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ASSESSING THE EFFECTIVENESS OF NONLINEAR SPRINGS FOR MODELLING SHEAR WALL CONNECTORS IN CROSS-LAMINATED TIMBER

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Abstract: *Engineered wood products have been developed and introduced to cover traditional wood's weaknesses, such as having a limited load-bearing capacity and being prone to environmental damage. In recent decades, Cross-Laminated Timber (CLT) panels have been used as lateral and gravity load resistance systems in timber and hybrid buildings as a sustainable alternative material for steel and concrete. Experimental test results indicate a relatively rigid behaviour for CLT panels, while connectors are responsible for most of the system's ductility and energy dissipation. Applying the capacity-based design method can ensure considering the connectors' ability to control the systems' behaviour and the ductile failure modes. Using the right backbone nonlinear spring for complex connectors in tension and shear is still challenging for researchers and designers, therefore, this paper compares the ability of three different numerical nonlinear springs. The rigid in-plane behaviour of CLT panels is modelled using linear shell elements in OpenSees, while connectors are defined utilising Pinching4, SAWS and Wood-Spring zero-length elements. The numerical models are validated against test results on hold-downs and steel-brackets in both axial and shear directions. The results have been compared after developing the model for CLT shear walls and investigating their behaviour under axial load and lateral cyclic displacement. According to the results from the numerical analyses, the SAWS model is the most straightforward spring and considers stiffness degradation. Besides stiffness degradation, strength degradation in unloading and reloading branches can be considered by Pinching4. Nevertheless, the Wood-Spring is the most detailed among the studied springs, which can consider the shear-axial coupling of connection springs and panel-to-panel friction effects. However, it has a relatively high computational effort. Although all three multi-spring models showed acceptable accuracy at both connector and wall levels, the suitable spring type should be chosen according to the desired model's accuracy and analysis time.*

Keywords; *Cross-Laminated Timber, CLT connector, shear wall, cyclic test, panel sliding, SAWS, Pinching4, Wood-Spring*

Introduction

The recent advancements in engineering timber open a new horizon to build sustainable, green and economic structures. The high strength-to-weight ratio of wood improves the performance of wooden buildings in earthquakes. Moreover, the low CO_2 emission from harvest to be used in buildings as structural elements makes it a green and eco-friendly alternative for other materials. However, relatively weak compressive strength perpendicular to grain, dimension limitations and natural defects have limited the conventional wood application to low-rise buildings. Engineered Wood Products (EWP)s have been invented to cover the conventional aforementioned wood's weaknesses and consider the timber's potential as gravity and load-bearing systems. EWPs are innovative wooden products purposely developed to improve timber's mechanical properties. There are several EWPs, such as Cross-Laminated Timber (CLT), Glued-Laminated Timber (GLT), Laminated Veneer Lumber (LVL), Laminated Strand Lumber (LSL), Parallel Strand Lumber (PSL), etc. (Green & Taggart, 2020).

In buildings, CLT shear walls are a structural system that uses CLT panels to resist lateral forces, such as wind or seismic loads. CLT shear walls consist of multiple layers of wood panels glued perpendicularly to create a thick, solid wall. This construction method provides excellent strength

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and stiffness in horizontal and vertical directions, making it an ideal solution for seismic and wind resistance.

CLT shear walls can be used in low-rise and mid-rise buildings and have been shown to perform well in earthquake-prone regions.

According to Sandoli et al. (2021), CLT construction is a robust alternative to heavyweight construction, offering benefits regarding seismic resistance and environmental sustainability. Despite the permanent damage to timber panels in connection zones, the material's lightness and the mechanical connections' satisfactory dissipative response provide an excellent seismic response to multi-story CLT buildings. Having a low carbon footprint and renewable resources make CLT construction a sustainable solution for environmental issues of future construction. Yu et al. (2017) have mentioned the significant impact of embodied carbon on construction materials in Australia. They concluded that timber materials could improve sustainability in the construction industry.

Numerous studies have investigated various aspects of CLT shear walls, such as their design, performance, and behaviour under diverse loading conditions to assess CLT shear walls' strength, stiffness, ductility, and seismic resistance. The SOFIE project, an extensive research initiative in the timber construction field, was coordinated and conducted by CNR-IVALSA (Trees and Timber Institute – Italian National Research Council) (Ceccotti, 2008). This project started in 2005 and aimed to investigate the performance of CLT systems under seismic actions. The project covered testing CLT assemblies at different levels, from connectors' behaviour under axial (tension) and shear load to full-scale multi-storey buildings' response to ground motions. Several single and coupled CLT shear walls with various connection layouts were tested under cyclic loads as a part of the SOFIE project (Ceccotti, et al., 2006a; Ceccotti, et al., 2006b; Ceccotti et al., 2013). Gavric et al. (2015) investigated the impact of connector design and different anchoring systems on the behaviour of three sets of single and coupled CLT shear walls through cyclic testing according to EN 12512 standard. They evaluated the stiffness, strength, ductility, strength degradation and equivalent damping ratios. The results showed that connector design significantly affects the overall structural behaviour of the system, with local failure of connectors due to force and deformation concentration in the connection parts. In order to prevent residual displacement of walls during seismic events, the authors recommend that angle brackets which experience shear forces remain elastic while hold-downs and angle brackets experiencing tension undergo plasticisation. This suggestion aims to avoid the so-called sliding kinematic failure mechanism and prefers the rocking kinematic failure mechanism.

As a part of an experimental campaign at the LEDA Research Centre of the University of Enna "Kore", Ruggeri et al. (2023) studied the behaviour of CLT shear walls perpendicularly connected using screwed wall-to-wall connections. The Single Wall and Perpendicular Wall (SW+PW) configuration had significantly better structural performance in terms of lateral and deformation capacity than the SW configuration due to the properties of the wall-to-wall connections and hold downs. Ma and Dai (2023) evaluated the lateral performance of conventional CLT shear walls by establishing equivalent decomposed wall models. The decomposed model with equivalent springs was developed using the calibrated springs and shell elements based on experimental results. The efficacy of the models was evaluated using case studies and seismic analysis of a full-scale building, revealing that the equivalent decomposed model had a better prediction in terms of the equivalent viscous damping ratio. Furthermore, the developed models provided a reliable estimation of the wall hysteresis behaviour.

The connectors' behaviour is crucial in accurately predicting the overall behaviour of CLT shear walls, as they have a significant role in transferring forces between panels. Izzi et al. (2018) experimentally studied the behaviour of typical wall-to-floor CLT connections. Using hold-downs and angle brackets, the researchers subjected the connectors to monotonic and reversed cyclic loadings. Subsequently, they proposed a numerical model to predict the connectors' mechanical behaviour and failure mode. In addition, the study explored the effect of various factors, such as plastic hinge formation and the axial-shear coupling effect. The results showed that considering the nail group effect improves the model's accuracy. In other experimental studies, Liu and Lam (2018, 2019) investigated the effects of hold-downs and angle brackets on the coupling behaviour of two-phase loadings. The results indicated that the presence of axial force led to a reduction in stiffness and strength capacity in the shear direction. The angle bracket was found to exhibit a more significant capacity reduction compared to the hold-down. These findings suggest that using angle brackets in two-phase loadings may reduce structural performance, highlighting the importance of careful consideration in their selection and application. Shen et al. (2021) assessed the uniaxial cyclic behaviours of standard angle brackets through comprehensive testing and numerical modelling. They compared five analysis methods to determine yield points and identified a universal approach. According to Ceccotti (2008), CLT panels exhibit a high degree of stiffness in both the in-plane and out-of-plane directions, which results in rigid behaviour. Moreover, CLT panels' shear and flexural deformations are negligible compared to the connectors' deformation if the CLT panels do not have openings. Consequently, connectors are responsible for the system's energy dissipation and ductility (Gavric & Popovski, 2014).

With the growing demand for sustainable and resilient building systems, CLT shear walls are gaining increasing attention as a viable solution for current construction projects; However, the nonlinear behaviour of CLT shear walls under cyclic loading is not well understood, and accurate simulations of this behaviour are necessary to design seismic-resistant structures. This study compares the performance of different nonlinear springs in simulating the nonlinear behaviour of CLT shear walls.

Methodology

The *SAWS* and *Pinching4* springs from the OpenSees library are applied for connectors' calibration (McKenna, 2011). The third applied spring is the *Wood-Spring*, which was purposely developed for the CLT connector by Rinaldin et al. (2013). The *Wood-Spring* can be used in ABAQUS and OpenSees, utilising the corresponding external subroutines. Researchers have used this zero-length element to model the CLT connectors in shear walls and buildings (Gavric, 2013; Rinaldin & Fragiacomo, 2016; Tamagnone, 2019). The utilised nonlinear springs have different abilities in considering the mechanical properties of connectors, such as the stiffness and strength degradation in reloading and unloading branches, friction and shear-axial coupling effects.

Folz and Filiatrault (2001) Introduced the *SAWS* model displayed in Figure 1-a. This model's symmetrical hysteretic backbone curve can be characterised using ten parameters. The slopes of different branches are determined by multiplying the initial stiffness by R1, R2, R3 and R4. R1 and R2 have positive and negative values to describe the hardening and softening parts of the curve, respectively. The envelope curve is defined using an exponential function and a linear descent line. Two strength intercepts, F0 and F1, regulate the asymmetric lines to the envelope curve. The stiffness degradation (KP) is based on the previous loading history, which can be controlled using α and β parameters.

In 2012, Mitra implemented the *Pinching4* model into OpenSees, which utilises eight breaking point coordinates (P1-P4 and N1-N4) to define the backbone curve and four points (P5-P6 and N5-N6) to control the behaviour of the unloading-reloading branches, as depicted in Figure 1-b. Furthermore, the model incorporates fifteen parameters to govern stiffness and strength degradation under cyclic loading. This model assesses the damaging impact by considering the time-history displacement and accumulative energy.

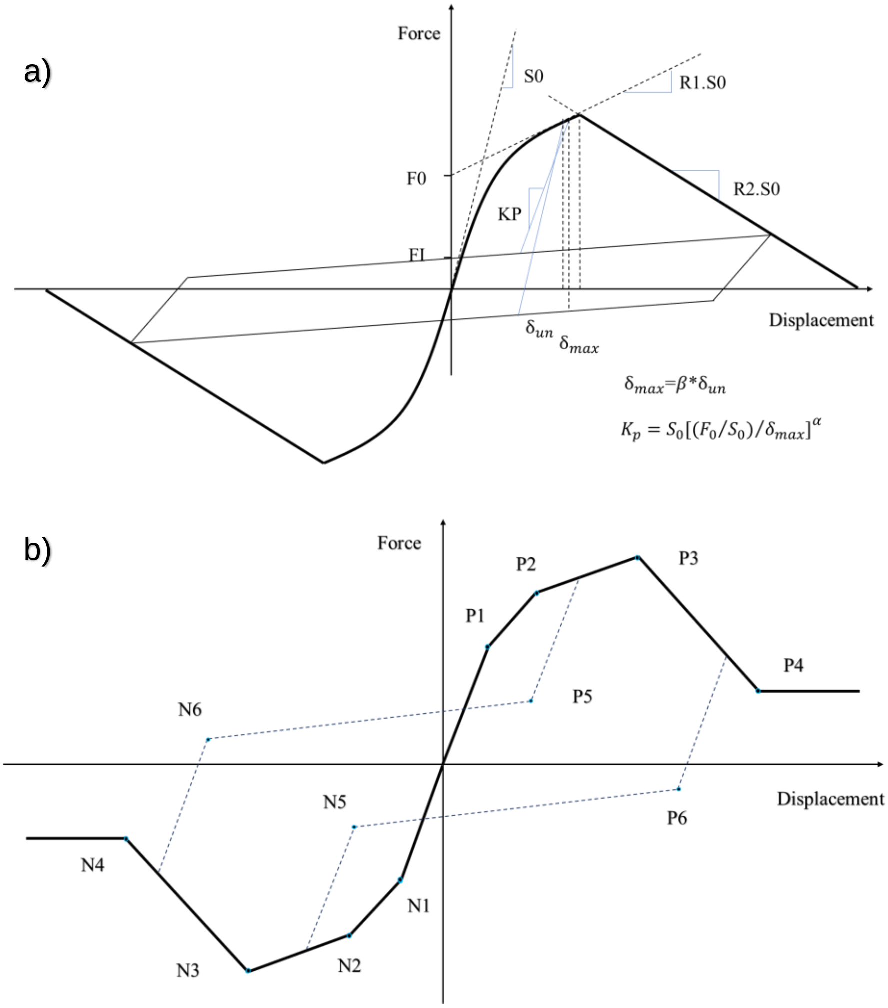

Figure 1. a) SAWS and b) Pinching4 model backbone curves

The *Wood-Spring* model comprises sixteen branches, where the backbone consists of one elastic and two plastic branches (hardening and softening), defined by parameters such as F_y, F_U and D_U, as well as the slope of each branch. As depicted in Figure 2, each branch is assigned a numerical identifier that is elaborated upon in the corresponding reference (Rinaldin et al., 2013). The relevant parameters govern the reloading and unloading path specifications. The *Wood-Spring* model, in addition to capturing the cyclic behaviour, also accounts for the shear-axial coupling effect and friction between panels. The model incorporates maximum displacement to capture stiffness degradation, while strength degradation depends on the energy dissipated during the load history. (Rinaldin et al., 2013). Rinaldin (2013) developed a program called *so.ph.i* for calibrating the *Wood-Spring* against experimental test results.

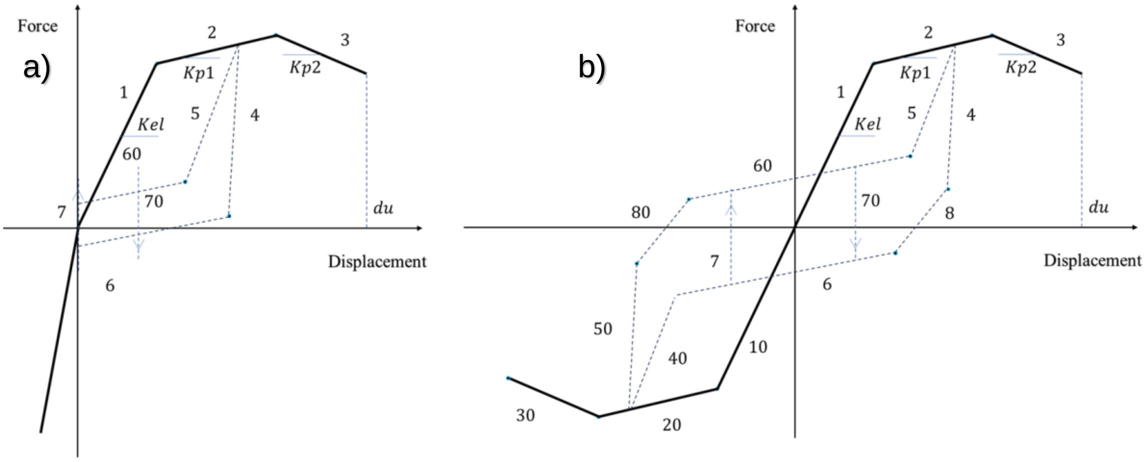

Figure 2. Wood-Spring backbone curve in a) axial b) shear directions

The rigid linear in-plane behaviour of the CLT panel is modelled using the shell elements (Gavric et al., 2012). The mechanical properties of the panels depend on the number and directions of the lamellas. This element considers an idealised isotropic linear behaviour for CLT panels. The homogenised Young's modulus is determined by E_m, n and t are the number and thickness of the layers perpendicular (90°), and parallel (0°) to the grain directions, the total thickness of the CLT panel is described by t_{tot} presented in Equation 1 (Blass & Fellmoser, 2004). Gap elements define the contact between the CLT panel and the foundation using a strong compression resistance and no tensile capacity.

$$\mathrm{E_m} = \frac{\mathrm{n_0 t_0 E_0 + n_{90} t_{90} E_{90}}}{\mathrm{t_{tot}}} \qquad (1)$$

Results and discussion

To evaluate the performance of different nonlinear springs, FE models were developed within OpenSees software, which emulated the conduct of shear walls subjected to experimental testing by Gavric (2013). The CLT panels were connected to the rigid foundation utilising hold-downs and angle brackets arranged in varying configurations. Then, the specimens were subjected to a consistent axial load and lateral reversed cyclic displacement. Nonlinear springs have been calibrated against the isolated connectors (hold-down and angle bracket) test results in axial and shear directions before being used in the numerical models. Figure 3 illustrates the tested connectors in axial direction. The tested hold-down is HTT22 type and connected to the foundation via bolts sized 16 mm while being connected to the panel using 12 annular ringed nails sized 4X60 mm. The angle bracket is of type BMF90X116X48X3 mm and connected to the panel using 11 annular ringed nails sized 4X60 mm.

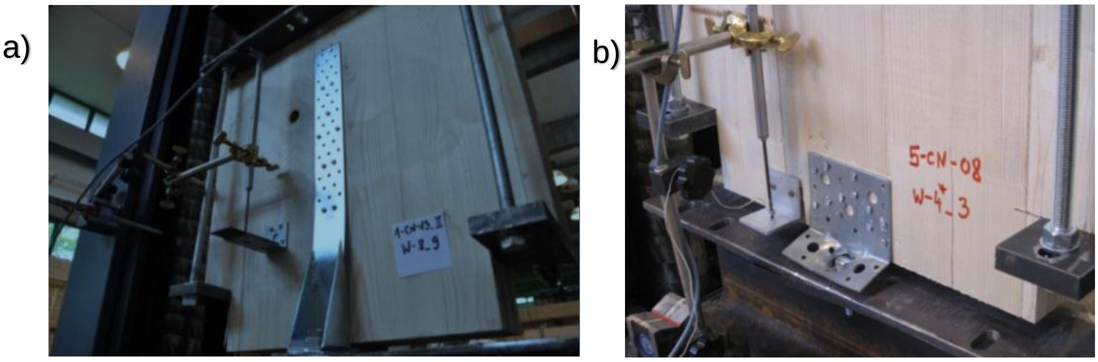

Figure 3. a) Hold-down and b) angle bracket axial (tension) test (Gavric, 2013)

The comparison of spring calibration outcomes for the angle bracket in shear, using *SAWS*, *Pinching4*, and *Wood-Spring*, is depicted in Figure 4. As it can be seen, all three springs can fit the numerical and test results in terms of stiffness in elastic and after-yield zones as well as the peak load; however, the efficacy of calibrating the stiffness and strength degradations in reloading and unloading branches is reliant on the capabilities of the springs. Figure 5 illustrates the springs' calibration for the behaviour of hold-down in the axial direction.

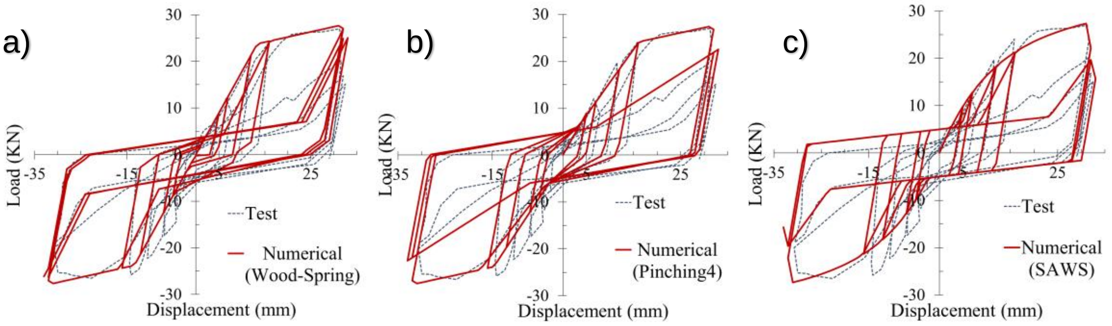

Figure 4. Angle bracket in the shear direction using a) Wood-Spring, b) Pinching4, c)SAWS

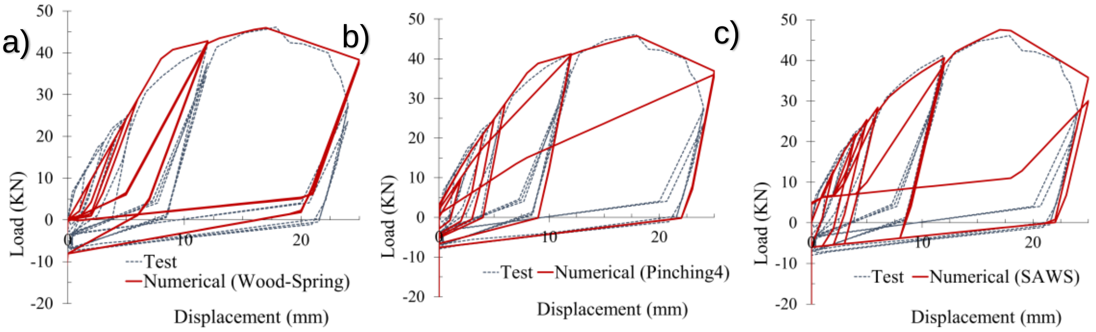

Figure 5. Hold-down in the axial direction using a) Wood-Spring, b) Pinching4, c) SAWS

Recent dynamic testing of CLT walls has shown that the behaviour of hold-down connections may not be limited to tension-only loading and angle brackets only in shear direction, as it was previously commonly assumed. Instead, significant lateral and uplift displacement has been observed, leading to a tension-shear coupled action in these connections. This finding suggests that the behaviour of angle bracket and hold-down connections may be more complex than previously assumed and highlights the importance of considering the potential for coupled loading effects in the design of CLT structures (Pozza et al., 2018). The *Wood-Spring* is the only zero-length element among the applied springs which is able to couple the shear and axial behaviour for CLT connectors as *SAWS* and *Pinching4* consider independent behaviours in axial and shear directions. Another benefit of the *Wood-Spring* is the capacity to account for panel-to-panel friction.

In contrast, when utilising the *SAWS* and *Pinching4* zero-length elements, extra springs are required to accommodate frictional forces. In friction springs, it is essential to take into account the yield point with respect to the static friction coefficient and the normal panel-to-panel load. The initial stiffness of the spring should also be of an adequate magnitude to ensure that no displacement occurs until the force surpasses the static friction threshold. It should be noted that using too large friction stiffnesses might cause convergence problems in the numerical model. The *Wood-Spring* has been purposely developed to model the behaviour of connectors in a highly detailed manner. This model considers various mechanical parameters essential for accurately capturing the behaviour of CLT connectors, such as pinching, stiffness, and strength degradation, as well as friction and axial-shear coupling effects. However, due to its complex nature and consideration of the above factors, the computational effort required to run the *Wood-Spring* model is relatively high compared to *SAWS* and *Pinching4*. This can result in longer processing times, particularly when dealing with complex models or large-scale simulations.

On the other hand, the *SAWS* and *Pinching4* spring models are relatively straightforward to implement as they are available within the OpenSees library. In contrast, applying the *Wood-Spring* requires the application of an external subroutine to be utilised in both OpenSees and

ABAQUS software packages. Despite their relative simplicity compared to the *Wood-Spring* model, *SAWS* and *Pinching4* offer the advantage of much faster processing times. *SAWS*, for example, is capable of modelling stiffness degradation in both reloading and unloading branches, while *Pinching4* can account for both strength and stiffness degradation. Therefore, these models can reasonably represent the behaviour of connectors while offering a practical and computationally efficient alternative to the *Wood-Spring* model.

Figure 6 illustrates Wall 1.1, tested by Gavric (2013). This configuration is connected to the foundation utilising two angle brackets in the middle and two hold-downs at the wall corners. This single shear wall was subjected to reversed cyclic displacement while a constant axial load of 18.5 KN/m was applied to the top of the wall. Wall's specifications are presented in Table 1.

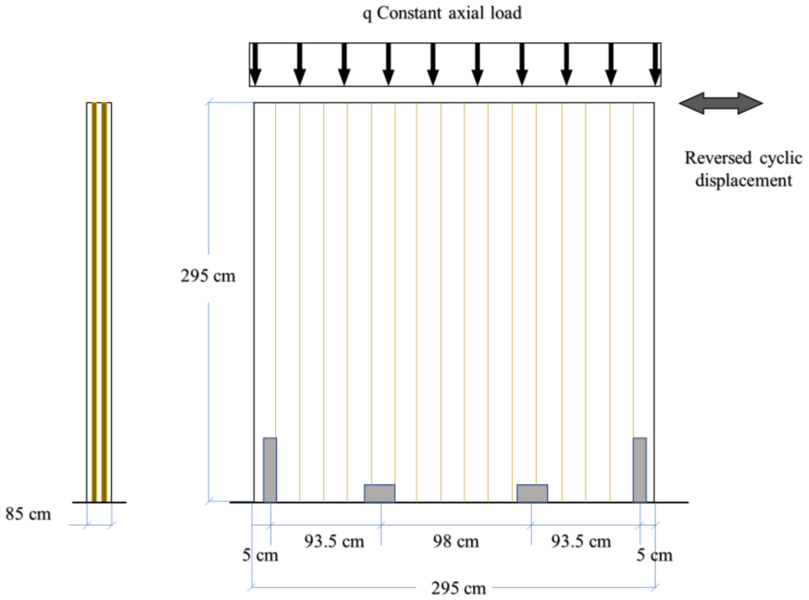

Figure 6. Wall 1.1 configuration (Gavric, 2013)

Connector Type	Connector name	No. of connectors	Bolt	Nails type	No. of nails
Hold-Down	Simpson HTT22	2	$\emptyset$ 16	$\emptyset$ $4X60$ Annual ringed	12
Angle-Bracket	BMF 90x48x3x16	2	$\emptyset$ 16	$\emptyset$ $4X60$ Annual ringed	11

Table 1. Specifications of the tested wall (Gavric, 2013)

The calibrated springs have been used to represent the behaviour of connectors in axial and shear directions. The in-plane behaviour of CLT panels is modelled using linear shell elements, and the homogenised modulus of elasticity is determined based on the number, thickness, and orientation of the individual layers comprising the panel, as described by Equation 1.

Figure 7 compares the test and numerical results of wall modelling using *SAWS*, *Wood-Spring*, and *Pinching4* to evaluate the springs' performance in predicting the wall's global behaviour. The global behaviour of the wall is characterized by the top horizontal displacement versus the base shear, wherein the displacement is a combined effect of uplift and sliding, while the in-plane CLT panel deformation is almost negligible. The results show a good match between the test and the numerical results regarding initial stiffness, maximum strength and ultimate displacement for all three applied springs (Table 2).

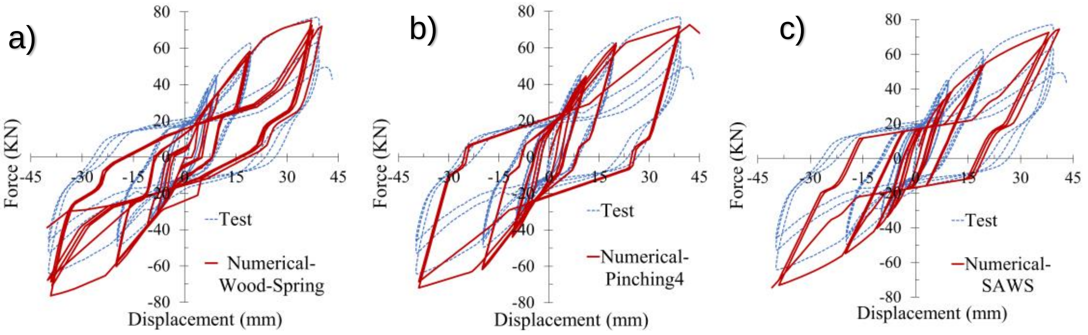

Figure 7. Comparing the experimental and numerical global behaviour of Wall 1.1 using a) Wood-Spring, b) Pinching4 and c) SAWS model for CLT connectors

According to the test results, Wall 1.1 failed primarily due to the shear strength of the angle brackets being exceeded. As a result, the majority of the wall's deformation resulted from the panel sliding, indicating that the failure mode was the shear type (Gavric, 2013). The observed failure mode of the wall highlights the importance of investigating the wall's sliding behaviour. So, the test and numerical wall sliding were compared. Figure 8 depicts the corner of the wall sliding against the base shear, employing different springs to model the connectors.

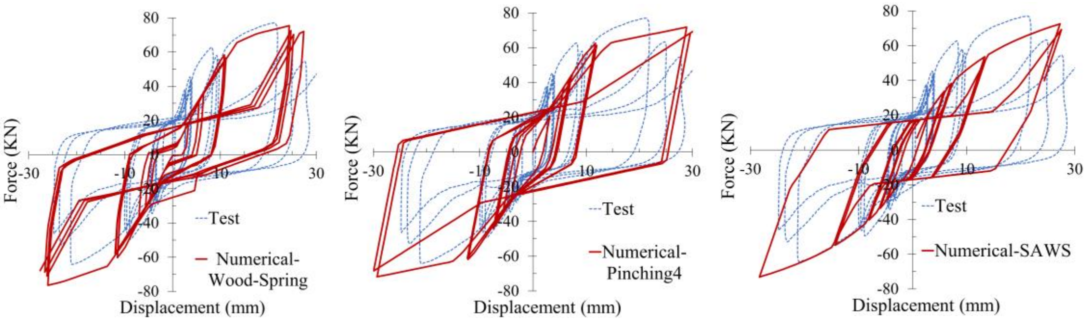

Figure 8. Comparing the experimental and numerical sliding behaviour of Wall 1.1 using a) Wood-Spring, b) Pinching4 and c) SAWS model for CLT connectors

Table 2 presents the wall's global and sliding test results as well as the numerical outcomes in terms of elastic stiffness, maximum strength, and ultimate displacement. In Table 2, the values presented in parentheses express the numerical-test error.

	Parameter	**Test**	**Wood-Spring**	**Pinching4**	**SAWS**
Global	Elastic stiffness (KN/mm)	5.66	5.0 (11.6%)	5.5 (2.8%)	5.1 (9.8%)
	Maximum strength (KN)	77.08	76.4 (0.9%)	71.3 (7.4%)	74.5 (3.3%)
	Ultimate displacement (mm)	38.54	37.1 (3.7%)	43.0 (11.5%)	41.0 (6.3%)
Sliding	Elastic stiffness (KN/mm)	14.30	11.2 (21%)	12.0 (16%)	12.3 (13.9%)
	Maximum strength (KN)	77.08	76.4 (0.9%)	72.6 (5.8%)	74.5 (3.3%)
	Ultimate displacement (mm)	22.00	24.6 (11.8%)	28.0 (27%)	28.0 (27%)

Table 2. Wall's global and sliding test-numerical data

Regarding the wall's global behaviour, the *Pinching4* provided the most precise prediction of the elastic stiffness, with only a 2.8 per cent underestimation. In addition, the *Wood-Spring* exhibited the lowest test-numerical difference in terms of maximum strength and ultimate displacement corresponding to the maximum strength, with 0.9 and 3.7 per cent variations, respectively.

Despite underestimating the elastic shear stiffness in all three comparisons, the wall's yielding in shear is observable in Figure 8. The shear yield point around the displacement of 10 mm, which corresponds to the angle bracket yield point, is predicted by the *Wood-Spring* and *Pinching4* models. However, the yield point predicted by the *SAWS* model is not as evident as in the other two models. This proves that sliding is the dominant behavioural model of the tested wall. The *Wood-Spring* model best predicted the ultimate strength and corresponding displacement, with only 0.9 and 11 per cent difference between the test and numerical data (Table 2). Owing to the consideration of the shear-axial coupling effect, the *Wood-Spring* model outperformed the other two models in predicting the maximum sliding of the wall, while ignoring the coupling effect in *SAWS* and *Pinching4* has resulted in noticeable sliding overestimation.

Conclusions

This paper has evaluated the performance of three nonlinear springs, *SAWS*, *Pinching4*, and *Wood-Spring*, in modelling the axial and shear behaviour of CLT connectors. Several FE models were developed using OpenSees to investigate the cyclic behaviour of a CLT shear wall using the above-mentioned springs. The comparison between the test and numerical results revealed the ability of all three models in the wall's global and sliding behaviour prediction with acceptable accuracy. However, considering the shear-axial coupling effect using the *Wood-Spring* resulted in the most precise prediction among other models, with a 0.9 and 3.7 per cent underestimation in the wall's maximum strength and corresponding displacement, respectively. Furthermore, the *SAWS* and *Pinching4* models have shown exemplary performance in predicting the global behaviour of the wall. However, ignoring the coupling effect has resulted in considerable sliding overestimation.

In conclusion, *Wood-Spring* has the highest computational effort due to its complexity, resulting in the highest running time compared to the other two models. Therefore, the choice of the spring should be based on the model's complexity and required precision. *SAWS* and *Pinching4* can be used for models in which the global behaviour of the structural members is being studied, while *Wood-Spring* is suitable for studying both global and local behavioural modes of panels.

These results are only valid for a single wall, and adding a 3D effect of floor-to-wall and wall-to-wall connection may significantly change the conclusion.

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