ORIGINAL ARTICLE

Bending resistance of steel-timber composite (STC) beams: analytical vs. numerical investigations

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Abstract

This contribution presents the results of analytical and numerical investigations of the ultimate bending resistance of simply supported steel-timber composite beams with uniformly distributed load, they consist of three main components: (i) the laminated veneer lumber (LVL) slab, (ii) the downstand steel beam (IPE-profile) connected by means of (iii) demountable shear connectors, which allow for deconstruction and circular reuse of the steel beam as well as the timber slab element. The mechanical properties of the cross-banded LVL as well as the load-slip curves of demountable shear connectors, which have been obtained experimentally, were implemented in both the analytical and numerical models. In this paper both solutions have been compared, first the ultimate bending resistance was calculated analytically through a strain-controlled approach with various degrees of shear connection. Then, numerical simulations were conducted in the finite element software ABAQUS to determine the ultimate bending capacity and the load-deformation behaviour. Finally, the results of both approaches are presented and compared.

Keywords

bending, steel-timber composite, LVL, demountable, deconstruction, reuse, circular economy, analytical, numerical

1 Introduction

Policies aimed at achieving carbon neutrality and the transition towards a circular economy to minimize resources depletion, such as the European Green Deal, the United Nation's Sustainable Development Goals, and the European Taxonomy Regulation, highlight the need for sustainable alternatives and improvements to existing non-sustainable practices.

In the construction sector, many of the current design and construction procedures follow a linear economy model, which means that the structures are not designed for easy deconstruction and reuse of their components. This leads to high resources consumption and waste generation. Therefore, it is evident that there is an urgent need for demountable and reusable structural solutions to facilitate the transition of the construction industry to a circular economy and to help with decarbonization efforts.

In response to the urgent need to provide sustainable structural alternatives, research projects focused on developing such solutions have increased in recent years. One example is the RFCS REDUCE project (grant No. SA710040) [1], which aimed to develop demountable and reusable steel and steel-concrete composite (SCC) structural solutions. The project has now been completed, and the resulting solutions offer promising alternatives to traditional construction practices.

On the basis of the REDUCE project, the research project "Prefa-SeTi: Steel-Timber Composite Beams" aims to develop demountable and reusable steel-timber composite (STC) beams and flooring systems. This project also seeks to extend the findings of REDUCE to STC beams, further contributing to the transition to a circular economy.

Within the framework of the Prefa-SeTi project, the bending resistances of various simply supported steel-timber composite (STC) beam configurations have been investigated using analytical and numerical approaches. This contribution presents results of the investigation, which include computations for different degrees of shear connection. Additionally, the experimentally obtained material properties of laminated veneer lumber (LVL) and the load-slip behaviour of demountable connections were considered in the computations.

2 Demountable shear connections

It is possible to create steel-to-timber connections using traditional timber connectors (e.g. bolts, screws, c-type connectors, etc.). Bolts work as dowels in compression, Geka connectors increase the surface in which the stress are transferred to the timber, and screws contribute with their withdrawal and shear capacities. However, screws can cause drilling damage and bolts can create preloading effects that may damage the timber, thereby hindering the reusability of the structural components. Therefore, alternative shear connectors are needed to develop structural solutions complying with the principles of circular economy. On basis of the existing structural solutions and adding steel tubes and plates, three solutions have been selected for investigation (see Figure 1):

- Shear Connector Type 1 (SCT-1): This basic solution, works majorly with the pure dowel solution and timber in bearing compression. The steel tube allows to reach the preload of the bolts while preventing crushing of timber. The round plate welded to the steel tube at the steel-timber interface increases the contact surface, hence, improving the embedment strength where compressive forces are larger.
- Shear Connector Type 2 (SCT-2): Similar to SCT-1, but the bottom round plate replaced by a Geka connector, making it an alternative option to SCT-1.
- Shear Connector Type 3 (SCT-3): Combines SCT-1 with a screwed solution, with the screws contributing to the connection's capacity and reducing slip compared to the other two solutions.

The connection devices are depicted in Figure 1. These connections were made of steel (S460) and consist of a steel tube welded to one of three steel plates: (i) a round steel plate, (ii) a geka connector, or (iii) a rectangular steel plate with four inclined screws, respectively for each connection type. These devices protect the timber from premature damage due to the bolt preload and increase the surface in which the forces are transferred to the timber. Inside the tube, a high strength bolt M20 grade 10.9 was installed and preloaded at 70% of its ultimate tensile strength.

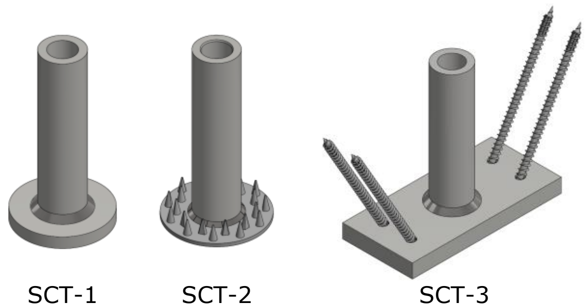

Figure 1 Shear connection devices developed in Prefa-SeTi project.

To determine the load-slip response of the connections, three push-out tests of each connection type were conducted. The connections were installed in Kerto-Q LVL [2] panels with a thickness of 144mm. The push-out test setup is illustrated in Figure 2, while the cross-section details of SCT-1 are displayed in Figure 3. The configurations for SCT-2 and SCT-3 are analogous to SCT-1.

The mean curves of each connection are presented in Figure 4. The three connections exhibited large deformation capacity. Their behaviour is non-linear, it is characterized by (see Figure 4): (i) an initial stiff response until the slip resistance given by the bolt preload, (ii) when the slip resistance is overcome, there is sliding friction, which results in a small displacement of about 2-4 mm with a negligible increase in the load until contact bearing between the bolt and both the steel flange and the connection device starts, (iii) then there is bearing and shear in the bolt and embedment of the connection device in the timber, which produce a nonlinear monotonic increasing branch in the load-slip curve.

Figure 2 Push-out test setup of Prefa-SeTi shear connections.

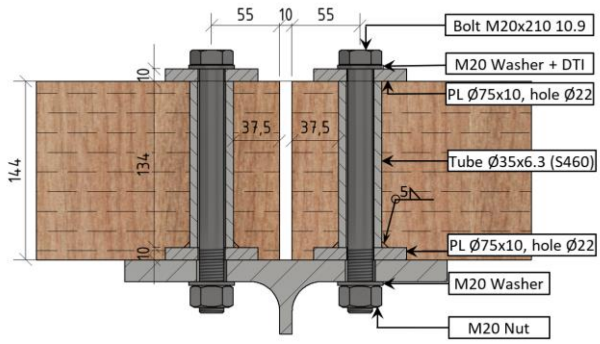

Figure 3 Cross-section details of shear connection type 1 (SCT-1).

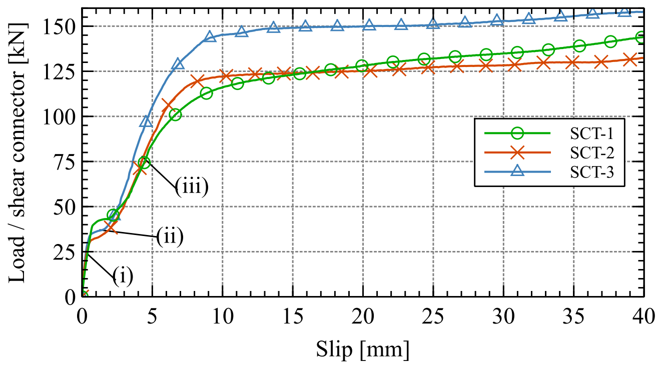

Figure 4 Mean load-slip curves obtained in the push-out tests of the three demountable connections of Prefa-SeTi project.

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(iii) then there is bearing and shear in the bolt and embedment of the connection device in the timber, which produce a nonlinear monotonic increasing branch in the load-slip curve.

In this study, the load-slip curve of STC-1 was incorporated into the numerical models to simulate various degrees of shear connection. The details of this implementation can be found in section 3.2.3.

3 Finite element modelling

3.1 General

The finite element (FE) models were developed using Abaqus software [3] and its implicit static solver. Simply supported beams with uniform distributed load (UDL), consisting of an LVL slab, steel beam, and two rows of shear connectors, were modelled using 4-node shell elements with reduced integration (S4R) for the slab and steel beam. The connectors were modelled as mesh-independent point-based fasteners. Table 1 summarizes the configurations analysed in this study for various degrees of shear connection η. Figure 5 shows the main components of the beams.

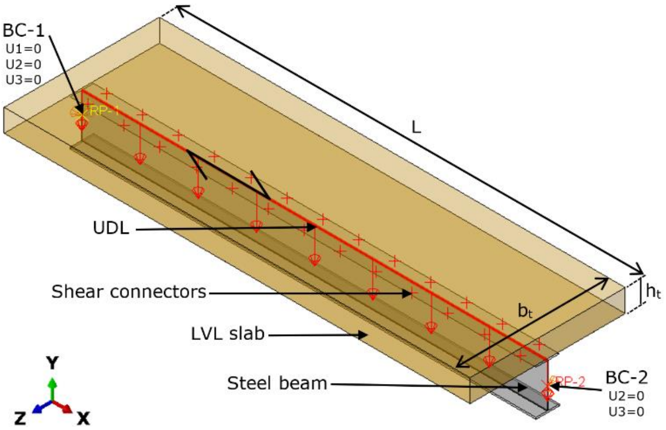

Figure 5 Finite element model of the steel-timber composite beam.

Table 1 Summary of STC beam configurations analysed in this study.

Configuration ID	Span L [m]	Steel beam section	LVL height h_t [mm]	LVL width b_t [mm]
6-IPE300-LVL144	6	IPE 300	144	1500
8-IPE360-LVL144	8	IPE 360	144	2000
10-IPE400-LVL144	10	IPE 400	144	2500
12-IPE450-LVL144	12	IPE 450	144	3000

3.2 Material properties and shear connection

3.2.1 Timber

The strength and stiffness of LVL is about ten times greater in the direction of the grain than the values perpendicular to the grain. Hence, to obtain a system with the maximum possible stiffness and resistance, the grain orientation of the timber panels was assumed to be parallel to the longitudinal direction of the beam. It was further assumed that the bending behaviour of the beam is governed by the mechanical properties in the direction of the grain. The mechanical properties of Kerto-Q LVL [2], a cross-banded LVL product manufactured by MetsäWood, were determined experimentally and the mean values were incorporated into the FE model.

The test results indicate that LVL exhibits plastic behaviour under compression and is brittle under tension. As such, a bilinear elastic perfectly plastic stress-strain relationship was considered for compression, while a brittle linear elastic stress-strain relationship was used for tension. The elastic moduli $E_{c,1}$ and $E_{t,1}$ were set to 8 000 MPa, with compressive strength $f_{c,1}$ at 41 MPa and tensile strength $f_{t,1}$ at 49 MPa. These stress-strain relationships are illustrated in Figure 6. Additionally, Poisson's ratio ν was set to 0.4.

3.2.2 Steel

The model incorporated the properties of structural steel S355, including a bilinear elastic-perfectly plastic stress-strain relationship as shown in Figure 7. The elasticity modulus E_s was set to 210 000 MPa, the yield strength f_y was set to 355 MPa and the Poisson's ratio ν was set to 0.3.

3.2.3 Shear connection

The LVL panels were connected to the top flange of the steel beams with mesh independent point-based fasteners. The fasteners were placed at predefined points along the beam to couple the slab to the top flange, allowing only relative displacement along the longitudinal direction of the beam.

Two rows of equidistant connectors were modelled and their behaviour of the connectors was defined by introducing the experimentally obtained mean load-slip curve of shear connection type 1 (see Figure 4). The number of connectors and the spacing between them was calculated for each configuration and degree of shear connection, based on the analytical estimation of the resultant normal force in the LVL slab and by following the algorithm proposed by Kozma [4,5] to determine the parameter k_{flex} and the effective load P_{eff} taken by each shear connector.

The values obtained for the parameter k_{flex} were 0.77, 0.73, 0.74 respectively for SCT-1, SCT-2 and SCT-3. Similarly, the effective resistance of the connectors was calculated and the following values were obtained: 73.9, 76.3 and 88.7 kN respectively for SCT-1, SCT-2 and SCT-3.

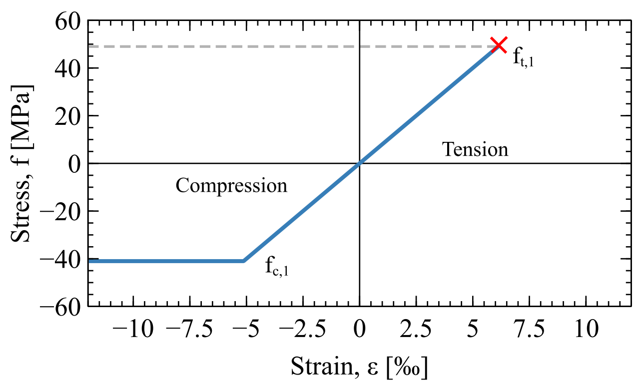

Figure 6 Stress-strain relationship of LVL in the longitudinal direction.

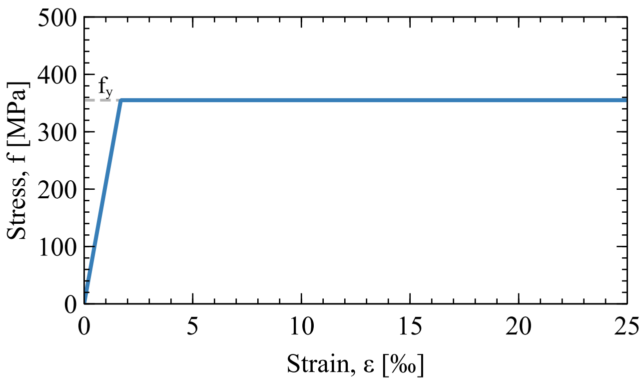

Figure 7 Stress-strain relationship of steel S355.

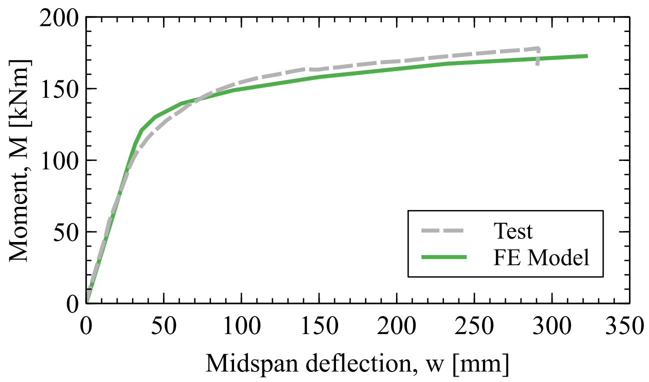

Figure 8 Experimentally and numerically obtained load-deflection curves of a STC beam tested by Hassanieh et al. [6].

3.3 Mesh, boundary conditions and loading procedure

3.3.1 *Mesh*

The slab and the steel beam were meshed with 4-node shell elements with reduced integration (S4R) and 5 integration points through the thickness. A sensitivity analysis was done to determine an efficient mesh size. The average mesh size of the steel beam was set to 50 mm and for the slab it was set to 100 mm. The analysis revealed that a finer mesh size would not significantly affect the results.

3.3.2 *Boundary conditions*

The boundary conditions (BC) were applied to two reference points located at both ends of the beam in the middle of the web, as shown in Figure 5. These points couple the nodes of the web. At one end, the beam was restricted from translating in the three principal directions (i.e., BC-1: U1=0, U2=0, U3=0), while at the other end, only the longitudinal displacement along the X-axis was allowed (i.e., BC-2: U2=0, U3=0). The rotation around the Z-axis was unrestrained at both ends (i.e., UR1=0, UR2=0).

3.3.3 *Loading procedure*

The uniformly distributed load was transferred to the beam as a traction load applied on a strip located on the surface of the top flange of the steel beam (see Figure 5) to produce sagging bending. The load was applied as a traction load in the direction of the Y-axis (vertical and downwards) using the vector (0,-1,0), these settings ensure its consistent application throughout the simulation.

3.4 Model validation

With the mentioned modelling approach, a simply supported STC beam tested by Hassanieh et al. [6] under 4-point bending has been modelled and the results obtained in the numerical model were compared with the experimental results. The STC beam with a span of 6 m, consists of a universal beam 250 UB 25.7, a 75 mm thick and 400 mm wide LVL slab, and M12 grade 8.8 bolts as shear connectors. The comparison between the curves obtained in the test and in the FE model (see Figure 8) showed good agreement in terms of load-deflection, thus affirming the validity of the modelling approach.

4 Analytical calculation of the ultimate bending resistance of the section

The ultimate bending resistance of the section was calculated using an iterative strain-controlled approach. In this method the components of the cross-section are divided into horizontal lamellas to determine the distribution of normal stresses at each lamella and the corresponding forces needed to satisfy the equilibrium condition (i.e. $\sum F = 0$).

In this approach the equilibrium is achieved for the maximum allowable curvature by defining strain limits for the materials and starting the iterations from the maximum possible curvature according to the strain limits. Then, the curvature is modified stepwise within the predefined strain limits, until a strain distribution and its corresponding stress distribution (defined by the material stress-strain relationships) produce resultant forces which are in equilibrium. This algorithm was implemented in MATLAB and the calculations were performed for the configurations and material laws of the present study.

The bending resistance of the section was calculated by considering the stress-strain laws of steel and timber, using (i) a plastic stress distribution ($M_{R,pl}$) and (ii) an elastic perfectly plastic stress distribution ($M_{R,el-pl}$). For the elastic-plastic scenario, the stress strain relationships presented in section 3.2 were utilized. In these investigations the strain limits for steel were set to 15% for tension and compression. For timber the strain limits were set to 1.5% for compression and 0.6125% for tension. The calculations were performed for various degrees of shear connection from 0 to 1 inclusive (i.e. $0 \leq \eta \leq 1$).

5 Results

The midspan deflection versus moment curves were obtained from the FE models for the four configurations analysed in this study and are presented in Figures 9 to 12. The last point of each curve corresponds to the point at which failure occurred, in all cases this happened when the tensile strength of the timber was reached in the soffit of the slab at midspan. According to these analyses, the beams exhibited large deformations at their ultimate state.

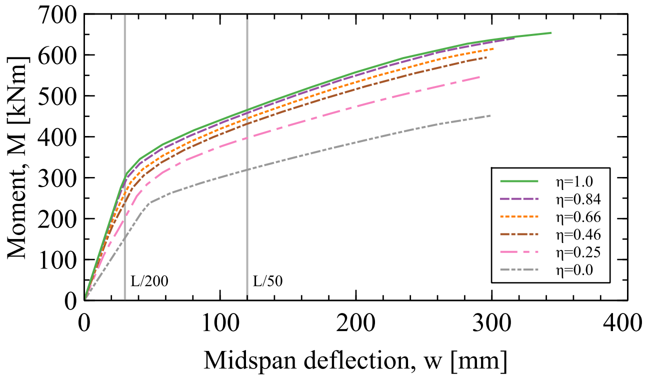

Figure 9 Midspan vs. Moment curves of the STC beam with a span of 6 m (configuration 6-IPE300-LVL144).

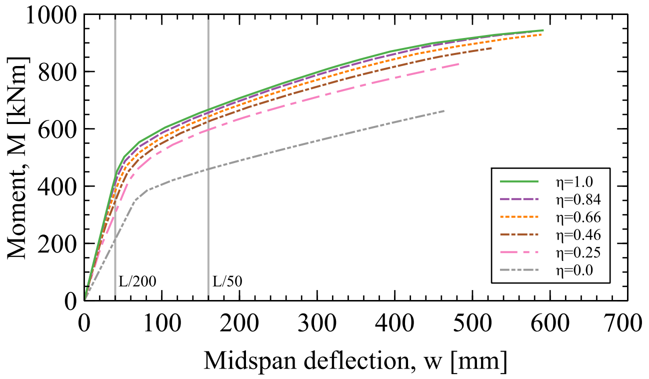

Figure 10 Midspan vs. Moment curves of the STC beam with a span of 8 m (configuration 8-IPE360-LVL144).

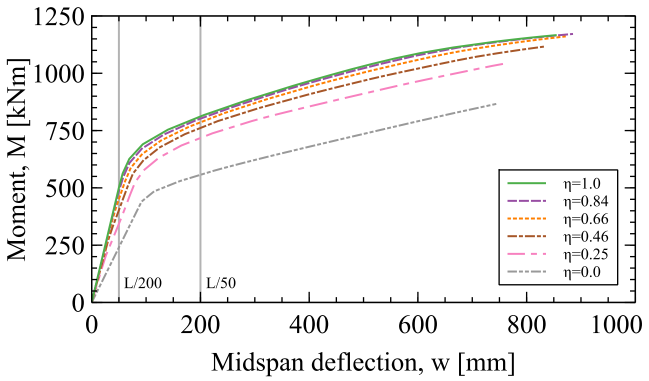

Figure 11 Midspan vs. Moment curves of the STC beam with a span of 10 m (configuration 10-IPE400-LVL144).

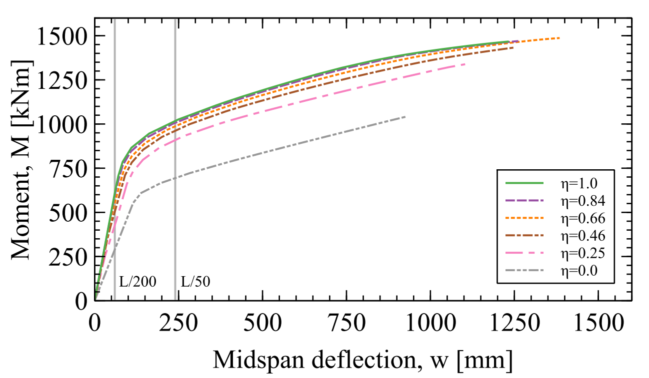

Figure 12 Midspan vs. Moment curves of the STC beam with a span of 12 m (configuration 12-IPE450-LVL144).

A comparison of the analytically and numerically obtained resistances is depicted in Figure 13. In both, the analytical and the FE models elastic-plastic stress-strain relationships were implemented. According to the results, the analytical calculations and the numerical models are in good agreement, the difference between both remains within a range of about ±5%.

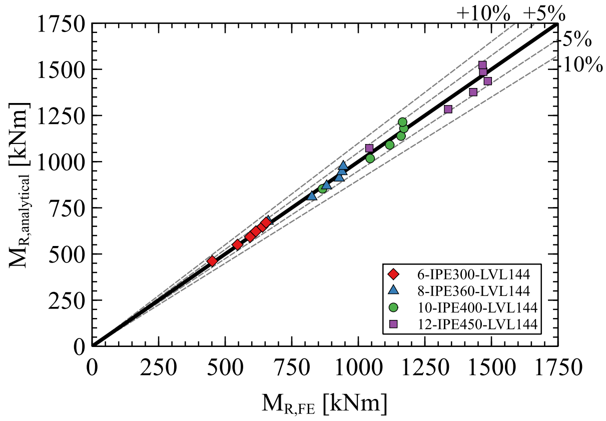

Figure 13 Comparison of the analytically (elastic-plastic analyses) and numerically (FE models) obtained resistance values.

Figure 14 shows a comparison of the analytical resistances calculated under perfectly plastic assumptions and the resistances obtained in the FE models which implemented elastic-plastic stress strain relationships for the materials. According to the results, the analytical model tends to overestimate the bending capacity of the STC beams, in most cases by more than 5% of the numerically obtained resistance values.

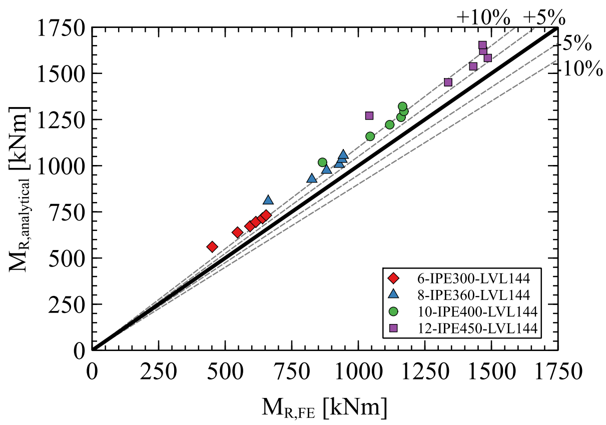

Figure 14 Comparison of the analytically (plastic analyses) and numerically (FE models) obtained resistance values.

6 Conclusions

The main conclusions of this investigation are the following:

- The use of the parameter k_{flex} to estimate the effective shear resistance and the number of connections required in the finite element model for various degrees of shear connection leads to ultimate resistance values that are in good agreement with the analytically obtained resistances when the elastic-plastic material laws are considered in both the numerical and the analytical computations.
- STC beams exhibit a large deformation capacity before

reaching failure. However, limitations are to be imposed to ensure that deflections remain within allowable values.

- The maximum bending capacity is reached beyond L/50. That means, additional investigations about maximum allowable slip and strain limit of the timber have to be investigated.
- The analytical model in which the materials are assumed to have a perfectly plastic behaviour was investigated due to its potential to be implemented in analytical equations. However, the resistances calculated under these assumptions tend to be larger than the ones obtained in the FE models. Therefore, additional factors are to be incorporated to the model to improve their accuracy and align the results with those obtained from the FE models.

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