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ALTERNATIVE LOAD PATH ANALYSIS OF TIMBER POST-AND-BEAM MODULAR BUILDINGS

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ABSTRACT: Timber modular buildings are an emerging construction method, due to the environmental and construction speed benefits. However, the inherent discontinuity and limited deformation capacity, hinders their ability to effectively redistribute loads under accidental load cases and thus, their robustness. A method to quantify the robustness of a building is to assess its behavior under notional column removal scenarios. This study numerically investigates the behavior of a hypothetical five-storey timber post-and-beam modular building under accidental damage events represented by four different column removal scenarios. The findings indicate that the structure could develop sufficient alternative load paths to sustain the amplified accidental limit state design load in most cases, primarily through flexural mechanisms. However, due to the limited ductility of these mechanisms, modular connections were optimally redesigned to enhance axial elongation and capacity, enabling the development of catenary action. The most effective strategy for achieving a robust catenary response was the introduction of a fuse element, significantly improving the ductility of the connection and enhancing the overall structural robustness.

KEYWORDS: alternative load path analysis, timber, modular construction, robustness.

1 – INTRODUCTION

Timber modular buildings present an innovative solution to critical challenges faced by the construction industry, such as environmental impact and construction speed. However, with innovation comes uncertainty, and one of the main uncertainties of modular timber buildings is the robustness of the system under accidental load cases. A common method to quantify the robustness of buildings is to assess their ability to develop alternative load paths to halt damage propagation or collapse under notional element removal scenarios [1]. Given the lack of information available in current guidelines regarding robustness assessment, defining the critical notional removal scenarios, is often left to engineering judgment. This presents a challenge for modular construction, as the critical elements might change significantly based on the project detailing. Given the inherent discontinuity, modular systems rely on inter-modular connections to develop alternative load paths [2]. Load redistribution between the modules can be achieved either by providing sufficient rotational stiffness in the inter-modular connections to simulate a monolithic system, or by ensuring sufficient rotational capacity to enable the development of horizontal ties. Either can present a challenge in the context of timber connections, due to local stress development and brittle failure mechanisms [3]. For this project, three notional column removal damage events were simulated for a five-storey hypothetical building made up of post-and-beam timber modules, implementing a concept developed by firm *Lister Buildings* (NL) and structurally designed by the engineering firm *Pieters Bouwtechniek* (NL). The modular concept was originally designed without assessing alternative load redistribution under notional column removal scenarios. Considerations of load

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redistribution where performed by engineering judgement. Therefore, the objective of this study was to assess the behavior and inherent robustness of the original design and optimally redesign the inter-module connection to allocate load redistribution mechanisms for the most critical identified column removal scenario.

2 – BACKGROUND

2.1 ALTERNATIVE LOAD PATH ANALYSIS

A common approach adopted by design guidelines to quantify and enhance the robustness of buildings, is to assess their ability to develop alternative load paths when subjected to notional element removal scenarios [4]. This approach assumes that a structural element fails instantaneously, requiring the surrounding elements to redistribute the resulting loads.

When a structural element fails suddenly, internal forces are lost instantaneously, generating dynamic effects in the remaining structure. To account for these effects in static analysis, a Dynamic Amplification Factor (DAF) is applied. This factor adjusts the gravity loads to approximate the equivalent static displacement of the system. Conventionally, the DAF is defined as the ratio of the maximum dynamic displacement Δ_{dyn} to the static displacement Δ_{stat} for a Single Degree of Freedom (SDOF) elastic system under the same loading conditions [5]. However, for an inelastic SDOF system, the DAF can also be expressed in force terms as presented in (1) [6].

$$DAF = \frac{P_{dyn}}{P_{stat}} \tag{1}$$

For an idealized linear elastic SDOF system, the instantaneous application of a load results in a DAF of 2.0. However, for inelastic systems, the factor varies depending on the load-displacement response. Structures with sufficient ductility can dissipate energy through plastic straining, leading to a lower DAF [6], thus values lower than 2.0 can be found in design guidelines prescribed for specific structural systems. Despite this, most guidelines in the context of timber structures, refer to a DAF of 2.0 as a conservative assumption [7, 8, 9].

A key measure to assess structural robustness is the Load Factor (LF), which evaluates the capacity of the structure to sustain alternative load paths. The LF is defined as the ratio between the maximum static load the damaged system can sustain and the design load, where the design load includes both permanent and variable loads, considering accidental load partial factors. Given the assumptions regarding the accidental actions acting on the system, a specific DAF may be applied to the design load load, to account for the loading rate of application.

2.2 CATENARY SYSTEM

A robust catenary system forms when equilibrium is achieved. For a two-floor-spanning axially restrained system, as the one shown in Figure 1, equilibrium under the applied load (*F*) is maintained through the development of a tensile catenary force (F_{cat}). This force depends on the beam span (L_1) and the elongation of the member (Δl), as given in (2).

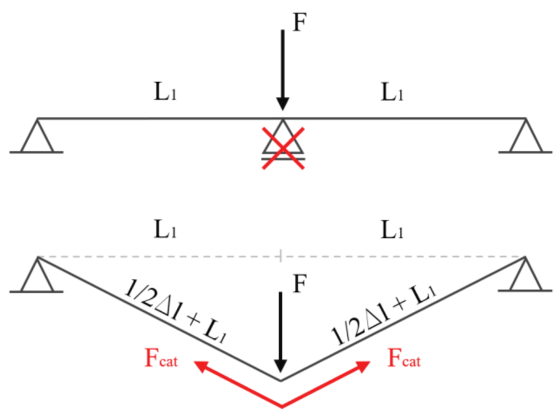

Figure 1. Catenary system over two floor spans

$$F_{cat} = \frac{F}{2\sqrt{1-\left(\frac{L_1}{\frac{1}{2}\Delta l + L_1}\right)^2}} \tag{2}$$

The catenary equation, defines the minimum required tensile resistance of all elements within the system to sustain a given elongation. If the maximum axial resistance of the system components exceeds the required catenary force, a robust catenary can develop. Consequently, for a system with a known maximum tensile resistance or elongation capacity, a theoretical Catenary Requirement Boundary (CRB) can established. The CRB represents the system's resistance requirements for a given applied load.

3 – METHODOLOGY

In this study, alternative load path analysis was conducted for different damage events. The analyses follow a nonlinear static procedure, where structural elements are removed quasi-statically and the structure is subjected to unamplified accidental limit state design

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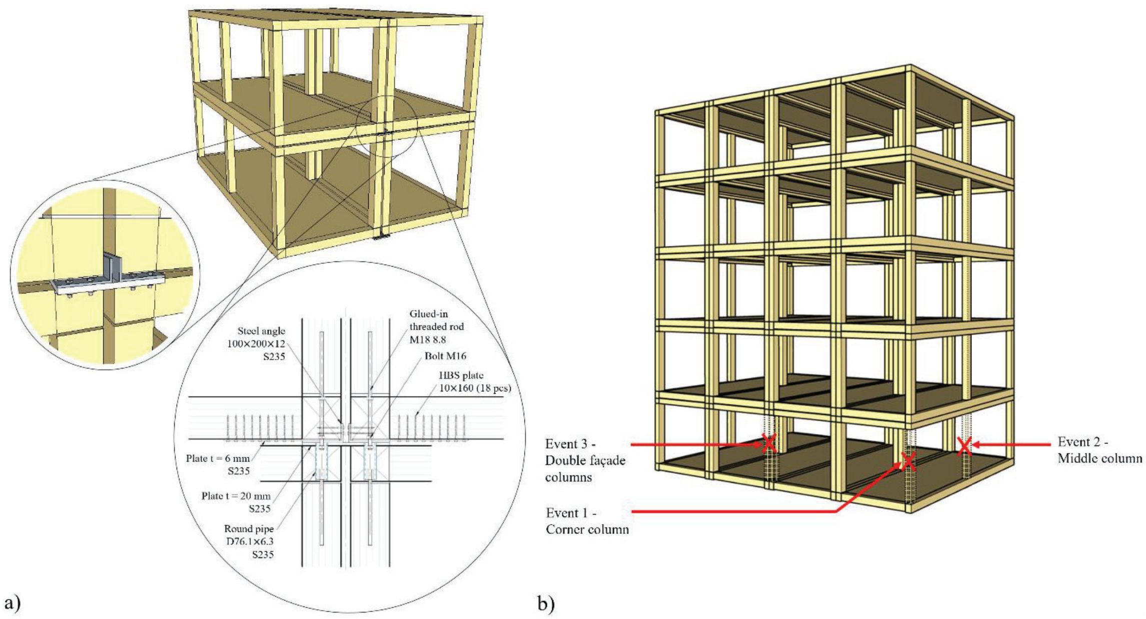

Figure 2. Case study project description, a) module and inter-modular connection design, b) column removal scenenarios.

loads. Furthermore, the parts of the structure surrounding the damaged elements are loaded with additional gravity loads representing the dynamic amplification of the loads, with an assumed DAF of 2.0.

This procedure is repeated for each damage event, assessing the ultimate capacity, rotation requirements and optimal load redistributing mechanisms. The ultimate capacity of the system is defined by the implementation of ductility limits for the connections, which once reached decrease their stiffness properties to zero, creating instability and terminating the analysis. The critical scenario is then identified as the one exhibiting the lowest load factor (LF) or the highest redistribution demand. For this critical scenario, the modular connections are re-designed to enhance the load redistribution, ensuring that the structural system can effectively accommodate applied loads and prevent progressive failure.

The structural analyses were carried out in the software Abaqus. The structural elements were modelled using one-dimensional B21 beam elements with a uniform cross-section and linear elastic material properties. The mesh employed an element size of approximately 1/6th of the bay size. The modelling approach incorporated concentrated plasticity at the connections. CONN2D2 connectors were used for both inter- and intra-modular connections, with specific behaviors assigned to the relative motion along each degree of freedom (DoF) to capture elastic, plastic and ultimate behaviors. The DoFs were assumed to be independent. No interaction was considered.

4 – PROJECT DESCRIPTION

This case study examines an innovative volumetric post-and-beam timber module developed by Lister Buildings in the Netherlands. The structural design of the modular system complies with ultimate limit state (ULS) and serviceability limit state (SLS) requirements outlined in Eurocode and Dutch standard. The structure consists of multiple modules stacked vertically and horizontally. The longer sides of the modules are positioned adjacent to each other, while in practice, the modules are accessed via the short side. Building stability is ensured through the use of steel trusses, which provide lateral and longitudinal support. For the scope of this investigation, only the section between the stability elements is considered.This section consists of four modules placed side by side, with four modules stacked on top, forming a 4 x 5 grid (see Figure 2).

The modules are constructed using glue-laminated timber (GLT) columns and beams, as well as cross-laminated timber (CLT) floor and ceiling slabs. Each modules contains six columns – four positioned at the corners and two located at midspan. The dimensions for the structural elements are provided in Table 1.

Table 1. Dimensions structural elements

Module (L × W × H)	9.4 × 3.0 × 3.2	m
Floor beam (h × w)	320 × 240	mm
Ceiling beam (h × w)	220 × 240	mm
Columns (h × w)	320 × 240	mm
Floor and ceiling slab (t)	160	mm

The intra-modular connection, namely the connection between beams and columns in the module, is achieved through glued-in threaded rods embedded in the column members, which are connected to steel plates screwed into the beam members (Figure 2a). The inter-modular connection, namely the connection between the modules, consists of a thick steel plate bolted to the intra modular connections. The yield resistance of this connection, when subjected to horizontal tie action, is governed by the weakest component – the tensile resistance of the plate net cross-section, as detailed in Table 2.

Table 2. Yield resistance of components in inter-module connection

Shear resistance threaded rod M18	147.4	kN
Bearing resistance tie plate with threaded rod	155.5	kN
Shear resistance of M16 bolt with 12 mm angle plate	120.6	kN
Bearing resistance 12 mm angle plate	153.6	kN
Tensile resistance of plate net cross-section	108.9	kN
Shear resistance 10 mm screw group	110.2	kN

The stiffness properties of the connection are determined using a component-based model, in which various elements in the assembly are represented as individual extension springs. The overall behavior of the connection is modelled through the serial summation of these individual components. Table 3 presents the translational stiffness properties defined for the inter-modular connection in cases where the horizontal tie force develops.

Table 3. Translation stiffness of inter-module connection components

Shear stiffness screw group	1.25E+05	kN/m
Axial stiffness 6 mm plate	1.53E+05	kN/m
Axial stiffness 30 mm plate	6.30E+06	kN/m
Elastic stiffness of connection	3.25E+04	kN/m

Ductility limits were defined for the connections in order to ensure analysis termination upon reaching these limits and to determine the ultimate capacity of the system. For the intra-modular connection, a maximum rotation of 0.15 radians was defined based on the experimental study conducted by Reçoubas on ductility and moment resistance of timber connections with glued-in rods [10]. For the inter-modular connection, an axial elongation ductility limit was defined, based on the predicted behavior of the steel plate.

In this study, three different notional column removal scenarios were defined to assess the ability of the structure to develop alternative load paths. Figure 2b illustrates the three damage events.

- Event 1: removal of a corner column from a corner module.
- Event 2: removal of the middle column at midspan along the long side of a corner module.
- Event 3: removal of two columns along the façade (i.e., the short side of the module). This scenario accounts for the potential loss of a corner column in adjacent modules, as their proximity may increase the likelihood of simultaneous failure.

5 – RESULTS

5.1 ALTERNATIVE LOAD PATH ANALYSIS

Figure 3 presents the load-displacement response of the structure when subjected to the three different damage scenarios, along with a visualization of the strains in the system at the ultimate load. The applied load is presented as a Load Factor (LF), which represents the ratio between

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the applied load and the accidental limit state design load. A LF of 1.0 indicates that the system can carry the full unamplified design load, while a LF of 2.0 signifies that the system can withstand the dynamic load when assuming a DAF of 2.0.

Figure 3a, shows the load-displacement behavior of the system when subjected to Damage Event 1, measured at the location of the corner column. The load-displacement curve shows a linear increase up to a vertical displacement of 0.3 meters. At this stage, analysis of the moment-displacement behavior of the intra-modular connection shows that the moment resistance of the connection has been reached, leading to yielding in the connection. With increasing plastic deformation, the structure continues to deform until reaching a vertical displacement of 0.42 meters, at which point the rotational ductility limit of 0.15 radians in the connection is exceeded. Once this limit is reached, the stiffness of the connection decreases to zero, resulting in the termination of the analysis. The results show that the system reached a LF of approximately 0.37, indicating that the system was unable to develop an alternative load path to carry the design load.

Figure 3b illustrates the load-displacement behavior of the system when subjected to Damage Event 2, measured at the location of the middle column. The load displacement curve shows that the structure exhibits linear behavior up to the ultimate deformation of 0.14 meters. Since no connection failure was observed, LF of 2.0 was achieved, indicating that the structure is capable of developing sufficient load distributing mechanisms to carry the amplified design load. However, a bending stress of 30.2 N/mm^2 develops at the beam midspan, exceeding the bending resistance of GL24h in the accidental limit state, which is taken as 26.4 N/mm^2 (assuming a k_{mod} = 1.1 for accidental actions). This suggests that a beam failure would occur at a LF of approximately 1.75.

Figure 3c illustrates the load-displacement behavior of the system when subjected to Damage Event 3, measured at the location of the double façade column removal. The load-displacement curve shows that the structure behaves linearly until it reaches a vertical displacement of 0.025 meters. Beyond this point, stiffening behavior is observed, attributed to the activation of catenary action in the system. At a vertical displacement of 0.165 meters, the structure exhibits a reduction in stiffness, followed by a linear response leading to system failure. At this stage, an analysis of the axial force development in the inter-modular connection indicates that the axial resistance of the connection has been reached, resulting in yielding of the connection. As yielding progresses, the connection undergoes plastic deformation until it reaches the

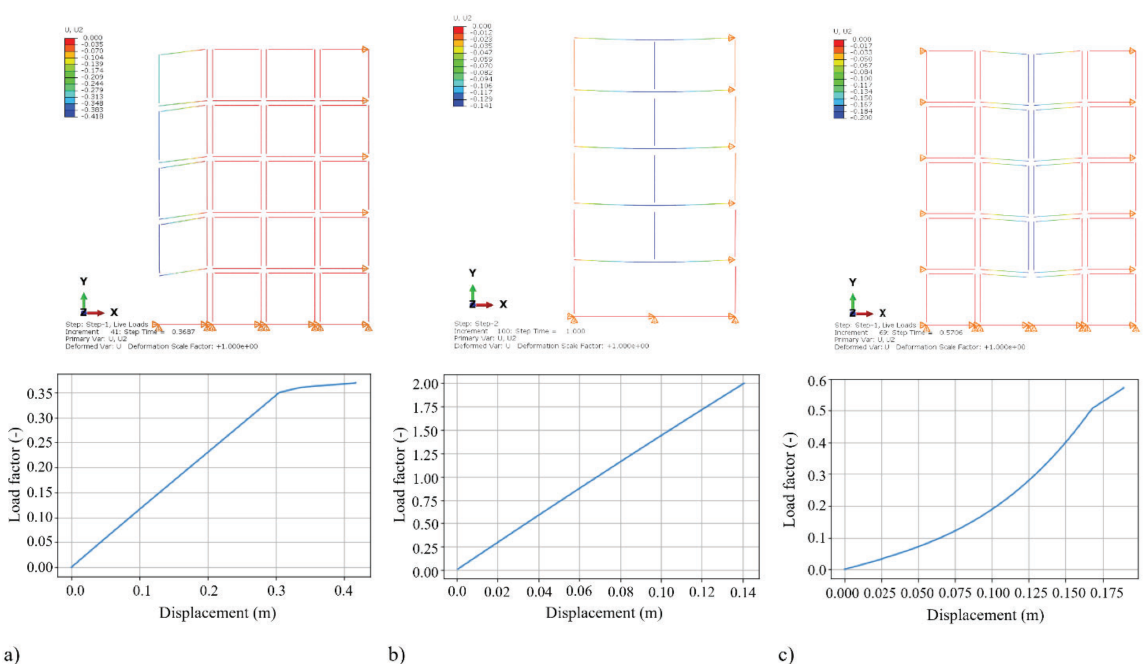

Figure 3. Load-displacement behavior and visualization of the system damaged state when subjected to a) Damage Event 1, b) Damage Event 2, and c) Damage Event 3.

ultimate vertical displacement of 0.2 meters, at which point the ductility limit of the connection in axial elongation is exceeded, leading to the analysis being terminated. The results show that the system reached a LF of 0.57 indicating that the system was unable to develop an alternative load path to carry the design load.

In this study, the 3D structure is assessed as a 2D frame, thus the resistance of mechanisms along the long side of the modules is neglected. For Damage Events 1 and 3, where the system is not able to carry the design load in the damaged state, the behavior along the long side of the module is investigated. A hypothetical Damage Event was defined to simulate the response of the structure in the opposite direction.

Damage Event 4, represents a corner column removal, simulating the behavior of the system along the long side of the module for both Damage Events 1 and 3. The structure is loaded following the same procedure as for the other Damage Events. The design load taken for the LF, is defined for a corner module, where the tributary width of the beam is half the module width along the short side. Figure 4 presents the load-displacement behavior of the system subjected to the hypothetical Damage Event at the location of the corner column. The load-displacement curve indicates that the structure behaves linearly until the ultimate vertical displacement of 0.175 meters. A minor stiffness reduction at a vertical displacement of 0.085 meters is observed, which corresponds to a LF of 1.0. This behavior can be attributed to the method of load application, where the additional loads are applied only to the damaged area. No connection failure was observed, allowing the system to reach a LF of 2.0. At the ultimate load level, a bending stress of 26.2 N/mm^2 develops at midspan above the middle column, which remains below the design bending resistance of GL24h at accidental limit state.

Figure 4, shows that the system is capable of developing sufficient alternative load paths to carry the amplified design load. Given that Damage Event 4 represents the structural behavior along the long side of the module during Damage Events 1 and 3, it is concluded that sufficient load redistribution could develop in a system level for these scenarios. However, Damage Event 4 relies primarily on flexural mechanisms, which are not ideal to ensure robustness. Exceeding the flexural capacity could result in brittle failure, compromising structural integrity. Therefore, Damage Event 3 is selected for optimizing the design of the modular connections, ensuring that robust alternative load paths develop through catenary action.

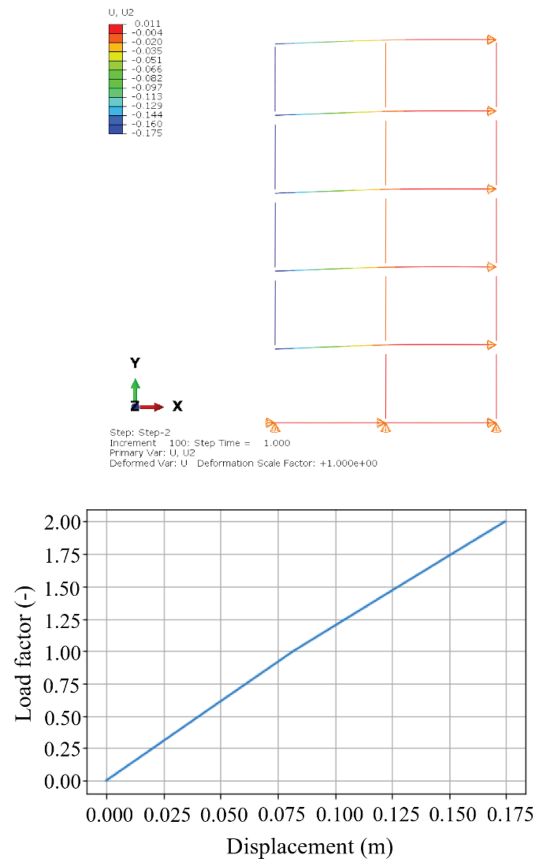

Figure 4. Load-displacement behavior and visualization of damaged state when subjected to hypothetical Damage Event 4.

5.2 CONNECTION OPTIMIZATION

As shown in Figure 3c, when the structural system is subjected to Damage Event 3, the primary load redistributing mechanism is the activation of catenary action. However, the current connection design lacks sufficient elongation capacity to carry the design load. The goal of the optimization is to redesign the inter- and intra-modular connection, to achieve a force-elongation response which meets the CRB of the system (see section 2.2). The optimization can be achieved by either increasing the resistance capacity of the connection, or enhancing its ductility. In this study, both methods are explored, resulting in two optimal connection designs.

Method 1 – High strength connection

Method 1 retains the original connection design, but increases its resistance capacity by increasing the cross-section of the steel plates, the strength of the steel, the dimension and quality of the bolts and screws. This approach leads to an increase in stiffness of the connection, influencing the load redistribution in the

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system. The optimization process is iterative, where (i) the stiffness properties of new connection design are determined, followed by (ii) a structural analysis of the updated system, and (iii) the assessment of the strength requirements in the connection.

The optimized high strength connection design is shown in Figure 5a. Apart from the adjustments in dimension and quantities in the different component, the steel strength of both the tie plate and angle plate is increased from S235 to S355. These modifications increase the tensile resistance of the connection to 264.6 kN, which is still governed by the tensile resistance of the net cross-section of the tie plate. The elastic deformation of the tie plate and screw group in the connection is found to be 1.9 mm, with no plastic deformation assumed. The tensile properties of the high-strength connection are summarized in Table 4.

Method 2 – Ductile connection

Method 2, aims to redesign the connection configuration to allow for sufficient elongation in the catenary mechanism. To determine the required elongation at a given applied load, the catenary equation presented in (2), can be rewritten as shown in (3).

$$\Delta l = -2 \cdot L_1 + \frac{2 \cdot L_1}{\sqrt{1 - \left(\frac{F}{F_{cat}}\right)^2}} \tag{3}$$

Table 4. Yield resistance of components in high strength inter-module connection design.

Shear resistance threaded rod M22	303.0	kN
Bearing resistance tie plate with threaded rod	431.2	kN
Shear resistance of M16 bolt with angle plate	282.6	kN
Bearing resistance 12 mm angle plate	601.4	kN
Net tensile resistance of tie plate	264.6	kN
Yield shear resistance screw group	265.0	kN
Ultimate shear resistance screw group	339.3	kN

The vertically applied force is defined as the accidental limit state load acting at the location of the double façade column removal, amplified with a DAF of 2.0. According to (3), the required elongation at a tensile load in the catenary of 108.9 kN (maximum tensile capacity of the original connection) is 425 mm, which must be accommodated by a single fuse element. Considering an ultimate strain rate of 20% for S235 steel, a fuse length of 2125 mm is needed, making the connection excessively large. Therefore, in addition to incorporating the fuse elements, the resistance capacity of the connection is also increased for reducing their required elongation, following a similar approach to Method 1.

Table 5. Resistance of components in ductile inter-module connection design.

Shear resistance threaded rod M22	303.0	kN
Bearing resistance tie plate with threaded rod	316.8	kN
Shear resistance M18 bolts with angle plate	276.5	kN
Bearing resistance 12 mm angle plate	308.6	kN
Net tensile resistance tie plate at the screws	272.2	kN
Yield shear resistance screw group	204.3	kN
Ultimate shear resistance screw group	261.6	kN
Yield tensile resistance fuse tie plate	169.2	kN
Ultimate tensile resistance fuse tie plate	259.2	kN

The connection design is based on the same three components, the thin tie plate, thick coupling plate and the angle plate. These components are connected with the same configuration as the original and high-strength connections. However, the ductile connection design, introduces a fuse element at the tie plate to enable controlled elongation. The optimized ductile connection design is shown in Figure 5b, while its tensile properties

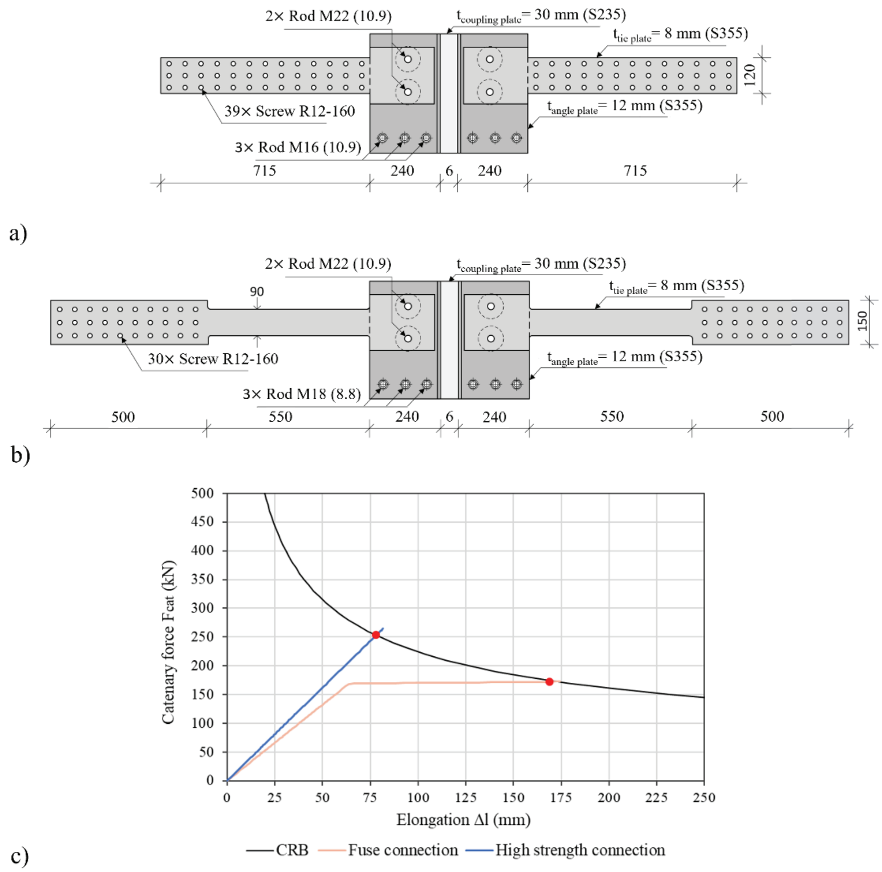

Figure 5. Optimized inter-modular connection design based on a)Method 1: increasing axial resistance and, b) Method 2: increasing ductility.

are presented in Table 5. This design achieves a yield tensile resistance of 169.2 kN, governed by the tensile resistance of the fuse tie plate, with an ultimate resistance of 259.2 kN. The elastic deformation of the connection was calculated as 2.05 mm, while the plastic deformation of the fuse is 110 mm.

Figure 5c presents the catenary force-elongation behavior of both the high-strength and ductile connections, along with the point where they intersect the CRB, which is defined as the optimal design. Additionally, Figure 6, shows the load-displacement behavior of the system subjected to Damage Event 3, when implementing both optimal connection designs.

High strength connection:

The load-displacement response of the system with the high strength connection demonstrates the development of a robust catenary mechanism. The LF exceeding 2.0 indicates that the system can sustain the amplified design load.

Ductile connection:

The load-displacement response of the system with the ductile connection shows a steady catenary development up to a vertical displacement of approximately 0.4 meters, corresponding to a LF of 1.2. Beyond this point, the connection reaches its yield capacity, and axial elongation is governed by plastic deformation in the fuse element. At the ultimate displacement, the system reaches a LF of 2.0, showing its ability to sustain the amplified design load.

From Figure 6 it can be concluded that both connection designs effectively redistribute the amplified vertical design loads. However, in order to determine which connection performs best, multiple criteria can be considered. Many design codes and guidelines favor ductile connections, over those with brittle failure modes, as they introduce additional redundancy into the system. The presence of visible deformations serves as an early warning mechanism, making the ductile connection a more favorable choice. Additionally, when comparing material efficiency, the ductile connection requires 4.9 kg

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less steel than the high strength connection, making it a more resource efficient solution. Finally, while both connections were designed to sustain amplified loads with DAF of 2.0, Figure 6 shows significant differences between the system load-displacement behavior. These differences may influence dynamic amplification effects on the surrounding structural elements. Specifically, the energy dissipation through plastic deformation in the ductile connection could potentially reduce the dynamic loads acting on the system. This suggests that a lower DAF could be used in design, further reducing material demands for the connection.

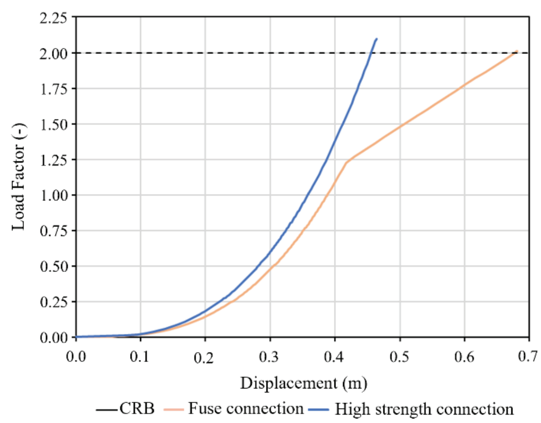

Figure 6. Load-displacement behavior of system subjected to Damage Event 3, for the optimized high strength and fuse connections.

6 – CONCLUSION

This study has investigated the ability of modular post-and-beam timber buildings to develop alternative load paths under different notional column removal scenarios. Through nonlinear static analysis, the structural response was evaluated under four damage events, identifying failure mechanisms, ultimate capacity, and critical structural behaviors.

The results demonstrated that the system was able to develop sufficient load redistribution mechanisms when subjected to Damage Event 2, which simulates the removal of a middle column along the long side of the modules. A LF of 2.0 was achieved for this scenarios, with no connection failures. However, for Damage Events 1 and 3, which represented a corner and double façade column removals, respectively, the system was unable to sustain the design loads, with LF of 0.37 and 0.57 respectively.

Since the 3D structure was simplified into a 2D frame, potential load redistribution mechanisms along the long side of the modules were neglected for these scenarios. To address this limitation, an additional Damage Event 4 was introduced to simulate the structural behavior along the long side of the module. The results confirmed that sufficient load redistribution could be achieved through beam flexure. However, since flexural mechanisms are not ideal for robustness design, due to the brittle failure mode, the design of the inter- and intra-modular connection was optimized to enable catenary action in Damage Event 3.

The connection optimization was conducted using two different methods: one prioritizing connection strength, and the other focusing on system ductility. The high strength connection increased the axial capacity by modifying cross-section of the steel plates, the steel grade and fastening components, which improved the overall stiffness and enabled a robust catenary response. On the other hand, the ductile connection introduced a fuse element to enhance elongation capacity, allowing plastic deformation while maintaining structural integrity.

Both optimized connection designs successfully redistributed the amplified vertical loads and achieved a LF of 2.0 when the system was subjected to Damage Event 3. However, a qualitative comparative analysis suggests that the ductile connection offers significant advantages in terms of redundancy, material efficiency and energy dissipation, which could potentially reduce the required DAF and lead to further material savings in design. Overall, this study highlights the critical role of connection design in ensuring modular timber structures can effectively resist localized failures.

7 – ACKNOWLEDGEMENT

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