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ROBUSTNESS ANALYSIS OF INNOVATIVE CONNECTIONS FOR POST-AND-BEAM MASS TIMBER BUILDINGS

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ABSTRACT: The design of structural robustness and the prevention of disproportionate collapses in mass timber buildings are becoming relevant considerations due to the increasing height of these buildings. Despite the number of multistory timber buildings has rapidly increased during the last decade, current codes lack specific guidance on mitigating disproportionate collapse. The paper, supported by ROTHO BLASS SRL., focuses on characterizing innovative beam-to-column connections with slotted holes to facilitate structural assembling, ensure hinged structural and catenary behavior under large deformations, particularly in scenarios involving column removal. The research unfolds in three stages: 1) Experimental characterization of the connection's constitutive laws; 2) Parametric study varying beam and slotted hole lengths referring to a simplified 2D post-and-beam subassembly virtual test; 3) Robustness analyses of a 3D post-and-beam archetype building under column removal scenarios using Alternative Load Path Linear and NonLinear Analyses. Results indicate that slotted holes, proposed in the innovative beam-to-column connections, enhance the structural robustness facilitating effective activation of Alternative Load Paths such as catenary effects. Finally, structural robustness of a 4-storey post-and-beam building assembled using the proposed innovative beam-to-column connection was numerically analyzed considering Alternative Load Path Linear and NonLinear analysis scenarios.

KEYWORDS: robustness, beam-to-column connectors, disproportionate collapse, alternative load path, post-and-beam

1 – INTRODUCTION

In recent years, there has been a global increase in the construction of tall post-and-beam timber buildings, driven by a growing focus on sustainability and resource efficiency. The literature about robustness is comprehensive concerning concrete and steel buildings but is rather limited regarding timber [1]. For post-and-beam timber buildings, guidance regarding robustness is scarce, but in some aspects they seem to be like steel frames and precast concrete: the beam-to-column connections are the key aspects [2]. Usually, metal connectors may provide the required joint ductility although standard beam-to-column connections are characterized by limited rotational capacity, resulting in reduced load redistribution and catenary effect in scenarios involving column removal [3]. In this work, an innovative beam-to-column connection for post-and-beam structure is experimentally and numerically investigated to define the rotational capacity and therefore the activation of catenary effects in scenarios involving column removal.

2 – CONNECTION DESCRIPTION AND CHARACTERIZATION

The robustness of post-and-beam structures primarily depends on the tensile strength of the beam to column connection, which enables the activation of the catenary effect. However, this mechanism can only occur if the connection has high rotational capacity. Manufacturers of connectors for timber structures are moving towards producing connectors that can ensure increasingly greater rotations and high tensile capacities. For this purpose, the connection studied is the 'ALUMEGA' model 240HV/JV with VGS9x180 screws by ROTHO BLAAS SRL (Fig. 1a) according to ETA-23/0824 [4]. The ALUMEGA connectors are two-piece connector system for use in

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making end-grain to side-grain connection in load bearing timber structures. Horizontally slotted holes with an axial tolerance of up to 8 mm ($\delta_{free} = \pm 4$ mm) allow free rotation of the connection ($\alpha_{free} = 2.545°$), facilitating structural on-site assembly and achieving a pure hinge connection. These rotations can be derived as a function of the distance between the outermost holes and the displacement allowed by the slot, termed δ_{free}, at the outermost bolt (Fig. 1b).

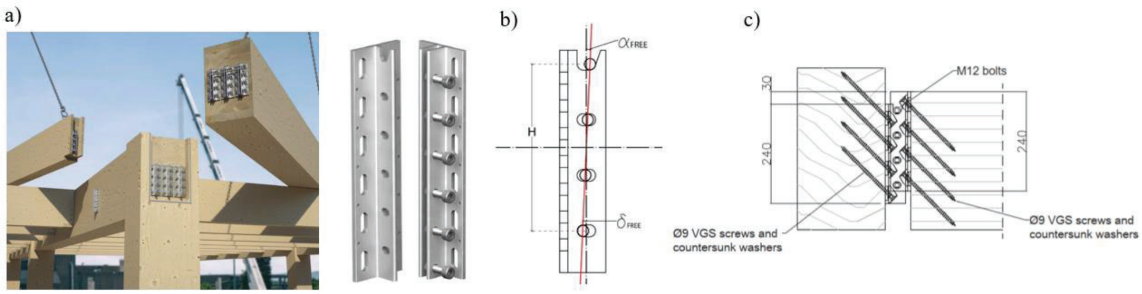

Figure 1. a) ALUMEGA connector; b) ALUMEGA free rotation scheme; c) ALUMEGA beam to beam connection

To characterize the behavior of this connection regarding structural robustness, constitutive relationships for axial, shear, and bending stress were extrapolated from the results of destructive experimental tests. The behavior of the connection under tensile and shear actions was experimentally characterized at the Karlsruhe Institute of Technology (KIT) - Test report no. 236106 [5] (Fig. 1c). Figures 2a), 2b) and 2c) report the axial and shear constitutive law obtained linearizing the experimental load-displacement curves and the bending constitutive load obtained based on axial tests and formulation hypothesis.

The presence of slotted holes, and consequently a translational gap δ_{free}, allows for constitutive relationships for normal stress and bending moment to be offset by values equal to δ_{free} and α_{free}, respectively. This means that, until the gap is closed, the connection will not activate the axial and bending components but will function simply as a hinge. Ultimately, the presence of these characteristics enables an additional rotational capacity due to geometric properties.

3 – A VIRTUAL EXPERIMENTAL SETUP FOR DAMAGE SCENARIO SIMULATIONS

3.1 VIRTUAL SETUP DESCRIPTION

With the aim of studying the behavior of two-dimensional frames used in solid timber buildings in a scenario of central column removal, a virtual experimental setup was developed [6]. The virtual setup is schematized in Figure 3 and consists of a two-bay frame with a central column removed to simulate the loss of an internal column. Horizontally, to simulate the constraints provided by adjacent bays of the building, each lateral column was connected to a restraint wall through two load cells per side (from LC1 to LC4). On the ground, the lateral columns rest on a Teflon layer via a steel plate attached to their bases to simulate a roller support. To prevent in-plane and out-of-plane rotations of the central column, a metal formwork lined with Teflon internally was devised to minimize friction forces.

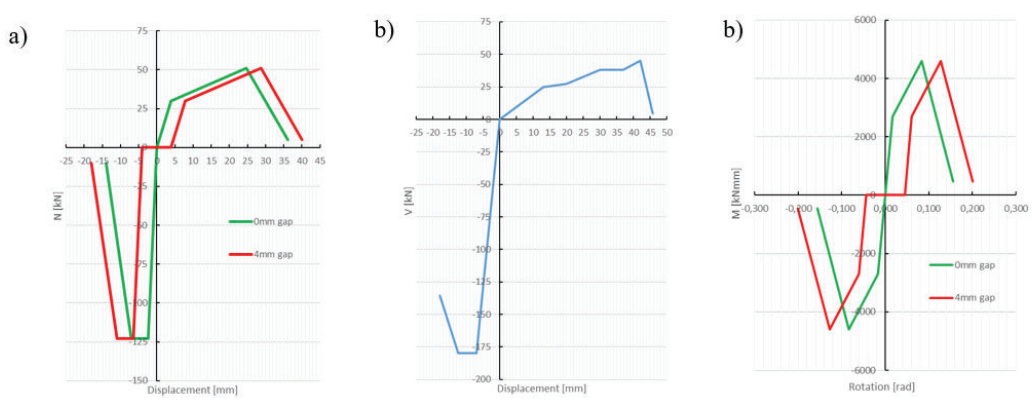

Figure 2. a) Axial multilinear constitutive law; b) Shear multinlinear constitutive law; c) Bending multilinear constitutive law

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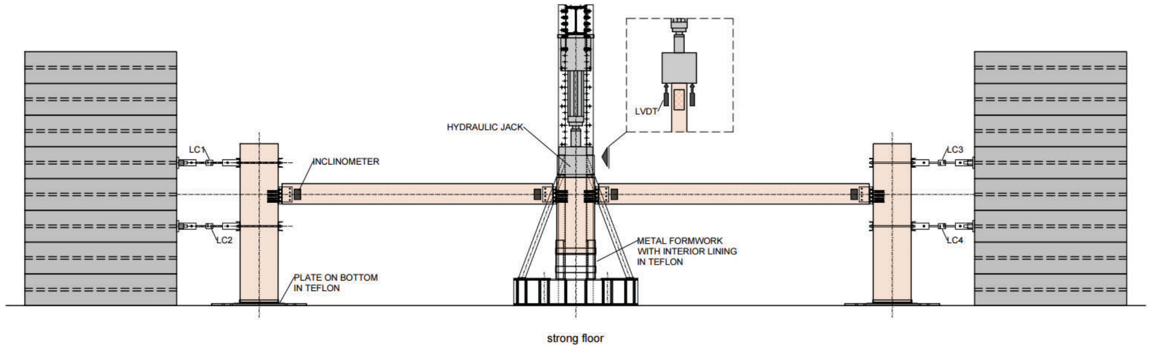

Figure 3. 2D post-and-beam subassembly middle column removal test setup

To simulate column removal, a hydraulic jack, anchored to a restraining frame, is operated to apply a quasi-static load, thus conducting a displacement-controlled test. Finally, the load cells on each lateral column allow for the measurement of axial and bending forces related to beams and connections.

3.2 PARAMETRICAL ANALYSIS

To analyze the structural response of a building to the notional removal of an element, an Alternative Load Path analysis (ALPAs) may be performed [7]. In order to conduct parametric analyses to obtain predictive results for future experimental tests, an analytical characterization of the setup described in the previous section was performed. Figure 4a) shows the static scheme of test setup.

The following basic assumptions are listed:

- The deformability of columns and beams is neglected.
- Perfect constraints are considered.
- Geometric non-linearities are considered.
- The decoupling of the equivalent constitutive laws of the connection is considered.
- The non-linear behavior of the connections (material non-linearity) is considered, modeled with the constitutive laws shown in Figures 2a), 2b), 2c).

Performing the structural analysis in deformed configuration (Fig. 4b) results in the following analytical formulation of the setup:

$$F = 2N\, sen(\alpha) + \frac{4M}{L}\, cos^2(\alpha) \tag{1}$$

$$N = \frac{1}{cos(\alpha)} \cdot \left[R_1 + R_2 + \frac{2d}{L}(R_1 - R_2)\, sen\,(\alpha)\, cos\,(\alpha)\right] \tag{2}$$

$$M = (R_1 - R_2)\, d \tag{3}$$

$$R_1 = \frac{N}{2} cos(\alpha) - \frac{V}{2} sen(\alpha) + \frac{M}{2d} \tag{4}$$

$$R_2 = \frac{N}{2} cos(\alpha) - \frac{V}{2} sen(\alpha) - \frac{M}{2d} \tag{5}$$

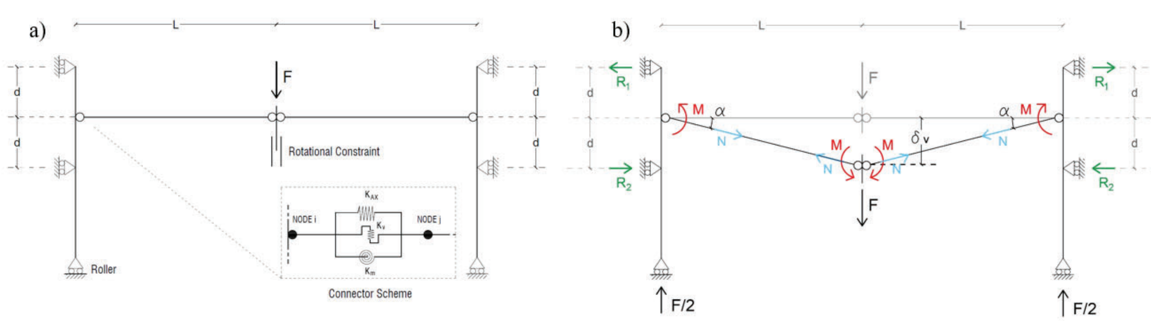

Figure 4. a) 2D post-and-beam subassembly middle column removal static scheme; b) 2D subassembly in deformed configuration

It can be observed that the formulations depend on the constitutive laws of the connection, the length of the beams, and the connection gap δ_{free}. In particular, the axial stress acting on beams and connections depends on

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the angle of rotation α. As the angle and thus the deformation of the setup increase, tensile stresses and the catenary effects also increase.

To study how these variables influence the structural response, several parametric analyses were performed using the analytical formulation of the experimental setup. Table 1 presents the analyses conducted by varying the lengths of the beams and the values of δ_{free} of the connection.

Table 1: List of parametric analysis

Parametrical Analysis		
Analysis identifier	***L [mm]***	***δfree [mm]***
T30_0	3000	0
T30_4	3000	4
T30_8	3000	8
T45_0	4500	0
T45_4	4500	4
T45_8	4500	8
T60_0	6000	0
T60_4	6000	4
T60_8	6000	8

3.3 PARAMETRICAL ANALYSIS RESULTS

Figure 5 shows the results of parametric analyses. The increase in the size of the slotted holes in the connection and thus in the translational gap δ_{free} and the free rotation α_{fre} allows for an enhancement in the capacity of the beam-connection system to withstand vertical loads resulting from column removals, both in terms of resistance (see Table 2) and ductility of the system.

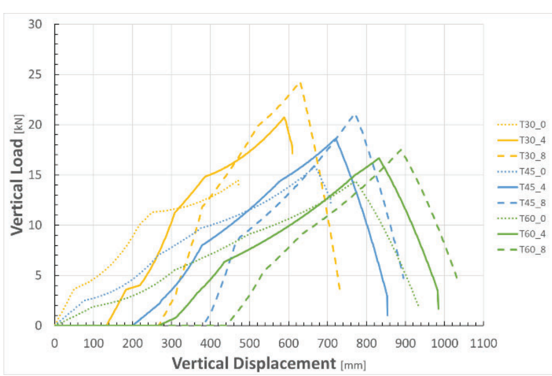

Figure 5. Force - vertical displacement curves.

Indeed, δ_{free} can be viewed as an addition of rotational capacity to the connection that does not originate from mechanical properties but from geometric ones. This facilitates a greater activation of alternate load paths, such as the catenary effect. The increase in the length of the beams leads to a progressive reduction in the utilization of the axial component in favor of a greater flexural contribution, resulting in a decrease in resistance to vertical loads arising from column removals (see Table 3).

Table 2: Comparison of vertical force as the gap changes

	T30_0	***T30_4***	***T30_8***
Fmax [kN]	18,29	20,73	24,30
Δ F [%]	-	13%	33%
	T45_0	***T45_4***	***T45_8***
Fmax [kN]	15,82	18,62	21,15
Δ F [%]	-	18%	34%
	T60_0	***T60_4***	***T60_8***
Fmax [kN]	14,37	16,67	17,49
Δ F [%]	-	16%	22%

Table 3: Comparison of vertical force as the beam length changes

	T30_0	***T45_0***	***T60_0***
Fmax [kN]	18,29	15,82	14,37
Δ F [%]	-	-13%	-21%
	T30_4	***T45_4***	***T60_4***
Fmax [kN]	20,73	18,62	16,67
Δ F [%]	-	-10%	-20%
	T30_8	***T45_8***	***T60_8***
Fmax [kN]	24,30	21,15	17,49
Δ F [%]	-	-13%	-28%

By rewriting the formula (1) as a function of the dependence on vertical displacement (δ_v) and beam length (L), the following expression is obtained:

$$F_N = 2N\left[\frac{\delta_v}{L\sqrt{1+\left(\frac{\delta_v}{L}\right)^2}}\right] \tag{6}$$

$$F_M = \frac{4M}{L}\left[\frac{1}{1+\left(\frac{\delta_v}{L}\right)^2}\right] \tag{7}$$

where F_N is the axial contribution to the total capacity and F_M is the flexural contribution. In the context of a given vertical displacement (δ_v), it is possible to analyze the axial and flexural contributions as a function of the beam span. The graphs presented in Figure 6 illustrate the variations in the axial and flexural contributions, expressed as a percentage of the total vertical force F, as the span length increases. It is evident that for small displacements, flexural contribution governs, as the relationships between bending moment (M) and axial force (N) are decoupled. Consequently, the flexural component is activated initially, followed by the axial component. However, as all components become activated, particularly at higher displacements, the catenary effect becomes more pronounced, although the flexural contribution continues to increase with span.

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Figure 6. Trends of axial and bending contributions as the length of beams changes

4 – ALTERNATIVE LOAD PATH FEM ANALYSIS OF A 4-STOREY ARCHETYPE BUILDING

4.1 ASSUMPTIONS AND DAMAGE SCENARIOS

The evaluation of structural robustness employs different approaches based on the knowledge of the event:

- Quantitative risk analysis is applied to events that can be modeled probabilistically, considering structural and action uncertainties, and quantifying risk in terms of losses;
- Scenario analysis is used for events that cannot be modeled, based on scenarios of initial actions or damages. This can be threat-dependent (based on the action) or threat-independent (based on initial damage). Robustness is assessed based on the load-bearing capacity of the damaged structure.

This research presents a deterministic analysis based on initial damage scenarios and the threat-independent approach. A direct approach was used, employing the Alternate Load Path method (ALP), to evaluate resistance to disproportionate collapse due to the removal of a structural element. Linear and nonlinear static analyses (ALPAs) were conducted on a four-story frame building, simulating the removal of a column at the ground floor, while neglecting dynamic effects.

4.2 ARCHETYPE BUILDING DESCRIPTION

A four-story case study residential post-and-beam building was studied (Fig 7b): with inter-story heights of 3.50 m, composed of a grid of columns with a cross-section of 32x32 cm, main beams with a cross-section of 24x48 cm and secondary beams with a cross-section of 14x36 cm placed at intervals of 100 cm. The span lengths are 5.00x4.00 m. A concrete core with a thickness of 20 cm serves as a bracing element. All timber elements are made of GL24h glulam [8]. All the beam-to-beam and beam-to-column connections were ALUMEGA. Above the beams, there are SWP panels with a thickness of 30 mm followed by non-structural layers (Fig. 7c) and 7d)).

All structural elements were designed for vertical loads only, both at the Ultimate Limit States (ULS) and Serviceability Limit States (SLS), as specified by the NTC2018 for residential buildings [9]. The live (LL) and superimposed (SID) loads were 2.0 kN/m2 and 3.97 kN/m2, respectively. The weight of all timber elements was 4.2 kN/m3. The total dead load (DL) accounted for both SID and self-weight. The four-storey building was modelled in SAP2000, a commercial FEA software, as shown in figure 5a.

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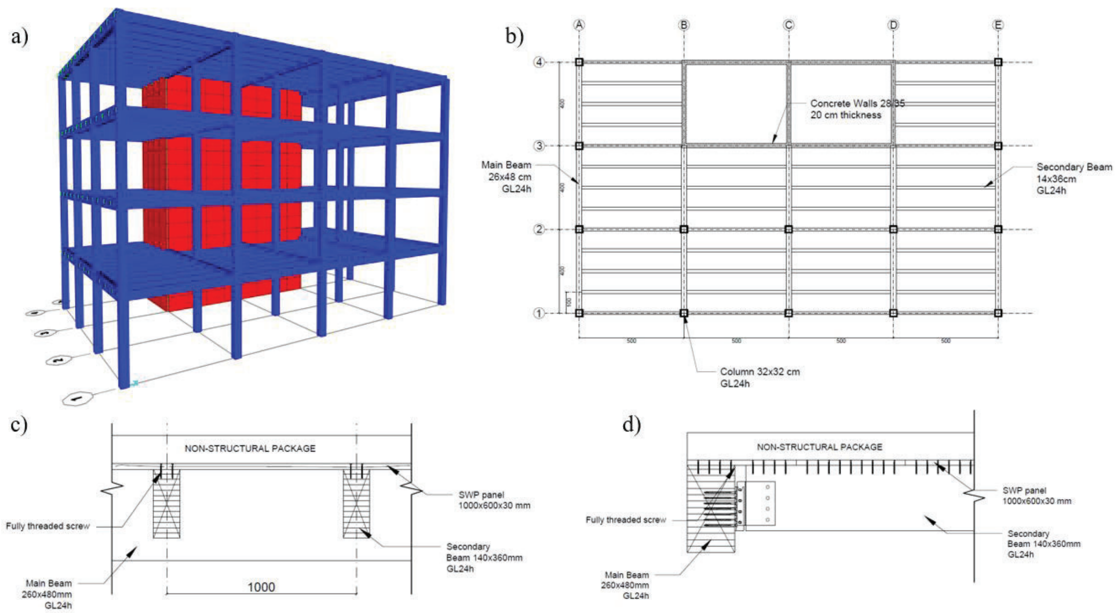

Figure 7. a) isometric view of FEM model; b) building plan floor [cm]; c) and d) construction detail of beam-to-beam connection [mm]

4.3 CONNECTION MODELING

In the FE model, wood has an elastic-brittle behavior and all non-linearities are concentrated at the connections, the following assumptions are introduced:

- Linear elastic constitutive laws for all beam and shell elements.
- Elastic-linearized constitutive laws for connections in linear analyses: to account for the geometric and non-linear properties of the connection, such as free rotation and post-elastic behaviors, equivalent stiffnesses were derived as linear segments connecting the null point to the point corresponding to maximum resistance (Fig. 8 a), b) and c)). The connections were modeled as "Linear Link" with uncoupled stiffnesses. Specifically, in the U3 direction (transverse to the joist axis and in the plane of the floor), infinite stiffness was considered, while in the R1 and R2 directions (torsion and in-plane rotation), due to the lack of experimental data, stiffnesses equal to a quarter of the out-of-plane R3 stiffness were used. All connections were thus modeled with elastic links of a length of 63 mm, corresponding to the extension of the real connection.
- Multi-linear constitutive laws for connections in non-linear analyses that account for plasticity, the free translation and rotation allowed by horizontal slots. The connections were modeled as "MultiLinear Plastic Link."
- The main beams were released for bending in-plane M2 and out-of-plane M3. All columns and shear walls were hinged at the base.

4.4 ROBUSTNESS MEASURES

An essential aspect of effectively assessing and mitigating the risk of disproportionate collapse is the quantification of robustness through appropriate measures.

For linear analyses, the method based on the Residual Influence Factor (RIF) [10] was employed. In order to measure the effect of full damage (or loss of functionality) of structural member no.i on the structural capacity, the so-called RIF-Value (sometimes referred to as the Damaged Strength Ratio) is defined by: RIFi = RSRfail,i/RSRintact where RSRintact is the RSR-value of the intact structure and RSRfail,i is the RSR-value of the structure where no.i is failed/removed. The RIF takes values between zero and one, with larger values indicating larger robustness.

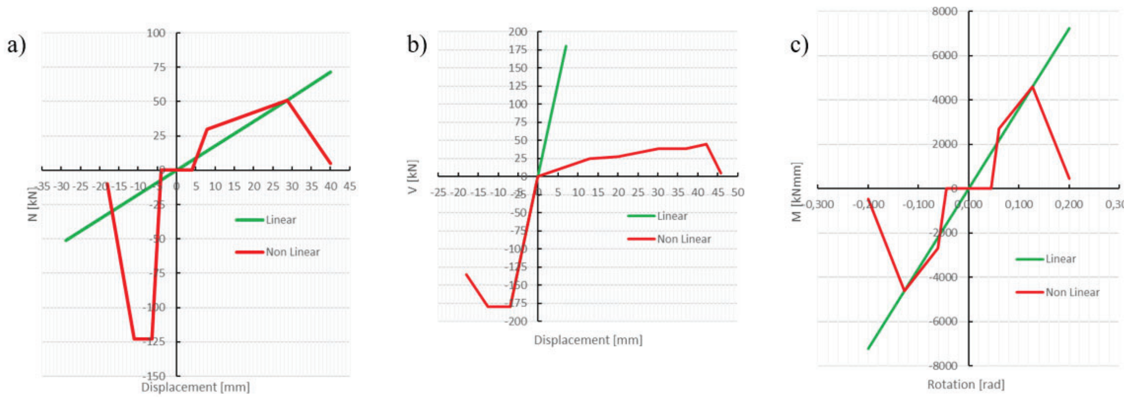

Figure 8. a) Linear and NonLinear axial constitutive law; b) Linear and NonLinear shear constitutive law; c) Linear and NonLinear bending constitutive law

For nonlinear analyses, vertical loads are applied through a multiplier (λ) according to the combination λ (G_1+G_2+0.3Q) [11]. The structure is considered robust if the maximum value of λ is greater than one. The analysis is a nonlinear static type and is based on the increment of gravitational loads or displacements until collapse or loss of numerical convergence. The multiplier α represents the portion of the reaction of the removed column that the structure can withstand until the first element fails.

In calculating the resistances, partial safety factors and k_{mod} values are used, varying based on loading conditions. For the intact structure, γ_m=1.45 and k_{mod}=0.80 were used, while for the damaged structure, γ_m=1.00 and k_{mod} =1.10 were used, considering it an accidental loading condition.

4.5 ALTERNATIVE LOAD PATH ANALYSIS RESULTS

This section provides a comparison of the results obtained in linear and non-linear analyses in terms of deformations, displacements, stresses and robustness levels obtained.

4.5.1 VERTICAL AND HORIZONTAL DISPLACEMENTS

In the linear analysis, the maximum vertical displacements on the order of five meters were obtained (Fig.9a), compared to one meter derived from the nonlinear analysis (Fig.9b). This significant divergence between the results should be interpreted considering the inherent characteristics of the two approaches: linear analysis, by its nature, does not account for mechanical nonlinearities, leading to numerically balanced results for the applied loads. This can, however, result in unrealistic estimates of displacements and deformations when the applied loads are particularly high. Figures 9c and 9d show the deformations with horizontal displacement values for the linear and non-linear analyses, respectively: in the non-linear analysis, a significant edge column deflection due to the catenary effect can be observed, while it is not detectable in the linear analysis. Consequently, it fails to capture second-order deformation mechanisms that enable the activation of alternative load paths such as the catenary effect.

4.5.2 INTERNAL FORCES ON BEAMS

The linear analysis is unable to capture the axial tensile internal forces in the beams caused by the catenary effect (Figure 10a), whereas with the non-linear analysis they are relevant. In addition, the considerable bending stresses on the edge column (Figure 10d) are worth mentioning. Forces due to second-order effects (e.g. due to catenary) must be considered because they can lead to premature collapse of the columns (e.g. due to buckling).

4.5.3 ROBUSTNESS INDEXES

A comparison was made between the robustness indexes obtained in terms of the capacity-to-residual-demand ratio [12-15]. In the linear static analysis, an RSR_{fail} value of 0.12 was obtained, associated with the bending failure of the connections applied to the removed column. In the nonlinear static analysis, the obtained multiplier is $\alpha_{fail,link}$ = 0.45 where collapse occurred due to bending failure of the connection. However, the edge column is not verified for buckling associated with the multiplier $\alpha_{fail,link}$, with a demand-to-capacity ratio of D/C = 1.67. Consequently, this multiplier was reduced to ensure a D/C ratio of 1.00. Thus, α_{fail} = 0.30 is obtained. Table 4 shows the results obtained.

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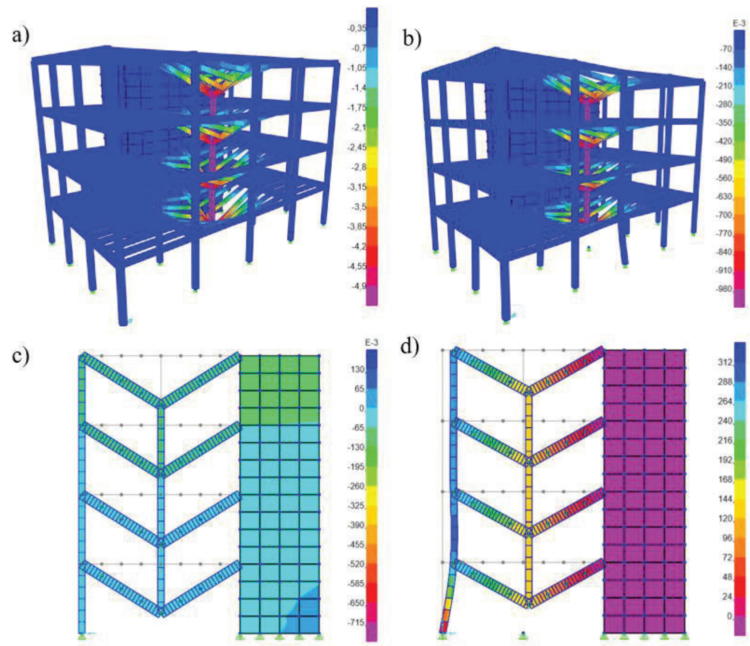

Figure 9. a) Vertical displacements of linear analyses; b) Vertical displacements of NonLinear analyses; c) Horizontal displacements of linear analyses; d) Horizontal displacements of NonLinear analyses

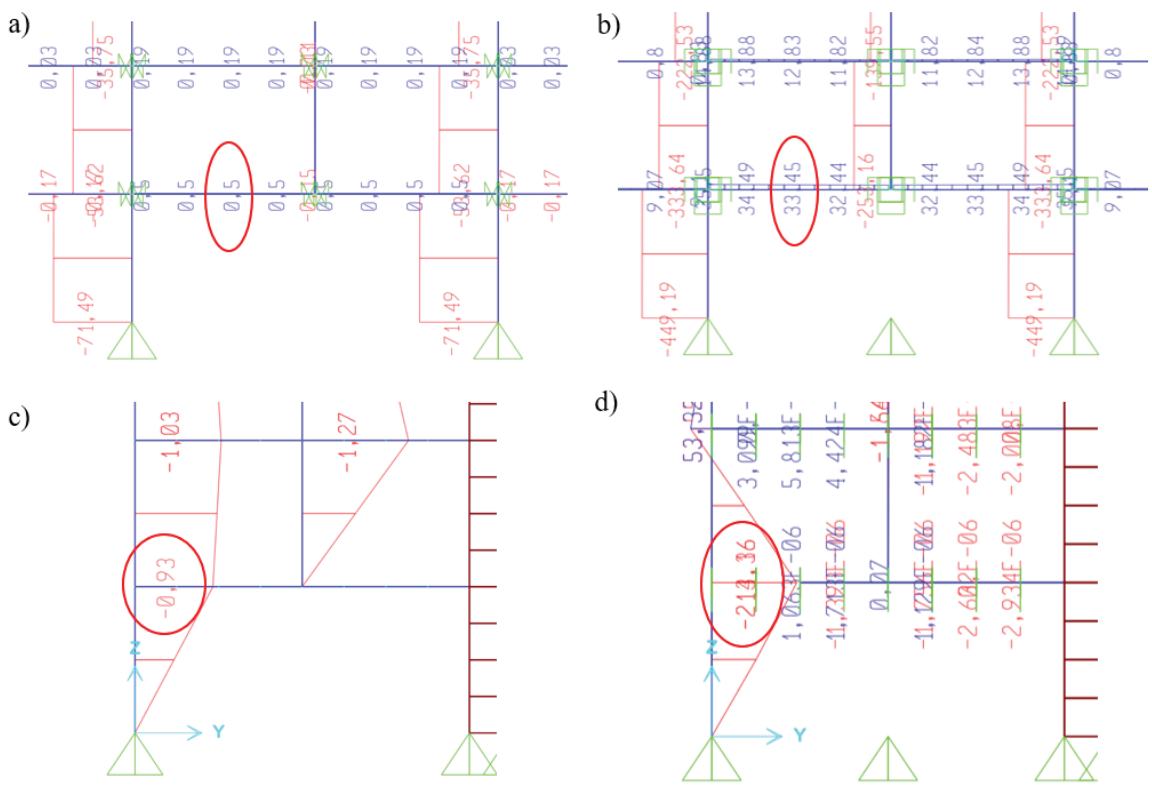

Figure 10. a) Axial stresses by linear analysis; b) Axial stresses by NonLinear analysis; c) Bending moment stresses by linear analysis; d) Bending moment stresses by NonLinear analysis

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The nonlinear analysis yields robustness values that are 250% higher. This is due to the ability of this analysis to take advantage of the mechanical nonlinearity of the connection, and thus its ductility.

Table 4: Robustness results by Linear and NonLinear analysis

Robustness Index	
Linear Analysis	***NonLinear Analys***
0.12	0.30

Although linear analysis has the advantage of being simple and computationally inexpensive, it is not suitable for a structural robustness study as it cannot account for various effects such as: geometric nonlinearity (second-order effects), material nonlinearities, redistribution of stresses and the catenary effect.

The use of this type of analysis generally leads to an excessively simplified solution and tends to underestimate structural robustness, with potential economic repercussions. Considering geometric and material nonlinearities allows for the use of additional reserves of strength, due to rope (or catenary) mechanisms. Thanks to these reserves, much higher vertical displacement capacities are achieved compared to the small displacement range and, most importantly, the ability to withstand greater gravitational forces.

In any case, the structure reached low levels of robustness against damage scenarios because of the redistribution of stresses due to the catenary effect.

6 – CONCLUSION

The research presented in this paper aimed to study and deepen the robustness behaviour of innovative connections for timber frame buildings and to compare different deterministic robustness analyses of a multi-story post-and-beam building, through both linear and nonlinear static analyses, based on an initial damage scenario.

First, the studies conducted on the innovative hinge connection led to the calibration and validation of a numerical model that accurately captured both the mechanical and, most importantly, geometric characteristics of this connection. This also involved the design, analytical formulation, and parametric analysis of a 2D post-and-beam subassembly, consisting of a 2-bay frame with a removed middle column, to simulate the loss of an interior column. Subsequently, both linear and nonlinear static ALP analyses were carried out to investigate the robustness of a 4-story post-and-beam archetype building subjected to an internal ground floor column removal scenario. From this study, the following conclusions can be drawn:

1) The use of slotted holes (i.e., free rotation) in the beam-to-column connection allows for an increase in the robustness of post-and-beam system to withstand vertical loads resulting from column removals. In fact, the gap introduced by slotted holes can be seen as an increase in the rotational capacity of the connection, which does not result from mechanical properties, but from geometrical ones. This allows greater activation of alternative load paths such as the catenary.
2) Linear static analyses, although they have the advantage of being simple and with a low computational burden, are not suitable for a study of structural robustness as there is no possibility of considering various effects due to geometric non-linearities and non-linear characteristics of the elements (e.g. connections). This generally leads to an overly approximate solution and tends to underestimate structural robustness, with possible economic repercussions. Considering geometric and material non-linearities, it is possible to draw on additional reserves of strength due to rope (or catenary) mechanisms. These reserves provide greater displacement capacities than in the field of small displacements and, above all, the ability to cope with greater gravitational actions.
3) The use of mechanical and geometric non-linearity assumptions allows the study of alternative load patterns and the definition of a reliable distribution of internal forces that could not be predicted through linear analyses.

The possibility of performing non-linear dynamic analyses to define the possibility of damage-induced impulsive effects and how innovative connections can mitigate these effects is considered an excellent topic for future development.

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