elements that are connected by nodes. A FEA simulation generates a mesh that consists of numerous finite elements that represent the overall form. The mesh converts the three-dimensional (3D) structure into a sequence of mathematical points, which can then be analysed. Computations are performed for each individual element in the mesh, and the results are combined to formulate the final result (English, 2019).

Several software packages incorporate design codes and regulations for compliance with structural safety standards and design requirements. *Räumlich* Finite Element Method (RFEM), by Dlubal Software, offers several international design standards, which allows users to perform design checks for different structural components. *Räumlich*, which translates to 'spatial', refers to the possibility to analyse 3D models (spatial) using the Finite Element Method (FEM), (Dlubal, 2019).

RFEM 6 allows for the interchange of models, analysis results, and design information, which can streamline interdisciplinary collaboration. The application tools generate reports, including calculations, diagrams, and documentation, which can facilitate communication with clients, regulatory authorities and construction teams. A 3D numerical model offers the advantage of creating a representation of the structure. In contrast, physical experiments can be expensive and logistically challenging to replicate the full-scale behaviour of the structure (Antunes do Carmo, 2020).

3.2 TIMBER SECTIONS

To gain an insight into the robustness of Timber structures, the model in this study aims to replicate the response of a building experiencing element loss. As a case study, a 3 x 4-bay, three-storey high representative Timber building is designed for office use. Each bay is 5 m x 5 m, and the column length in each storey is 4 m floor-to-floor. The area of the structure is 15 m x 20 m. The plan and side view layouts are shown in Figure 3-1 and Figure 3-2, respectively.

The columns are pin connected to the ground surface. All beams and columns are Glued laminated timber (Glulam) structural products and Cross Laminated Timber (CLT) surfaces. Engineered Wood Products (EWPs) such as Glulam, CLT and Laminated Veneer Lumber (LVL) are gaining international recognition as a result of legislative changes and an increased awareness on sustainability (Bezabeh, 2018).

The material properties for the beams and columns are defined as Isotropic Linear Elastic. Its section properties do not vary with direction; it is uniform in all directions. The relationship between the load applied to the structure and its response is linear. Additionally, the applied loads in the analysis do not vary with time, ignoring inertial and damping forces. The basic property of the material is listed in Table 3-1, and the detailed section properties are in Appendix A.

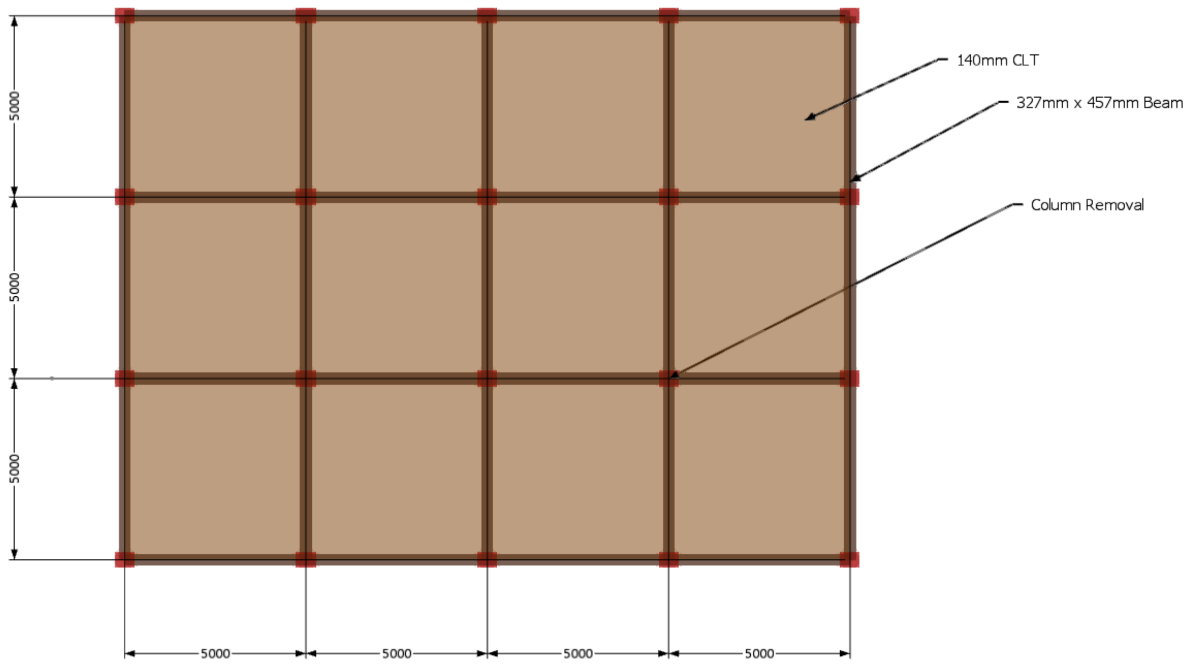


Figure 3-1: Plan view of case study Timber Structure

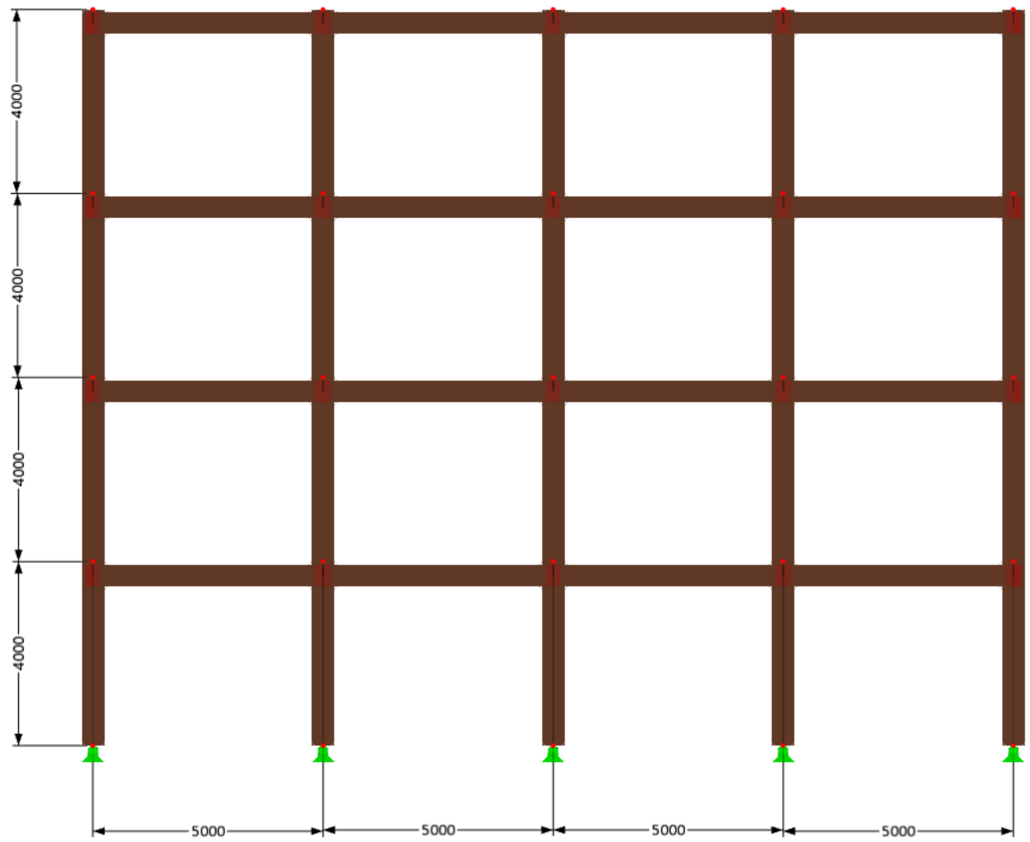


Figure 3-2: Side view of case study Timber Structure

Table 3-1: Timber basic section properties

Description	Symbol	Value	Unit
Modulus of elasticity	E	9500.0	N/mm^2
Shear modulus	G	590.0	N/mm^2
Mass density	ρ	570.00	kg/m^3
Specific weight	γ	5.70	kN/m^3
Coefficient of thermal expansion	α	0.000005	$1/°C$

3.3 CROSS LAMINATED TIMBER

There is an increase in the production and implementation of Cross-laminated Timber (CLT) construction in the past two decades (Brandner, 2016). CLT panels are made up of laminated perpendicular layers of parallel solid wood boards that may support loads about its strong and weak axes. Due to its configuration, CLT exhibits consistent strength that is achieved through its ability to disperse defects, and it may span in two perpendicular directions (Gagnon and Pirvu, 2011).

The CLT surface is defined as Orthotropic Linear Elastic, which is recommended for timber surfaces as it displays different properties in three perpendicular directions with distinct characteristics along its longitudinal, radial, and tangential axes. Each layer of timber in CLT restricts the dimensional changes of the adjacent layers at right angles to one another. This results in similar uniformity across products from different production groups (Schickhofer, 2016).

The CLT in the case-study consist of five layers - the two outer layers are 40 mm, and the centre layers are 20 mm thick – commercially available by Binderholz in the European Union (EU). The CLT cross-section is depicted in Figure 3-3 with a self-weight of 3.15 kN/m.

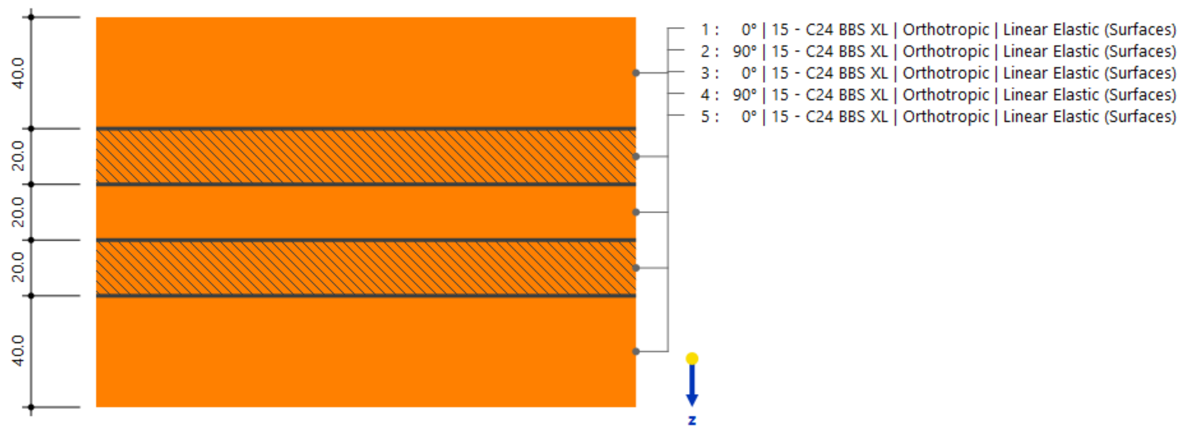


Figure 3-3: CLT cross-section

The timber design add-on module evaluates the adequacy of the timber components to withstand the applied loads. Timber components that fail under quasi-permanent loads, with a deflection greater than 20 mm as it is the maximum amount of deformation a 5 m surface length is permissible (EN,1990) are flagged and displayed in the model. This allows for modifications to optimise the structure to improve the structural integrity, safety, and economy.

3.4 LOADING

The superimposed dead load representing floor finishes is 1 kPa, and a variable load for office use of 3 kPa. The self-weight of the 327 mm x 457 mm Glulam Beam is 0.85 kN/m. The self-weights for the 342 mm x 444 mm, 394 mm x 495 mm and 445 mm x 546 mm columns are 0.87 kN/m, 1.10 kN/m and 1.34 kN/m respectively. Furthermore, the column sections have similar aspect ratios, which is the height divided by the width of the cross-section, equating to 1.30, 1.26 and 1.23, respectively. The detailed section properties are in Appendix A, outlining the section values in relation to bending, shear, torsion and plasticity.

Timber sections that are currently available in South Africa are limited to two manufactures namely, Mass Timber Technologies (MTT) and XLAM. This limits the range of commercially available Glue-laminated Timber (Glulam) and CLT sections in South Africa. Therefore, the sections used in the programme are currently only available in the European Union (EU). The Eurocode 2 (EN:2) is followed and adhered to in this case-study.

The Ultimate Limit State (ULS STR/GEO) philosophy is used for sizing elements, but Serviceability Limit State – Quasi-permanent - for robustness checks (EN 1990, Timber). In ULS, the dead and live loads are amplified by a factor of 1.35 and 1.50 respectively. In SLS, the permanent and imposed loads are multiplied by a magnitude of 1.60 and 1.18 respectively.

3.5 DESIGN APPROACH

Rfem 6, by Dlubal Software, has several analysis capabilities, including linear and non-linear static analysis, dynamic analysis, and stress-strain analysis. This versatility enables the analysis of multiple loading conditions and their effects on the structure.

Linear analysis is suitable for evaluating the Service-Level Earthquake (SLE) and for the Design Earthquake (DE), (PEER/TBI, 2017). Part of the properties sought after in seismic design are also regarded in the robustness of structures. Both analyses consider events that have a low probability of occurrence, which makes the actions challenging to quantify (Branco and Neves, 2009).

3.6 COLUMN REMOVAL

Three different column sizes were selected through an iterative process based on code compliance of structural components under Ultimate Limit State (ULS) philosophy. Importantly, the only variable that changes across the three structures is the column dimensions. The members were chosen so that all three buildings are stable, whilst keeping the beam size, CLT floor and loads constant. In the ULS analyses, the first interior column carried the maximum load in each configuration. The 342 mm x 444 mm column resisted an ultimate load of 455 kN, illustrated in Figure 3-4. Furthermore, the 394 mm x 495 mm and 445 mm x 546 mm columns resisted a total force of 458 kN and 462 kN, respectively.

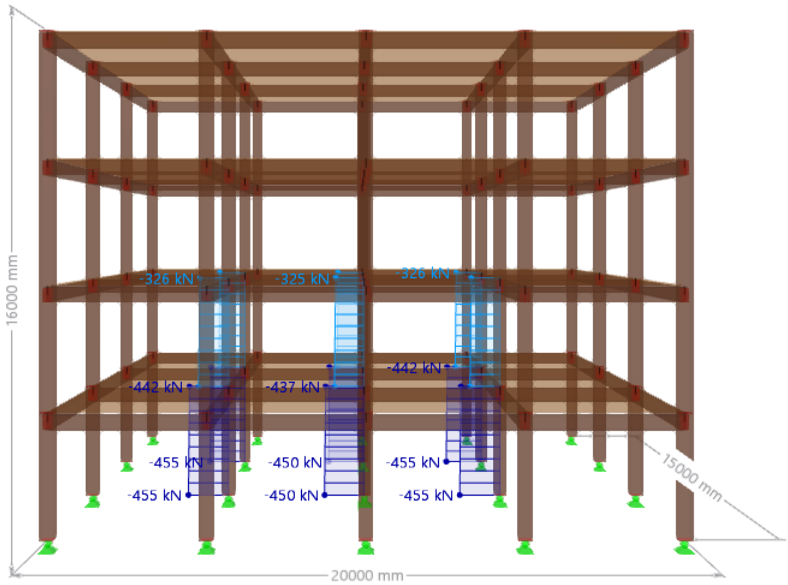


Figure 3-4: Maximum interior column force in 342x444 column

The column is assumed to fail instantaneously due to accidental loading, and it is therefore, removed from the structure. After the column is removed, the structure is analysed under SLS loading. The programme flags members and surfaces that do not meet design requirements, and the failed elements are subsequently removed from the building. The remaining load-bearing members that meet design conditions are assessed for the successive analysis. This iteration is repeated 10 times for each of the three columns. The steps to the process is summarised in Table 3-2.

Table 3-2: Analysis Framework

Step	Description
1	Removal of interior column from structure.
2	Perform structural analysis of structure without interior column.
3	Identification of members that do not meet design criteria. The inadequate members are removed from the model.
4	The structure is analysed once more, and step (3) follows. This step is repeated for 10 iterations.

3.7 VALIDITY

A sensitivity analysis is continuously performed on each structure by varying input parameters and observing the changes in the model's output. This aids in a better understanding of the programme and the response of the structure.

In this research, validation with analytical solutions is employed to compare the simulation results with the analytical results. The forces are calculated from first principles by making use of analytical theory. It can be a strenuous task to solve complex structures, which is a limitation of this method. However, it is for this reason numerical models are created (Godoy and Dardati, 2001), (Archambeault and Connor, 2008).

4 ANALYSIS AND DISCUSSION

4.1 INTRODUCTION

For robustness, the structure is evaluated using a systems approach. A holistic perspective of the entire structure when a column is removed is analysed. This approach recognises the influence of connected parts, and the effect that an initial damage can have on members as failures can propagate, which could lead to progressive collapse of the structure.

Table 3-2 outlines the procedure to access the effect of column sizes due to the removal of an interior column. In the analysis conducted for this study, the timber structure is examined with three different column configurations: 342 mm x 444 mm, 394 mm x 495 mm and 445 mm x 546 mm, beam and CLT sections are constant. Moreover, all applied forces have a uniform distribution, which includes the self-weight of structural elements and the building design for office use.

Importantly, the model serves only as an approximate representation of a structure's behaviour in a real-life scenario. Therefore, the results of the research are only applicable to the case-study examined and does not necessarily reflect the behaviour of all timber structures.

4.2 SYSTEM CONSTRAINTS

The timber beam, column and floor sizes were confined to the available materials and section sizes that are within the Timber Design add-on module. The material properties of timber beams and columns were assumed to be isotropic linear elastic. The CLT floors were inputted as an orthotropic linear elastic material with standard stiffness type and plane geometry. All timber components are assumed to be in the category of Service class 1 – Dry. This entails that the structure is exposed to a temperature of 20°C, and the relative humidity of the surrounding air exceeding 65% for a few weeks per year.

The study does not implement the design of different connection and detailing types between members and surfaces hence, structural elements are connected with no eccentricity, and do not undergo initial sway. Lateral forces, such as wind loads, are not taken into account, it is presumed that shear panels provide adequate bracing in this respect as to confine the study to the timber components. The system is therefore analysed under gravity loads, subjected to a first-order static analysis. When an element fails, it is deactivated in the programme. In reality, the failed member can cause more damage as a consequence of the way it collapses. It can fall on the floor below causing an additional impact force, which can have a ripple effect on the rest of the structure. The impact of this scenario is not assigned in the analysis. Moreover, not all design codes are supported in the Timber Design analysis. Regional codes, such as SANS 10160 (2019), may be available to select, but it is currently not supported in the application. As a result, the study makes use of the Eurocode, EN-1990, CEN 2014-05.

4.3 RESULTS AND DISCUSSION

In this study's analysis, the structure is examined using three different column configurations, which comprised of 342x444, 394x495, and 445x546 rectangular timber sections that are referred to as small, medium and large, respectively. The purpose of this analysis is to evaluate structural robustness and compare the performance of these three structural systems. The first three iterations in the simulation reveal a sequence of failures, specifically affecting CLT floors numbered 12, 11, and 23, respectively. The first substandard panel is located on the roof's edge, closest to the removed column. Subsequently, surface 11 fails, which is situated adjacent to the first failed roof edge panel. The next failure was the floor directly under surface 12 in the third storey of the building model. The sequence of failure by surface number is 12, 11, 23, 8, 24, 36, 9, 48, 20 (below 8), 35 and 47 for the ten iterations respectively. The positions of surfaces are shown in Figure 4-1 and Figure 4-2.

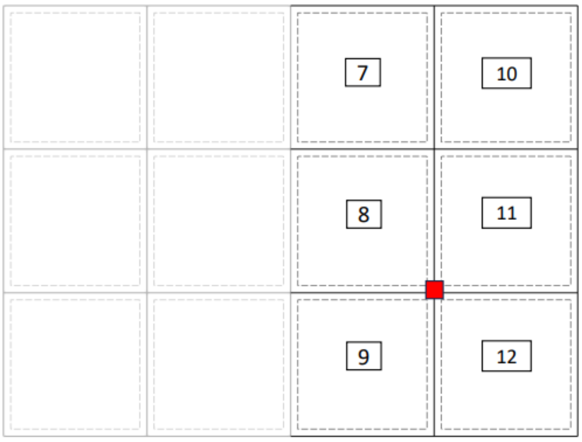


Figure 4-1: Surface numbering plan view

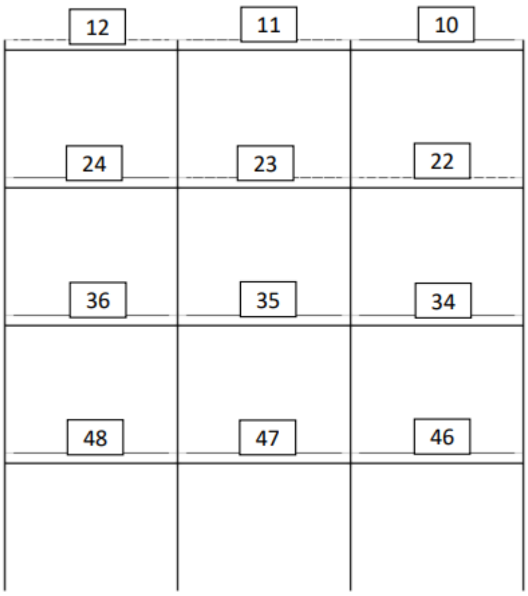


Figure 4-2: Surface numbering side view

The removal of the load-bearing column creates additional strain in and around that region. Figure 4-3 illustrates the global deformations experienced by the structure designed with 445x546 column sections upon the removal of the first interior column exhibiting a maximum global deformation of 27.9 mm.

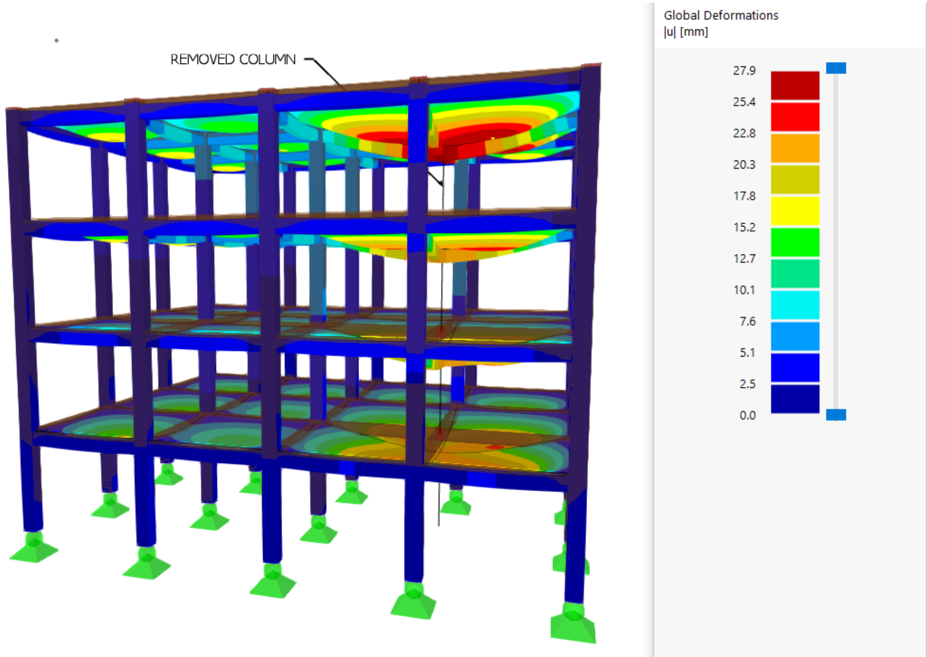


Figure 4-3: Global deformation due to 445x546 column removal

The analysis reveals that the smallest column, 342x444, experiences the most progressive displacement, resulting in a global deformation of 302.48 mm. The medium-sized column, 394x 495, yielded less overall structure displacement with an accumulated 279.08 mm deformation. Lastly, the largest column, 445x546, displayed a final deformation of 263.02 mm, the least deformation across the three sections after the 10 iterations. In Figure 4-4, the graph showing the relationship between the accumulative displacement of the structure against each iteration number indicates that the increasing column section results in a more robust system with lower deformations.

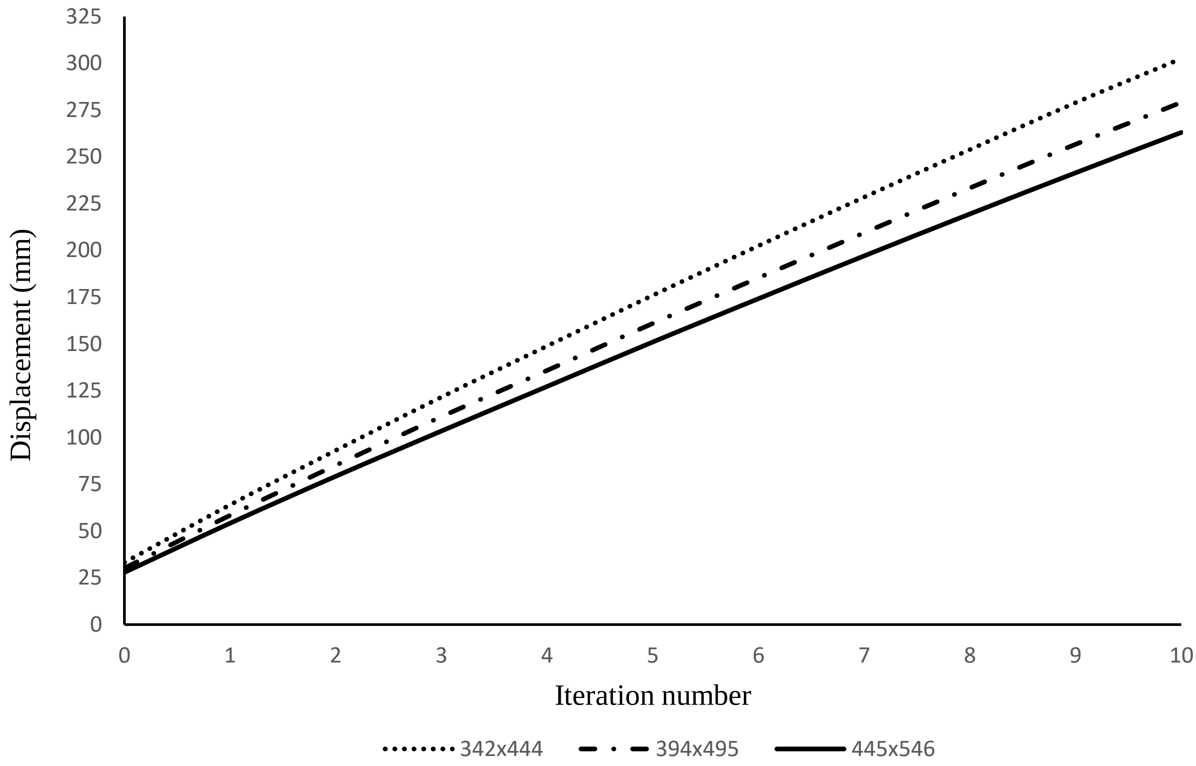


Figure 4-4: Graph depicting displacement v. iteration number for each column section

The graph shows that as the column size increases the displacement of the structure decreases, proving to be a more robust section compared to the smaller rectangular sections. The behaviour of sequential failures highlights the unpredictability of progressive collapse. Furthermore, the analysis proves to be sensitive to changes in input parameters, such as varying column sizes, resulting in distinct model outputs.

5 CONCLUSIONS AND RECOMMENDATIONS

5.1 CONCLUSIONS

The analysis of timber structures is a complex task that requires careful consideration of various parameters and assumptions to ensure accurate and comparable results. The input parameters used in design software play a crucial role in determining the behaviour of the structure, and minor changes can have significant implications. Consequence modelling is a crucial phase in evaluating risk for the assessment of structural robustness and possible robustness-improving measures (Janssens et al., 2012).

The study at hand sheds light on the behaviour of a timber office building subjected to an interior column failure. It is critical to recognise that the removal of members and surfaces that fail under Serviceability Limit State philosophy can have severe consequences in the form of human injuries and fatalities, economic loss, and environmental damage. The case-study emphasises the effect of column size on structural robustness through the examination of various aspects, including material properties and structural behaviour, to gain insight into the benefits and challenges associated with timber structures. Furthermore, the report highlights the significance of adhering to building codes and standards.

The 445x546 large column proves to be more robust compared to the medium and small column sections, as evidenced by lower displacements in all measurements. This shows that as the size of the column cross-section increases the structure is more robust. By opting for larger column sections in the design of timber structures, it can improve the overall stability and resistance against progressive collapse, but this is not definite as the relationship is not linear, and the sequence of propagating failures is unpredictable.

5.2 RECOMMENDATIONS

To better understand robustness in timber structures further research is warranted. One recommended avenue is to compare the behaviour of a timber structure to that of a reinforced concrete structure under similar loading conditions, and progressive collapse scenarios. Additionally, to study the amount of embodied carbon released during each progressive collapse scenario can provide valuable data for accessing the sustainability of timber construction

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APPENDIX A

Timber Material Properties	Symbol	Value	Unit
Strengths			
Characteristic strength for bending	$f_{m,k}$	18.000	N/mm^2
Characteristic strength for tension	$f_{t,0,k}$	11.000	N/mm^2
Characteristic strength for tension perpendicular	$f_{t,90,k}$	0.600	N/mm^2
Characteristic strength for compression	$f_{c,0,k}$	18.000	N/mm^2
Characteristic strength for compression perpendicular	$f_{c,90,k}$	4.800	N/mm^2
Characteristic strength for shear/torsion	$f_{v,k}$	3.500	N/mm^2
Rolling shear strength	$f_{R,k}$	1.200	N/mm^2
Moduli			
Modulus of elasticity parallel	$E_{0,mean}$	9500.0	N/mm^2
Modulus of elasticity perpendicular to grain	$E_{90,mean}$	630.0	N/mm^2
Shear modulus	G_{mean}	590.0	N/mm^2
Modulus of elasticity parallel	$E_{0,05}$	8000.0	N/mm^2
Modulus of elasticity perpendicular	$E_{90,05}$	422.1	N/mm^2
Shear modulus	G_{05}	496.9	N/mm^2
Densities			
Characteristic density	ρk	475.00	kg/m^3
Mean density	ρm	570.00	kg/m^3

Table A-1: Material properties of timber in numerical model

Section Properties 394x495	Symbol	Value	Unit
Bending			
Area moment of inertia about y-axis	I_y	398647.60	cm^4
Area moment of inertia about z-axis	I_z	251873.99	cm^4
Polar area moment of inertia	I_o	650521.59	cm^4
Radius of gyration about y-axis	r_y	143.0	mm
Radius of gyration about z-axis	r_z	113.7	mm
Polar radius of gyration	r_o	182.6	mm
Maximum statical moment of area about y-axis	max Q_y	12072.91	cm^3
Maximum statical moment of area about z-axis	max Q_z	9596.42	cm^3
Elastic section modulus about y-axis	S_y	16097.22	cm^3
Elastic section modulus about z-axis	S_z	12795.22	cm^3
Shear			
Shear area in y-direction	A_y	1625.00	cm^2
Shear area in z-direction	A_z	1625.00	cm^2
Torsion			
Torsional constant	J	519756.70	cm^4
Section modulus for torsion	S_t	17030.69	cm^3
Plasticity			
Plastic section modulus about y-axis	Z_y	24145.83	cm^3
Plastic section modulus about z-axis	Z_z	19192.84	cm^3
Plastic shape factor about y-axis	Z_y/S_y	1.500	--
Plastic shape factor about z-axis	Z_z/S_z	1.500	--

Table A-2: Section Properties of 394x495 column section

Section Properties 445x546	Symbol	Value	Unit
Bending			
Area moment of inertia about y-axis	I_y	603263.50	cm^4
Area moment of inertia about z-axis	I_z	399674.30	cm^4
Polar area moment of inertia	I_o	1002937.80	cm^4
Radius of gyration about y-axis	r_y	157.6	mm
Radius of gyration about z-axis	r_z	128.3	mm
Polar radius of gyration	r_o	203.3	mm
Maximum statical moment of area about y-axis	max Q_y	16570.14	cm^3
Maximum statical moment of area about z-axis	max Q_z	13487.32	cm^3
Elastic section modulus about y-axis	S_y	22093.52	cm^3
Elastic section modulus about z-axis	S_z	17983.10	cm^3
Shear			
Shear area in y-direction	A_y	2022.85	cm^2
Shear area in z-direction	A_z	2022.85	cm^2
Torsion			
Torsional constant	J	808886.49	cm^4
Section modulus for torsion	S_t	23805.42	cm^3
Plasticity			
Plastic section modulus about y-axis	Z_y	33140.28	cm^3
Plastic section modulus about z-axis	Z_z	26974.64	cm^3
Plastic shape factor about y-axis	Z_y/S_y	1.500	--
Plastic shape factor about z-axis	Z_z/S_z	1.500	--

Table A-3: Section properties of 445x546 column section

Section properties 342x444	Symbol	Value	Unit
Bending			
Area moment of inertia about y-axis	I_y	250958.28	cm^4
Area moment of inertia about z-axis	I_z	149345.79	cm^4
Polar area moment of inertia	I_o	400304.07	cm^4
Radius of gyration about y-axis	r_y	128.3	mm
Radius of gyration about z-axis	r_z	99.0	mm
Polar radius of gyration	r_o	162.1	mm
Maximum statical moment of area about y-axis	max Q_y	8468.78	cm^3
Maximum statical moment of area about z-axis	max Q_z	6533.06	cm^3
Elastic section modulus about y-axis	S_y	11291.71	cm^3
Elastic section modulus about z-axis	S_z	8710.75	cm^3
Shear			
Shear area in y-direction	A_y	1270.16	cm^2
Shear area in z-direction	A_z	1270.16	cm^2
Torsion			
Torsional constant	J	315623.16	cm^4
Section modulus for torsion	S_t	11672.48	cm^3
Plasticity			
Plastic section modulus about y-axis	Z_y	16937.57	cm^3
Plastic section modulus about z-axis	Z_z	13066.12	cm^3
Plastic shape factor about y-axis	Z_y/S_y	1.500	--
Plastic shape factor about z-axis	Z_z/S_z	1.500	--

Table A-4: Section properties of 342x444 column section

APPENDIX B

EVALUATION FORM

UNIVERSITY OF PRETORIA

DEPARTMENT OF CIVIL ENGINEERING

MARKING SHEET FOR UNDERGRADUATE RESEARCH PROJECT REPORTS

Student:			
Minimum requirements – the project report will be referred back if it does not meet the following requirements:			
The student identified, assessed, formulated and solved a complex problem (ECSA GA1)		The report is of a professional quality and appearance (ECSA GA6)	
The student applied fundamental knowledge to solve an engineering problem (ECSA GA2)		The student is aware of the impact of engineering activities on the environment and the community (ECSA GA7)	
The student solved the complex problem systematically (ECSA GA3)		The student worked independently to submit a unique research report (ECSA GA9)	
The student designed an experiment, interpreted and derived information from data (ECSA GA4)		Project is largely the students own work. Student should clearly indicate his/her own work. (ECSA GA10)	
The student used appropriate methods and computer technology to solve the problem (ECSA GA5)		The report is submitted by the specified date. (ECSA GA10)	

❑

❑ **Marks**					
❑ 30% Very Bad	❑ 45% Bad	❑ 55% Acceptable	❑ 65% Good	❑ 75% Distinction	❑ 90% Exceptional

Factors taken into account during evaluation:		**MAXIMUM MARKS**	**MARKS**
1	Problem definition		
2	Literature review (relevance, completeness, critical evaluation)		
3	Design of experiment		
4	Execution of experiment		
5	Presentation of results		
6	Analysis of results		
7	Evaluation of results		
8	Conclusions		
9	Recommendations		
10	Technical content		
11	Layout of report		
12	Style of writing		
13	Originality		
14	Level of difficulty		
15	Neatness of report		
16	General impression		
Final mark		100	

COMMENTS

Examiner: Date: