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UNREINFORCED AND STEEL-REINFORCED COLUMNS MADE OF EUROPEAN BEECH GLUED-LAMINATED TIMBER

Katharina Sroka [1], Pedro Palma [2], René Steiger [3], Thomas Ehrhart [4], Andrea Frangi [5], Thomas Strahm [6], Ernst Gehri [7]

ABSTRACT: With the aim of promoting the use of glued-laminated timber (GLT) made from European beech (*Fagus sylvatica* L.), experimental and numerical studies were conducted on high-strength columns for residential, office, and industrial applications. The experimental research comprised compression tests on stocky and slender unreinforced and steel-reinforced beech GLT columns of strength classes GL40h, GL48h, and GL55h. A finite element model was developed to perform parametric studies on geometrical and mechanical properties and to evaluate the load-carrying capacity across different slenderness ratios. The investigations revealed that the columns can be designed using the *effective length method*, however with adapted critical relative slenderness $\lambda_{rel,0}$ and straightness factor β_c. Corner steel reinforcement of grades ST900/1100 and ST950/1050 leads to a marked increase in the load-carrying capacity of stocky as well as slender GLT columns.

KEYWORDS: hardwood, GLT, column, buckling, compressive strength, steel-timber hybrid member

1 INTRODUCTION

Growing interest in timber high-rise buildings is creating demand for high-strength timber columns. In high-rise or industrial buildings, the required cross-sections of timber columns can become excessively large, critically limiting the available floor space or even the structural feasibility, and obstructing window views. Compared to softwood, European beech (*Fagus sylvatica* L.) glued-laminated timber (GLT) offers a high compressive strength parallel to the grain. Beech wood is widely available in Central European forests, but it is barely used for structural purposes [1] and beech GLT is not integrated in the European standards.

With the aim of using beech GLT for high-strength columns in residential, office, and industrial applications, the company *neue Holzbau AG* developed hybrid beech GLT columns reinforced with steel bars. The steel reinforcement substantially improves the load-carrying capacity and helps to overcome the buckling issues that limit the utilisation of the higher compressive strength, often making slender columns uneconomical. The steel reinforcement offers additional advantages by enabling an increased ductility of the columns, stiff connections to other elements, a ductile failure mode governed by the connections, reduced differential settlements to concrete elements in hybrid structures, and better resistance to impact loading. The glued-in steel bars can also be used to increase structural robustness by allowing floor-to-column and column-to-column connections to carry tensile forces and hold the floor below in case a column fails [2]. A similar strategy was implemented using glued-in steel rods as vertical ties in the HoHo building in Vienna [3].

Full-scale experiments and numerical simulations were conducted to investigate the axial load-carrying capacity of unreinforced and steel-reinforced columns with different slenderness ratios. Key factors influencing the load-carrying capacity were identified from parametric studies. A design model that can be easily adopted in engineering practice was developed.

The objective was to enhance the understanding of the buckling behaviour of these high-strength columns and to demonstrate their application potential.

[1] Katharina Sroka, Empa – Swiss Federal Laboratories for Materials Science and Technology, Structural Engineering Research Lab, Switzerland, katharina.sroka@empa.ch
[2] Pedro Palma, Empa – Swiss Federal Laboratories for Materials Science and Technology, Structural Engineering Research Lab, Switzerland, pedro.palma@empa.ch
[3] René Steiger, Empa – Swiss Federal Laboratories for Materials Science and Technology, Structural Engineering Research Lab, Switzerland, rene.steiger@empa.ch
[4] Thomas Ehrhart, Schnetzer Puskas Ingenieure, Switzerland, t.ehrhart@schnetzerpuskas.com
[5] Andrea Frangi, ETH Zürich – Swiss Federal Institute of Technology, Institute of Structural Engineering, Switzerland, frangi@ibk.baug.ethz.ch
[6] Thomas Strahm, neue Holzbau AG, Switzerland, thomas.strahm@neueholzbau.ch
[7] Ernst Gehri, Prof. emeritus ETH Zürich, Dr. h.c., Switzerland, gehri@emeritus.ethz.ch

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2 EXPERIMENTS

2.1 MATERIALS

2.1.1 Hardwood GLT

Swiss-grown European beech boards were used for the production of the GLT columns. The boards were strength graded into tensile strength classes T33 (for GL40h), T42 (for GL48h), and T50 (for GL55h), applying the strength grading rules described by Ehrhart et al. [4]. The laminations were bonded using a 1-comp. PU adhesive and a primer [5]. The lamination thickness was 25 mm and the wood moisture content was $\omega = 8 \pm 2\%$ representing the climatic conditions of indoor use.

2.1.2 Reinforcement

The steel reinforcement bars placed either in the centre or in the four corners of the cross-section (Figure 1) were profiled Swiss-GEWI bars of grade ST900/1100 for diameters below 29 mm, and of grade ST950/1050 for diameters greater than 29 mm. The steel grades were chosen from products available on the market. To avoid preliminary yielding of the reinforcement, the yield strength of steel was chosen based on Formula (1).

$$\frac{f_y}{E_s} > \frac{f_{c,0,05}}{E_{c,0,05}} \therefore f_y > \frac{62.5 \cdot 210\,000}{15\,400} \approx 850 \text{ MPa} \qquad (1)$$

where f_y = yield strength of steel, E_s = modulus of elasticity of steel, $f_{c,0,05}$ = 5th percentile value of the compressive strength parallel to the grain for GL48h made of beech, and $E_{c,0,05}$ = 5th percentile value of the compressive modulus of elasticity parallel to the grain for GL48h made of beech. 5th percentile values for GL48h columns with a square cross-section and side length of 200 mm were reported by Ehrhart et al. [6, 7].

To insert the steel reinforcement, the GLT specimens were cut lengthwise at the designated position of each steel bar. Lengthwise notches for the steel bars were milled into the GLT pieces which were then block glued under pressure, using a 1-comp. PU adhesive with primer. Finally, the steel bars were inserted into the notches which were filled using an epoxy adhesive.

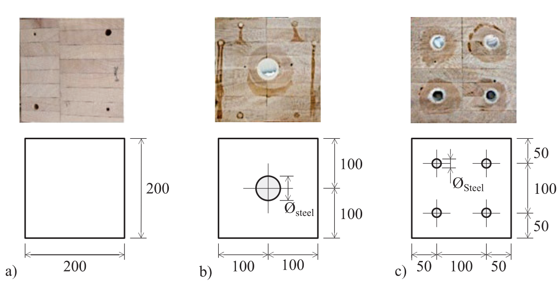

Figure 1: Unreinforced (a) and reinforced cross-sections using one central rebar (b) or four corner rebars (c).

2.2 COMPRESSION TESTS ON STOCKY COLUMNS

2.2.1 Test configurations

Compression tests on 1200 mm long stocky columns were carried out to evaluate the axial compressive strength of unreinforced and steel-reinforced European beech GLT (Table 1). Due to the support conditions prescribed by EN 408 [8] (loading heads locked against rotation during the test), the effective length was $L_{eff} \approx 0.6\, L$ [9]. The tests on unreinforced beech GLT columns were performed by Ehrhart et al. [6, 7]. For the steel-reinforced columns, different arrangements (diameter $Ø_{steel}$ and number of bars n_{rebars}) and degrees (ρ) of steel reinforcement were investigated to optimise the axial load-carrying and deformation capacities, while considering the increase in production effort.

***Table 1:** Compression test specimens (square side length a = 200 mm, L_{eff} = 720 mm).*

Strength class [-]	$Ø_{steel}$ [mm]	n_{rebars} [#]	ρ [%]	$n_{specimens}$ [#]
GL40h	-	-	-	7
GL48h	-	-	-	7
GL55h	-	-	-	7
GL55h	30	1	1.77	3
	40	1	3.14	3
	15	4	1.77	3
	20	4	3.14	3

2.2.2 Test results

During the compression tests on the stocky columns, no horizontal displacement was observed before failure. Crushing of the fibres initially occurred close to local imperfections (i.e. knots, finger joints, deviations in the direction of wood fibres) and finally progressed over the whole cross-section, followed by splitting. Sample force-displacement curves of individual specimens are shown in Figure 2.

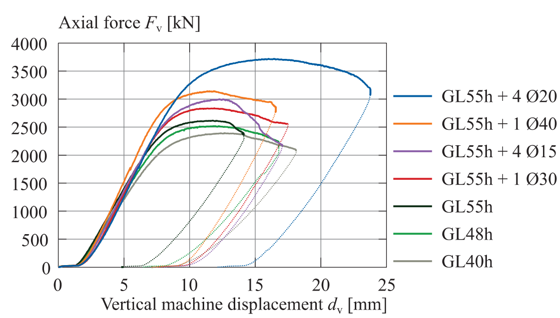

Figure 2: Sample axial force-displacement curves obtained for individual specimens in the compression tests on stocky columns.

The steel reinforcement bars increased the compressive strength and the modulus of elasticity parallel to the grain of the beech GLT columns (Table 2). The corner reinforcement with 20 mm diameter (ρ = 3.14%) led to a particularly pronounced increase in the axial load-carrying capacity of around 40% compared to the unreinforced specimens. An equal amount of central reinforcement only led to a 20% increase, likely due to a less favourable distribution of the applied axial load into the cross-section.

In the tests on unreinforced columns, very similar mean values of compressive strength parallel to the grain $f_{c,0,mean}$ were obtained for all tested strength classes, with an increase of only 3% from GL48h to GL55h (Table 2). The stocky steel-reinforced specimens (Section 2.2) were of strength class GL55h, whereas the slender steel-reinforced columns (Section 2.3) were of strength class GL48h. The results of both test series can be analysed together given the very small difference in compressive strength between GL48h and GL55h, which indicates that the strength class (GL48h or GL55h) has little influence on the axial load-carrying capacity in compression.

Table 2: *Mean experimental axial load-carrying capacity $R_{c,0,mean}$, compressive strength $f_{c,0,mean}$, and modulus of elasticity $E_{c,0,mean}$ parallel to the grain.*

Strength class [-]	$n_{specimens}$ [#]	$R_{c,0,mean}$ [kN]	$f_{c,0,mean}$ [MPa]	$E_{c,0,mean}$ [GPa]
GL40h	7	2393	60.4	15.1
GL48h	7	2526	63.8	16.0
GL55h	7	2607	65.8	17.0
GL55h + 1 Ø30	3	2851	71.3	20.1
GL55h + 1 Ø40	3	3104	77.6	22.2
GL55h + 4 Ø15	3	2985	74.6	19.0
GL55h + 4 Ø20	3	3585	89.6	23.3

2.3 BUCKLING TESTS ON SLENDER COLUMNS

2.3.1 Test configurations

Slender columns of strength class GL48h and with effective lengths of 2400 mm and 3600 mm were subjected to buckling tests. These effective lengths were chosen to represent the typical heights of columns in high-rise and industrial buildings. The planned initial eccentricity was $e_0 = L_{eff}/500$ but deviations occurred due to manufacturing inaccuracies. The tested configurations are listed in Table 3, the test setup is depicted in Figure 3. The tests on unreinforced beech GLT columns were performed by Ehrhart et al [6, 7].

Table 3: *Buckling test specimens (a = 200 mm, GL48h).*

L_{eff} [mm]	$Ø_{steel}$ [mm]	n_{rebars} [#]	$n_{specimens}$ [#]	$L_{eff}/e_{0,mean}$ [-]
2400	-	-	5	382
3600	-	-	5	590
2400	40	1	3	436
3600	40	1	3	554
2400	20	4	3	415
3600	20	4	3	618

2.3.2 Test results

The buckling tests on slender columns focused on the reinforcement layouts with higher reinforcement ratio (1× Ø40, 4× Ø20), since these demonstrated the maximum gain in mechanical properties during the compression tests on stocky columns.

The test results are presented in Figure 4, sample load-displacement curves are shown in Figure 8. For GL48h, the central reinforcement of 40 mm diameter led to a 20% gain in the axial load-carrying capacity in columns of low to medium effective length (≤2400 mm) and to a 10% gain for long columns (3600 mm). The corner reinforcement with four rebars of 20 mm diameter and an edge distance of 50 mm led to a 40% increase in the load-carrying capacity across the investigated buckling lengths because it increases the bending stiffness of the columns.

Figure 3*: Buckling test of slender column at Empa.*

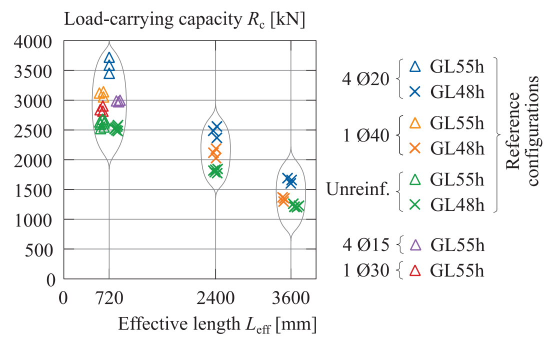

Figure 4*: Experimentally-determined axial load-carrying capacity of unreinforced and steel-reinforced European beech GLT columns.*

3 NUMERICAL SIMULATIONS

3.1 FINITE ELEMENT MODEL

A framework to model unreinforced and steel-reinforced GLT columns under compressive axial loads was developed using OpenSeesPy [10], which is a Python [11] library for the OpenSees finite element framework [12]. The model considered geometric and material nonlinearity. The initial deformed shape of the discretised column followed a sinusoidal curve with user-defined initial mid-span eccentricity. Boundary conditions were defined as depicted in Figure 5. An external vertical compression force (or displacement, as chosen by the user) was applied at the upper node.

The cross-section of the column elements was modelled as a fibre section using different patches for the timber laminations and the steel reinforcement (Figure 6). The fibres in these patches were associated with the uniaxial stress-strain relationship of the corresponding material. A bilinear material without hardening was adopted for the steel fibres, while the timber fibres were assigned the nonlinear material model for European beech defined by Glos [13].

Beech GLT material properties were based on the mean experimental results for unreinforced GL48h columns

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with a section size of 200 mm ($f_{c,0,mean}$ = 63.8 MPa; $E_{c,0,mean}$ = 16.0 GPa). The quasi-static structural analyses were performed under load control, since the aim of the simulations was to determine the load-carrying capacity of the columns.

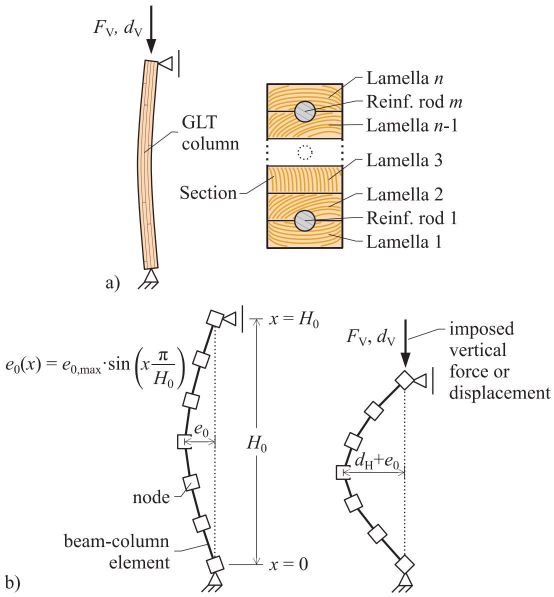

Figure 5: *Numerical model of the GLT column: a) idealisation of the column and cross-section; b) FE model, including initial deformed shape.*

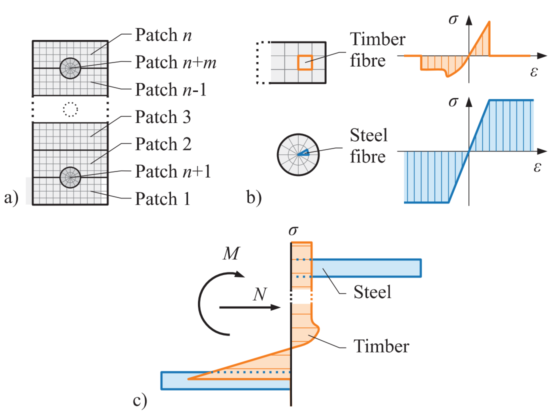

Figure 6: *Modelling of the cross-sectional behaviour: a) fibre patches; b) uniaxial material models for timber and steel; c) stresses and internal forces in the cross-section.*

3.2 MODEL VALIDATION

The model was validated using the experimental results. A comparison between simulated and experimental load-carrying capacities is presented in Figure 7, showing a very good agreement. A small deviation occurred for the unreinforced specimens with effective length of 3600 mm. In contrast to the experimental results, the simulated load-carrying capacity of the unreinforced column was higher than that of the centrally reinforced column at high slenderness (see the orange and green data points in Figure 7 with load-carrying capacities between 1000 and 1500 kN). Since the buckling curves of unreinforced and centrally reinforced GLT columns converge for high slenderness, the impact of this deviation is negligible.

The load-displacement curves obtained from the simulations also agree well with the experimental results, as shown in Figure 8 for three exemplary specimens with effective length of 2400 mm. Since the simulations were carried out under load control, the post-peak behaviour is not captured by the numerical results.

The validation shows that the model was able to simulate the buckling behaviour of the columns and estimate the mean load-carrying capacity with adequate accuracy.

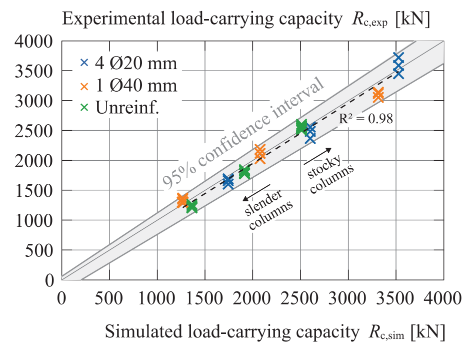

Figure 7: *Validation of the numerical results: comparison with the experimental mean axial load-carrying capacity for each reference configuration (GL48h unreinforced, GL48h with 1× Ø40, GL48h with 4× Ø20).*

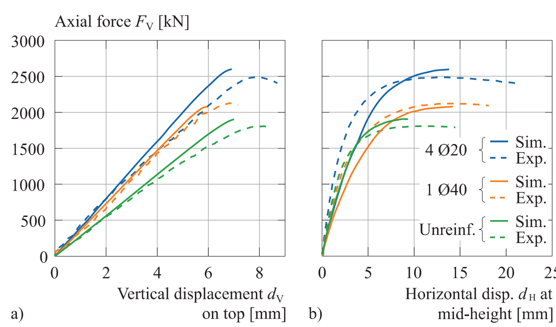

Figure 8: *Comparison of simulated and experimental load-displacement curves for GL48h columns with L_{eff} = 2400 mm: a) vertical displacement at the top of the column; b) horizontal displacement at mid-height of the column.*

3.3 PARAMETRIC STUDIES

The objective of the numerical framework was to gain further insights into the buckling behaviour of the composite columns. First, buckling curves were generated by simulating columns of varying effective lengths around the experimental range (Figure 9). Second, parametric studies were conducted on the most important parameters identified in the experimental campaign: i) initial eccentricity ($e_0 = L_{eff}/300$, $L_{eff}/500$, and $L_{eff}/1000$); ii) diameter of the central and corner steel bars (1× Ø30, 1× Ø40, 4× Ø15, 4× Ø20, 4× Ø25); iii) size of the cross-section (a = 200, 250, and 300 mm); iv) timber strength class (GL40h, GL48h, and GL55h). Benchmarking values were $e_0 = L_{eff}/500$, a = 200 mm, and GL48h.

Maximum reductions in the load-carrying capacity of 13 to 17% were observed when increasing the initial eccentricity from $L_{eff}/1000$ to $L_{eff}/300$ (Figure 10). Decreasing the diameter of the central steel rod from 40 to 30 mm led to reductions of 15% for stocky columns and of less than 5% for slender columns (Figure 11). The load-carrying capacity for all investigated slenderness ratios reduced by more than 20% when changing the diameter of the four corner steel bars from 25 to 15 mm (Figure 12). The size of the cross-section has a pronounced impact on the load-carrying capacity for column lengths relevant for structural applications (Figure 13). The edge distance of 50 mm was kept constant when varying the section size of corner-reinforced columns. Figure 14 confirms that the influence of the GLT strength class is very limited.

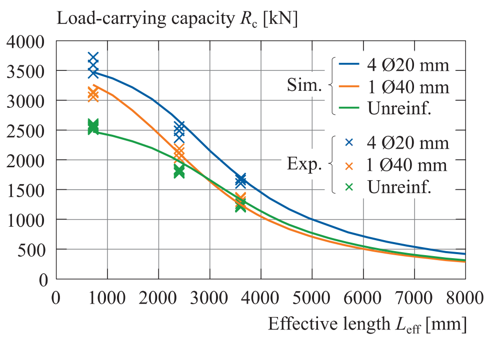

Figure 9: *Simulated buckling curves for the three reference configurations, compared with the experimental results.*

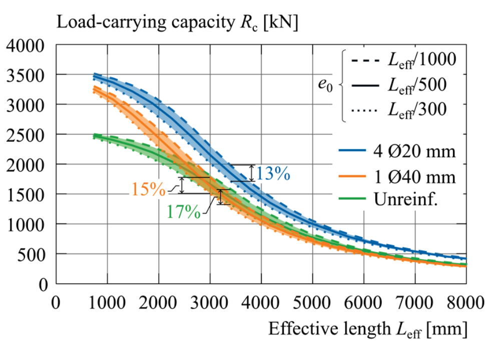

Figure 10: *Parametric study on the impact of the initial eccentricity e_0 on the load-carrying capacity of the column.*

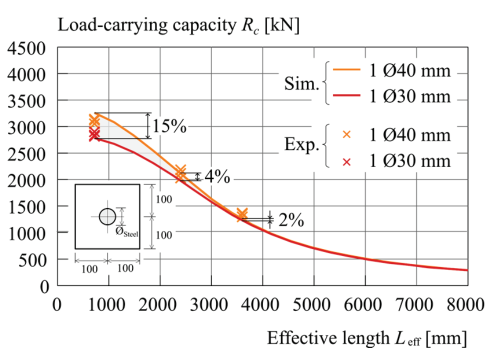

Figure 11: *Parametric study on columns with different diameter of a single central reinforcement bar.*

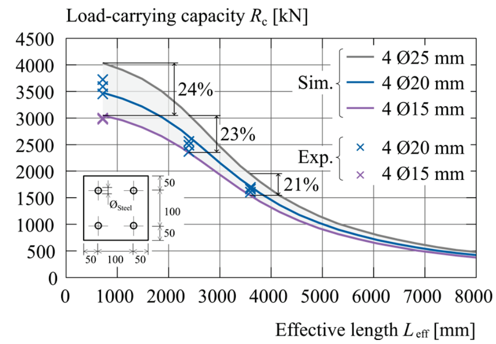

Figure 12: *Parametric study on columns with different diameters of the four corner reinforcement bars.*

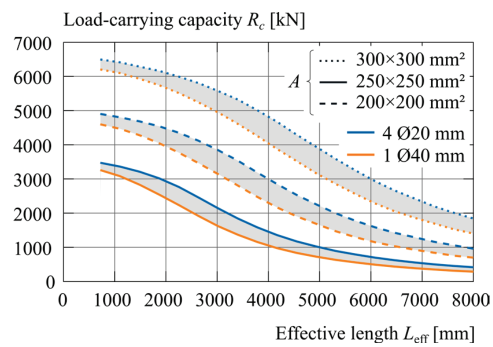

Figure 13: *Parametric study on the impact of the cross-section size on the axial load-carrying capacity of the column.*

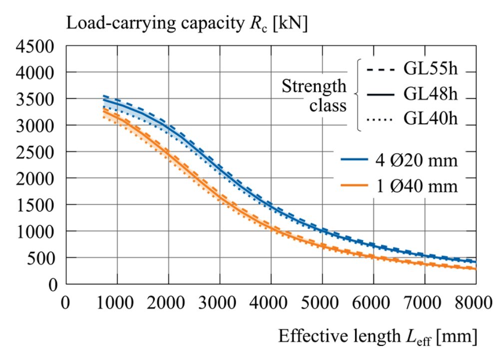

Figure 14: *Parametric study on the impact of the GLT strength class.*

All parametric studies were carried out for effective lengths of up to 8000 mm to approximately match the range of slenderness ratios covered by buckling curves in current design standards, even though such long columns are not relevant in typical structural applications. Simulated results for these lengths were not verified by experiments and should thus be regarded with caution.

4 DESIGN MODEL

4.1 UNREINFORCED COLUMNS

The design rules in EN 1995-1-1:2004 (Eurocode 5) [14] for verifying the stability of columns subjected to axial compression are based on the *effective length method*, see Formulae (2)-(5).

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$$R_{c,0} = k_c \cdot f_{c,0} \cdot A \tag{2}$$

$$k_c = \frac{1}{k + \sqrt{k^2 - \lambda_{rel}^2}} \tag{3}$$

$$\lambda_{rel} = \frac{L_{eff}}{\pi \sqrt{\frac{I}{A}}} \sqrt{\frac{f_{c,0,k}}{E_{0,05}}} \tag{4}$$

$$k = 0.5 \left(1 + \beta_c \left(\lambda_{rel} - \lambda_{rel,0}\right) + \lambda_{rel}^2\right) \tag{5}$$

where k_c = buckling factor, A = cross-sectional area, λ_{rel} = relative slenderness ratio, I = second moment of area, $f_{c,0,k}$ = characteristic value of the compressive strength parallel to the grain, $E_{0,05}$ = 5th percentile value of the modulus of elasticity parallel to the grain, β_c = straightness factor or imperfection coefficient, and $\lambda_{rel,0}$ = critical relative slenderness ratio. Both β_c and $\lambda_{rel,0}$ are material-dependent. The straightness factor β_c describes the steepness of the buckling curve while the critical relative slenderness $\lambda_{rel,0}$ defines the threshold relative slenderness above which buckling should be taken into account.

The experiments on unreinforced slender columns [6, 7], accompanied by the numerical investigations showed that the design parameters $\lambda_{rel,0} = 0.30$ and $\beta_c = 0.10$ currently specified for softwood GLT in Eurocode 5 [14] lead to an overestimation of the axial load-carrying capacity of up to 18% (L_{eff} = 2400 mm) in the case of unreinforced beech GLT columns. This deviation arises because the ratio between the compressive strength and modulus of elasticity parallel to the grain is higher for beech (1/250) [15, 16] than for softwood (1/370 to 1/420) GLT [17].

Ehrhart et al. [7] found a low scatter in compressive strength and modulus of elasticity parallel to the grain, based on tests on stocky columns according to EN 408 [8]. Values on a similar level were also reported by Westermayr et al. [16].

The calibration of buckling curves was conducted on the mean level of the material properties, including only results from test specimens of strength class GL48h. Mean values $f_{c,0,mean}$ = 60.6 MPa and $E_{c,0,mean}$ = 15.7 GPa were used, based on a more comprehensive set of test specimens with varying cross-section sizes [7]. The critical relative slenderness $\lambda_{rel,0}$ was determined based on the effective length of the stocky specimens. The straightness factor β_c was calibrated to the experimental data. A critical relative slenderness ratio of $\lambda_{rel,0} = 0.25$ and a straightness factor of $\beta_c = 0.25$ are proposed to ensure that the *effective length method* can be applied to verify the stability of beech GLT columns of strength classes GL40h, GL48h, and GL55h [7].

Figure 15 shows the load-carrying capacity as a function of the effective length according to the current and proposed buckling curves, along with the experimental and numerical results for unreinforced beech GLT of strength class GL48h.

4.2 STEEL-REINFORCED COLUMNS

At the same reinforcement ratio, corner reinforcement was shown to be much more effective in improving the mechanical properties of GLT columns than a single central rebar. Despite the more complex manufacturing process, four steel bars placed in the corners of the section are preferable for an economic design in demanding applications. A design approach was thus developed only for the corner reinforcement with four rebars of 20 mm diameter and 50 mm edge distance.

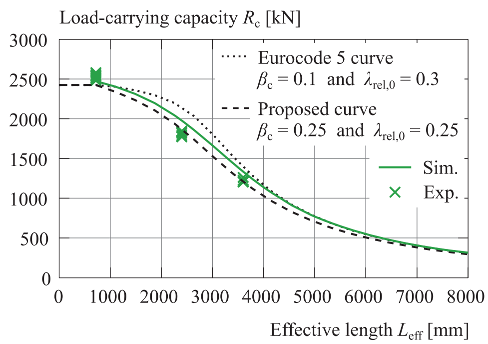

Figure 15*: Comparison of current Eurocode 5 and proposed buckling curves for unreinforced European beech GLT columns of strength class GL48h, along with the experimental and numerical results [7].*

The *effective length method* cannot be applied directly to composite sections because the load is distributed across the materials in proportion to their stiffness. An equivalent unreinforced GLT section can be determined using Formulae (6)-(8). The method is valid for rigid (i.e. glued) composite sections and assumes direct load transfer to the cross-section at the supports without any load redistribution within the section. The steel has a reinforcing function and should not yield before the timber fails.

$$n = \frac{E_s}{E_{c,0,mean}} \tag{6}$$

$$A_c = A_{GLT} + (n - 1) \cdot A_s \tag{7}$$

$$I_c = I_{GLT} + (n - 1) \cdot I_s \tag{8}$$

where n = scaling factor to account for stiffness variation, A_c = cross-sectional area of an equivalent timber section, A_{GLT} = gross cross-sectional area of the timber (including the holes for the steel bars), A_s = cross-sectional area of the steel, I_c = second moment of area of an equivalent timber section, I_{GLT} = gross second moment of area of the timber (including the holes for the steel bars), and I_s = second moment of area of the steel cross-section.

The *effective length method* (Formulae (2)-(5)) can be adapted for composite sections as given in Formulae (9)-(10). As for unreinforced beech GLT, the buckling curve was calibrated based on the mean experimental results for GL48h, i.e. $f_{c,0,mean}$ = 60.6 MPa and $E_{c,0,mean}$ = 15.7 GPa [7].

$$\lambda_{rel} = \frac{L_{eff}}{\pi \sqrt{\frac{I_c}{A_c}}} \sqrt{\frac{f_{c,0,mean}}{E_{c,0,mean}}} \tag{9}$$

$$R_{c,0} = k_c \cdot f_{c,0,mean} \cdot A_c \tag{10}$$

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A design based on the parameters currently specified by Eurocode 5 leads to an overestimation of the experimental buckling resistance by up to 20% (L_{eff} = 2400 mm) in the case of the corner-reinforced beech GLT columns, as shown in Figure 16. The same figure shows the buckling curve fitted for corner-reinforced European beech of strength class GL48h with four ST900/1100 rebars of 20 mm diameter and 50 mm edge distance. A least squares fitting to the experimental data points resulted in a straightness factor $\beta_c = 0.255$ and a critical relative slenderness ratio $\lambda_{rel,0} = 0.261$. Since the deviation from the buckling curve proposed for unreinforced European beech GLT is small, the same curve ($\beta_c = 0.25$ and $\lambda_{rel,0} = 0.25$) is proposed for corner-reinforced beech GLT. The resulting curve of the proposed buckling factor k_c is given in Figure 17.

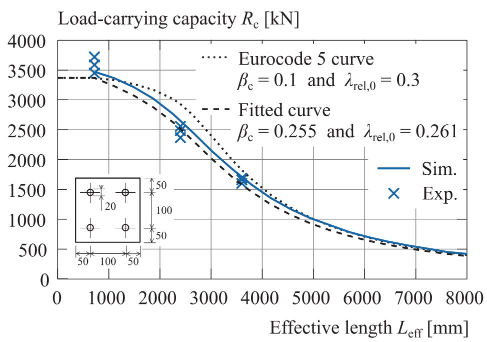

Figure 16*: Comparison of current Eurocode 5 and fitted buckling curves for corner-reinforced beech GLT columns of strength class GL48h, along with the experimental and numerical results.*

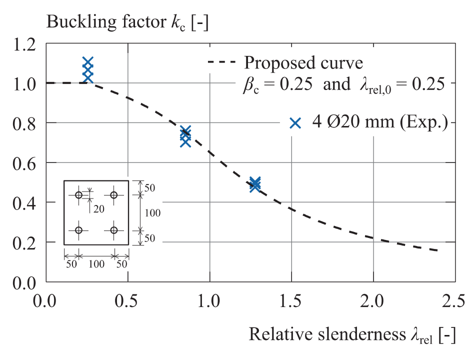

Figure 17*: Proposed buckling curve for corner-reinforced beech GLT columns of 200×200 mm² and strength class GL48h with four ST900/1100 rebars of 20 mm diameter and 50 mm edge distance, along with the experimental results.*

5 CONCLUSIONS

The full-scale experiments on unreinforced and steel-reinforced European beech GLT columns revealed their immense potential for application in highly demanding residential, office, and industrial projects.

Material properties of European beech GLT were identified from the experiments on unreinforced columns. Different layouts of steel reinforcement were investigated in compression and buckling tests. A finite-element model was developed and shown to reliably predict the buckling behaviour of these high-strength columns and parametric studies were performed to investigate the main factors influencing their load-carrying capacity. The ultimate failure load was found to be mainly dependent on the column slenderness, the arrangement and diameter of the steel bars, the section size, and the initial deformed shape (eccentricity e_0). The strength class (GL40h, GL48h, or GL55h) was found to have very little impact.

The steel reinforcement using profiled Swiss-GEWI bars of grades ST900/1100 and ST950/1050 led to an increase of both the compressive strength and the modulus of elasticity parallel to the grain of European beech GLT. A central ST950/1050 rebar of 40 mm diameter was found to increase the load-carrying capacity of beech GLT columns by 20% for low ($\lambda_{rel} < 0.5$) to medium ($\lambda_{rel} \approx 1.0$) slenderness and by 10% for slender columns ($\lambda_{rel} \approx 1.5$).

At the same reinforcement ratio, corner reinforcement (four bars of 20 mm diameter, 50 mm edge distance, and grade ST900/1100) offers a much more pronounced gain of about 40% in the load-carrying capacity across all slenderness ratios relevant in structural applications. This is because the eccentric arrangement contributes to the bending stiffness of the column. Though requiring more manufacturing steps, corner reinforcement thus enables highly demanding structures such as high-rise buildings to be built using timber columns while avoiding excessively large cross-section sizes that reduce available floor space and obstruct window views. Sufficient timber cover must be provided for all steel bars to ensure fire safety and prevent buckling of the steel bars.

Current Eurocode 5 [14] design parameters were found to overestimate the load-carrying capacity of both unreinforced and reinforced beech GLT columns. Based on the experimental and numerical results, the *effective length method* as applied by Eurocode 5 was adapted for beech GLT. Proposed design parameters are $\lambda_{rel,0} = 0.25$ and $\beta_c = 0.25$ for both unreinforced and corner-reinforced European beech GLT columns. The buckling curve for corner-reinforced beech GLT was developed for square cross-sections of 200 mm width and strength class GL48h, reinforced with four ST900/1100 bars of 20 mm diameter, which were placed in the corners of the section with an edge distance of 50 mm.

Further experimental investigations and numerical simulations with probabilistic material modelling should be carried out to verify the presented findings while taking into account the variability of the mechanical properties. Research into the manufacturing process of corner-reinforced GLT could reduce its complexity and increase the economic viability of reinforced GLT columns, which offer a huge potential regarding moment resisting and tension connections, thereby enabling a reduction of the effective length and the use of columns to transfer tensile forces in alternative loads paths following element-removal scenarios.

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REFERENCES

[1] FOEN: Jahrbuch Wald und Holz. Swiss Federal Office for the Environment, Bern, Switzerland, 2019.

[2] Mpidi Bita H., Huber J., Palma P., Tannert T.: Prevention of disproportionate collapse for multi-story mass timber buildings: review of current practices and recent research. *Journal of Structural Engineering*, 148(7):04022079, 2022.

[3] Woschitz R., Zotter J.: Holzhochhaus HoHo Wien – Das Tragwerkskonzept (High-rise timber building HoHo Vienna – the structural concept). *Österreichische Ingenieur- und Architekten-Zeitschrift*, 162(1-12):69-73, 2017.

[4] Ehrhart T., Fink G., Steiger R., Frangi A.: Strength grading of European beech lamellas for the production of GLT and CLT. In *Proceedings of INTER-Meeting 49*, paper 49-5-1, 2015.

[5] Lehmann M., Clerc G., Lehringer C., Strahm T., Volkmer T.: Investigation of the bond quality and the finger joint strength of beech glulam. In *Proceedings of World Conference on Timber Engineering*, 2018.

[6] Ehrhart T., Steiger R., Lehmann M., Frangi A.: European beech (*Fagus sylvatica* L.) glued laminated timber: lamination strength grading, production and mechanical properties. *European Journal of Wood and Wood Products,* 78(5):971-984, 2020.

[7] Ehrhart T., Steiger R., Palma P., Gehri E., Frangi A.: Glulam columns made of European beech timber: compressive strength and stiffness parallel to the grain, buckling resistance and adaptation of the effective-length method according to Eurocode 5. *Materials and Structures,* 53(4):91, 2020.

[8] EN 408:2012. Timber structures – Structural timber and glued laminated timber – Determination of some physical and mechanical properties. CEN European Committee for Standardization, Brussels.

[9] von Tetmajer L.: Zur Frage der Knickfestigkeit der Bauhölzer. *Schweizerische Bauzeitung,* 17(11/12):110-113, 1888.

[10] Zhu M. J., McKenna F., Scott M. H.: OpenSeesPy: Python library for the OpenSees finite element framework. *Softwarex,* 7:6-11, 2018.

[11] Python (version 3.9), 2020. Python Software Foundation.

[12] McKenna F., Scott M. H., Fenves G. L.: Nonlinear Finite-Element Analysis Software Architecture Using Object Composition. *Journal of Computing in Civil Engineering,* 24(1):95-107, 2010.

[13] Glos P., Denzler J. K., Linsenmann P. W.: Strength and stiffness behaviour of beech laminations for high strength glulam. In *Proceedings of CIB-W18 Meeting 37*, paper 37-6-3, 2004.

[14] EN 1995-1-1:2004. Design of timber structures–part 1-1: General – Common rules and rules for buildings. CEN European Committee for Standardization, Brussels.

[15] Ehrhart T., Steiger R., Palma P., Frangi A.: Mechanical properties of European beech (*Fagus sylvatica* L.) glued laminated timber. In *Proceedings of INTER-Meeting 51*, paper 51-12-4, 2018.

[16] Westermayr M., Stapel P., Van de Kuilen J.-W. G.: Tensile and compression strength of small cross section beech glulam members. In *Proceedings of INTER-Meeting 51*, paper 51-12-2, 2018.

[17] Blaß H. J.: Tragfähigkeit von Druckstäben aus Brettschichtholz unter Berücksichtigung streuender Einflussgrössen. Dissertation, University Fridericiana Karlsruhe, Germany, 1987.

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