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RACKING RESISTANCE OF CLT PANELS MANUFACTURED FROM C16 GRADE TIMBER

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ABSTRACT: This study examines the typical lateral/racking resistance of cross-laminated timber (CLT). The use of CLT in construction has grown in many regions of the world, not only due to its impressive structural properties but also the improved environmental performance achieved for this mass-timber system compared to more traditional construction materials. In Europe, CLT is primarily manufactured with C24 grade timber or a combination of C24 with small proportions of C16 grade material in internal layers. In many parts of the world, the supply of C24 timber is limited and it is important to establish the structural properties of CLT manufactured with lower-grade material. This study presents a numerical model developed to predict the load-displacement behaviour and structural racking resistance of CLT panels and will inform the experimental testing of CLT panels manufactured from C16 grade material in combination with typical connections utilised in structural mass timber systems.

KEYWORDS: Connections; Cross Laminated Timber (CLT); Engineered Wood Products (EWPs); Racking resistance; Sitka spruce

1 INTRODUCTION

In recent years, there has been a significant rise in the number of advanced engineered wood products (EWPs) coming to market. Cross-laminated timber (CLT) presented in Figure 1 is one of the most significant advances in EWP technology and has been responsible for the rise in timber structures and has the potential to significantly reduce the environmental footprint of the built environment in the fight against climate change [1].

Figure 1: *Typical cross laminated timber (CLT) panel.*

In Europe, this product is primarily manufactured using C24 grade [2] material with limited quantities of lower-grade material being used [3]. As the market interest in mass-timber increases, the demand for natural timber resources will benefit from increased knowledge of the structural properties of CLT manufactured from lower-grade materials. To date, there has been a significant amount of work carried out to establish the manufacturing properties of CLT from lower-grade materials [4–6] however, there is limited information on the racking resistance of C16-grade CLT panels [7]. In this study, a numerical model is developed to examine the racking resistance of CLT panels subjected to in-plane racking loads. The numerical model is validated against experimental tests on C24 grade CLT carried out by Hughes [8]. In the study by Hughes [8], a 5-layer CLT panel manufactured from C24 grade material was subjected to a racking test.

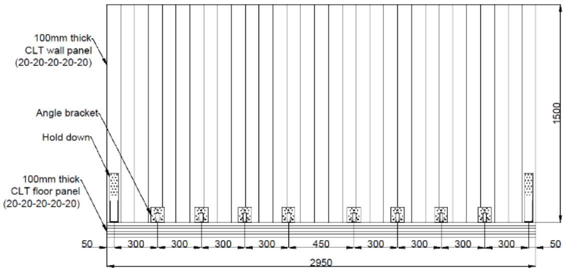

Figure 2: *CLT test set-up* [8] *(Dims. in mm).*

The CLT wall panel measured 1.5 m in height and 2.95 m in length as shown in Figure 2. The panel was connected to a CLT floor with a series of typical brackets (hold downs and angle brackets) and the floor panel itself was anchored to the ground at four fixing points. The vertical load was varied to understand its influence on the load-displacement behaviour of the CLT panel. The developed numerical model is validated based on the results presented by Hughes [8] and utilised to examine the

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corresponding load-displacement behaviour of CLT panels manufactured from C16-grade timber. This numerical study will then inform the experimental testing programme of CLT subjected to racking tests in accordance with EN 594 [9].

2 RACKING RESISTANCE OF CLT

CLT wall panels are typically connected to floor panels through a combination of angle brackets and hold-downs. Angle brackets are used to resist shear and hold-downs are used to resist overturning. Dujic et al. [10] tested 2.44 m x 2.44 m glued lamellate panels subjected to monotonous and cyclic horizontal loads in combination with a constant vertical load. The tested wooden panels have relatively high stiffness and load-bearing capacity, therefore the influence of the anchoring system on the shear stiffness and strength of the wall was studied. It was observed that the racking behaviour of cantilever massive wood walls is very sensitive to the magnitude of vertical load and the type of anchorage system. Ceccotti et al. [11] performed a series of full-scale wall tests as part of the SOFIE project. 2.95 m x 2.95 m wall panels anchored using commercially available hold-downs and angle brackets were subjected to monotonic and cyclic tests to assess their racking behaviour. The panels were designed to behave rigidly and the connections were designed to exhibit ductile behaviour. Due to this, all forces and displacements were locally concentrated around the connections leading to local failure, confirming that the dissipated energy results from the connections. Subsequently, full-scale 3-storey and 7-storey buildings were designed as per Eurocode 8 [12] and tested for seismic loads using a 1D and a 3D shaking table respectively. Ductile failure modes with fastener bending and embedment were observed in these buildings. The buildings were observed to resist a whole series of earthquakes in 1D and 3D, keep their shapes and remain fully operational. Gavric et al. [13] investigated the behaviour of several configurations of single and coupled CLT wall panels subjected to cyclic loads according to EN 12512 [14]. Angle brackets and hold-downs were used to anchor the CLT walls to a steel foundation. The experimental results were then compared to advanced analytical models which were developed for nonlinear pushover analysis of the CLT wall system, which showed that angle brackets have sufficient tension capacity in addition to the shear capacity. However, hold-downs did not exhibit significant shear strength. Izzi et al. [15] proposed a numerical model to predict the mechanical behaviour and failure mechanisms of CLT wall systems. The CLT wall and the element to which it is anchored are modelled as 3D solid bodies using ABAQUS, while the connections are modelled as nonlinear hysteretic springs. Racking tests by Gavric et al. [13] were reproduced on the numerical models. The results of these analyses highlighted that in order to obtain a realistic prediction of the load-carrying capacity of the angle brackets, it is necessary to consider the axial load resistance of the angle brackets. Hughes et al. [7] studied the behaviour of tall CLT buildings under monotonic lateral loading. This was achieved by subjecting the CLT wall system to a constant vertical load replicating gravity loads at storeys within a 10-storey CLT building. Non-linear relationships between vertical load and total lateral displacement, and between vertical load and uplift were observed, which implied that vertical load has a significant influence on the behaviour of CLT wall systems.

3 FINITE ELEMENT MODEL

This section discusses the development of a 3-Dimensional finite element model (FEM) of CLT panels subjected to vertical loads and horizontal racking loads using ABAQUS FEM software.

3.1 MODEL GEOMETRY

The model was developed in line with the study presented by Hughes [7,8]. This comprised a 5-layer CLT wall and floor panel with dimensions of 1.5 m in height, 2.95 m in length and 0.1 m thick. The CLT panels were modelled as an orthotropic elastic material with each layer and corresponding grain direction (longitudinal, radial and tangential) being specified within the model. The CLT panels were meshed with 8-noded linear brick elements with reduced integration (C3D8R). The connections were not discretely modelled, however, connector elements with specified elastic and plastic stiffness were specified to mimic the hold-down and angle bracket behaviour.

A steel plate was utilised to apply the structural racking load and hard contact was defined between the surface of the timber and the steel plate with a tangential friction coefficient of 0.3. The vertical pressure was applied to the top surface of the CLT wall, and the base of the floor was fixed to mimic the experimental conditions.

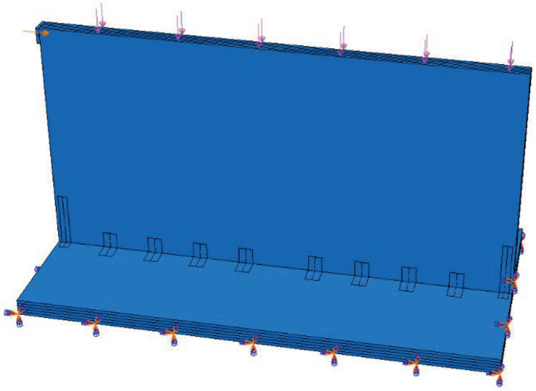

Figure 3: *FEM model geometry*

3.2 MATERIAL PROPERTIES

The material properties of the C24-grade CLT timber presented in Table 1 were sourced from a study by Uí Chúláin [16]. The subscripts *L*, *R* and *T* refer, respectively, to the directions of assumed elastic symmetry, longitudinal, radial and tangential directions. *E* and *G* represent the elastic modulus and shear modulus, respectively. The steel plate was modelled as a linear elastic isotropic material with an elastic modulus of 210 GPa and a Poisson's ratio of 0.3.

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***Table 1:** FEM material properties for C24 grade timber*

Property	Timber (C24)	Unit
E_L	12000	MPa
E_R	370	MPa
E_T	370	MPa
G_{LR}	690	MPa
G_{LT}	690	MPa
G_{RT}	50	MPa
ν_{LR}	0.511	-
ν_{LT}	0.511	-
ν_{RT}	0.203	-

4 FEM RESULTS

This section presents the result of the finite element model which is validated based on the study performed by Hughes [8]. The validated model is then used to examine the corresponding behaviour for C16-grade CLT.

4.1 MODEL VALIDATION

The simulated load-displacement behaviour of the finite element model results is presented in Figure 4. The results compare the lateral/racking displacement (U_{LAT}) and the lateral stiffness (k_{LAT}) of the panel. The lateral displacement is defined as the horizontal displacement of the top of the wall, measured on the top right corner, minus the horizontal displacement of the floor panel measured at the top of the floor panel directly under the wall panel at the right-hand end of the wall panel. This allows the relative movement of the wall panel to be examined. The numerically simulated horizontal movement of the CLT wall is graphically presented in Figure 5a. The simulated uplift of the wall was examined at the bottom left corner of the wall panel which can be graphically seen in Figure 5b. In Figure 4, the lateral displacement versus the applied lateral load is compared to the simulated results for a series of different vertical loads. The different vertical loads of 10.5 kN/m, 31.5 kN/m and 84.0 kN/m correspond to the associated structural loads of a 2-storey (T1 [8]), 4-storey (T6 [8]) and 9-storey (T8 [8]) CLT building, respectively. It can be seen that the behaviour for different vertical loads can be relatively well predicted. The behaviour of the T8 specimen [8] with a vertical load of 10.5 kN/m was found to have the largest lateral deflection when subjected to a lateral load of 50 kN. This was found to be 4.00 mm and the numerical model demonstrated good agreement and predicted a lateral displacement of 4.24 mm for a common vertical load. The connection properties of the hold-down and angle brackets utilised in this study were based on results presented by Hughes [8]. It was stated in this study that the vertical load had a significant effect on the stiffness behaviour of the individual connections and the associated load-displacement behaviour. This was also noted upon increasing the vertical load within the numerical model and utilising common properties for the connections. It was observed that the load-displacement behaviour deviated from that observed experimentally. It can be seen in Figure 4 that the load-displacement behaviour of Specimen T6, which is subjected to a vertical load of 31.5 kN/m, does not match that of the experimental curve initially as a much stiffer response was simulated than that observed for T8 which was only subjected to a vertical load of 10.5 kN/m. The yielding of the connections was also observed in the numerical model. Similarly, a stiffer response was simulated for Specimen T1 which had a vertical loading of 84 kN/m (equivalent to a 9-storey building).

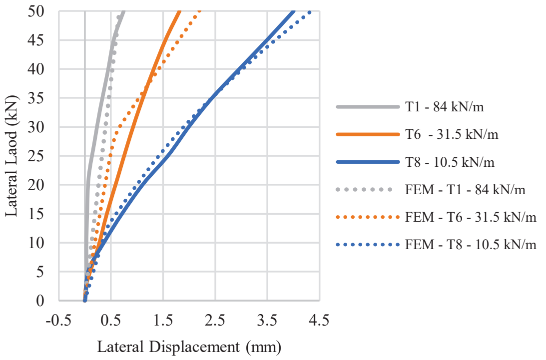
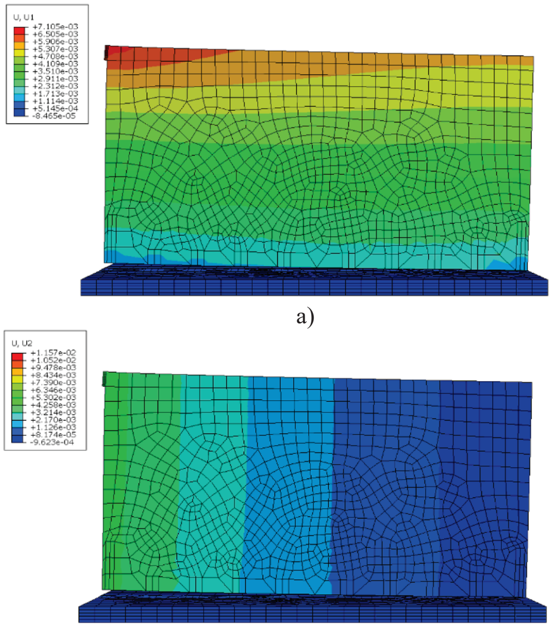


***Figure 4:** Load-displacement behaviour of CLT tests [8] compared to the FEM model results.*

The model was also utilised to compare the experimentally determined lateral stiffness of the panel under different vertical loads. The results are presented in Table 2 and graphically presented in Figure 6. It can be seen that with increasing vertical load, the lateral stiffness of the panel increases as expected and the numerical model predicts the trend observed experimentally.



a)

b)

***Figure 5:** CLT Racking FEM displacement results at a horizontal load of 100 kN: a) horizontal slip/movement (U1) and b) Vertical uplift (U2). Dimensions are presented in meters and a deformation scale factor of 10 was applied.*

In all cases examined in this study, the predicted lateral stiffness is greater than that observed experimentally. The experimental values of T1, T6 and T8 were found to be 13.5 kN/mm, 26.8 kN/mm and 45.0 kN/mm, respectively.

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The results of the model which were taken between 10% and 40% of the maximum load were found to provide values of 15.8 kN/mm, 31.3 kN/mm and 51.7 kN/mm, for T1, T6 and T8, respectively.

Table 2: *Experimental results [8] vs numerical model (FEM)*

FEM Model	Vertical Load (kN/m)	Experimental		FEM	
		U_{LAT} (mm)	k_{LAT} (kN/mm)	U_{LAT} (mm)	k_{LAT} (kN/mm)
FEM-10.5	10.5	4.00	13.5	4.24	15.8
FEM-31.5	31.5	1.82	26.8	2.17	31.3
FEM-84.0	84.0	0.74	45.0	0.62	51.7

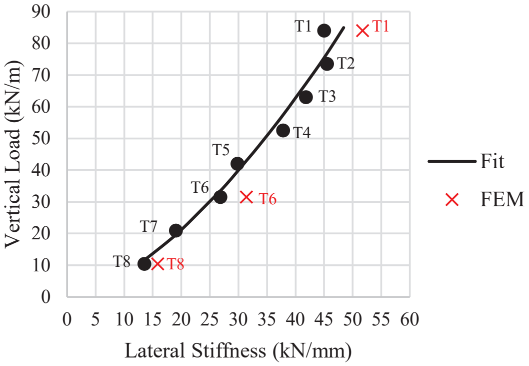

Figure 6: *Vertical load versus the mean lateral stiffness presented by Hughes [8] and the FEM results.*

4.2 RACKING OF C16 GRADE CLT

The developed model was subjected to similar structural loads, however, the structural properties of the CLT were changed from C24-grade material properties to C16-grade properties in accordance with O'Ceallaigh [17] to examine the influence of the grade of timber used to manufacture the CLT. The material properties are presented in Table 3.

Table 3: *FEM material properties for C16 grade timber*

Property	Timber (C16)	Unit
E_L	9222	MPa
E_R	663	MPa
E_T	415	MPa
G_{LR}	659	MPa
G_{LT}	619	MPa
G_{RT}	66	MPa
ν_{LR}	0.529	-
ν_{LT}	0.333	-
ν_{RT}	0.558	-

The results of both models are presented in Figure 7. It can be seen that the stiffness of the CLT has an insignificant effect on the overall lateral displacement and the connections govern a significant proportion of the behaviour. The connection properties are not varied in this study but will form a significant component of future testing on typical bracket connections under tension and shear loading. The developed model will inform the experimental test programme for C16-grade CLT connections.

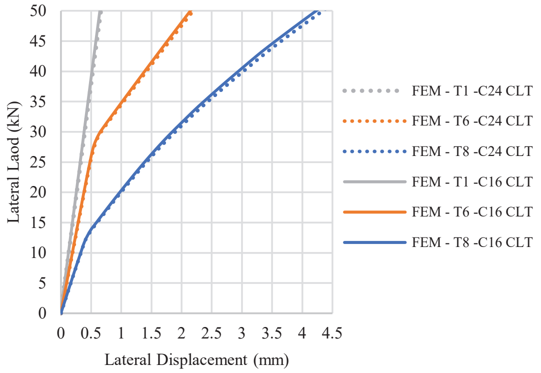

Figure 7: *Comparison of the load-displacement behaviour of CLT manufactured from C24-grade timber and C16-grade timber under different vertical loading situations.*

5 FUTURE EXPERIMENTAL TESTING

This section discusses the planned manufacture and experimental testing of C16-grade CLT under racking loads in accordance with EN 594 [9]. The further development of a suitable numerical model for the prediction of the behaviour of C16-grade CLT connections requires a series of experimental tests to be carried out. While a significant amount of research has been carried out for CLT connections, most of the testing has been carried out for C24-grade CLT. Only a limited number of studies focused on C16-grade timber. O'Ceallaigh & Harte [5] used C16 grade CLT panels to study the behaviour of commonly available steel-to-timber angle bracket connections. The connections were subjected to monotonic and cyclic loadings and were tested for different fastener lengths. The experimental results were compared to analytical models developed using the provisions of Eurocode 5 [18] and the analytical model developed by Blaß & Uibel [19,20]. The results were shown to perform well for C16-grade material. Therefore, it is proposed to test C16-grade CLT connections comprising individual brackets which will allow for an improved numerical model to be developed for C16-grade CLT. It is proposed to extend upon this testing to examine a series of brackets/connections in racking wall systems.

5.1 C16 CLT MANUFACTURE

For the proposed testing, the CLT will be manufactured in accordance with EN 16351 [1]. Two different sizes and test configurations of CLT panels will be manufactured, namely a 3-layer (40-40-40) and a 5-layer (40-20-20-20-40) panel. Boards of C16 Irish Sitka spruce (*Picea sitchensis*) with nominal cross-sectional dimensions of 100 mm × 35 mm (for panels comprising 20 mm thick layers), and 175 mm × 47 mm (for panels comprising 40 mm thick layers), will be used to manufacture the CLT panels. The boards will be initially stored in a conditioning chamber at a relative humidity of 65 ± 5% and at a temperature of 20 ± 2°C, before specimen preparation. Prior to fabrication, all of the sides of the boards will be planed to the desired board thickness (20 mm and 40 mm) and width (80 mm and 160 mm). All of the panels will be bonded using a one-component PUR adhesive (PURBOND HB S309), with a spreading rate of 160 g/m^2, and a pressure (face bonding only, no edge bonding) of 0.6 N/mm^2.

5.2 CONNECTION TESTING

It is proposed that typical CLT bracket connections are to be subjected to compression and shear testing. The influence of three different fastener lengths is to be studied to establish the elastic and ductile behaviour of these connectors for C16 grade CLT. Five sets of tests for each fastener length are proposed for both compression and shear loads. In addition to angle brackets, timber plates or stud rails connected using self-tapping screws is an alternate method for floor-to-wall connections and will be investigated.

5.3 RACKING TEST SET-UP

The connection testing will inform the racking tests. Racking panels, measuring 2.4 m x 2.4 m, will be manufactured in accordance with EN 16351 [1] and tested in accordance with EN 594 [9], which governs the test procedure for determining the racking strength and stiffness of timber frame wall panels. The test protocol allows for comparative performance values for the materials used in the manufacture of the panels to be determined and provides evidence in the form of experimental data for use in structural design. The racking frame in the University of Galway can be seen in Figure 8.

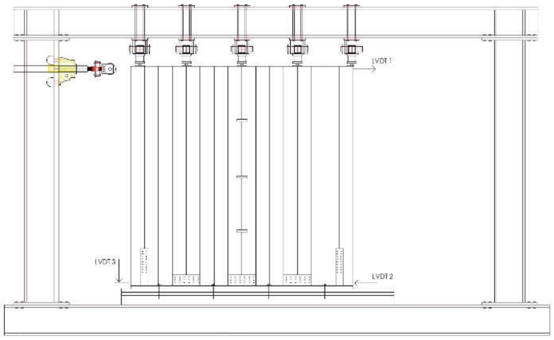

Figure 8: *Racking frame test set-up*

The self-restraining racking frame has the capability to apply a horizontal racking load of up to 200 kN and a series of vertical loads ranging from 1-5 kN. The lateral or racking load (*F*) is applied via a contact roller to the top left of the panel. The panels will be tested with no vertical load and with the vertical loads (F_v) set to apply a maximum vertical load of 5 kN at five different points along with the top of the wall panel. The five vertical loads are equally spaced along the top of the panel allowing for 100 mm from the leading edge of the wall in accordance with EN 594 [9]. For the purpose of this testing, the leading edge of the panel is the left edge (Figure 8) and the lateral/racking load is applied to the top of the leading edge of the wall panel. The right edge of the panel (see Figure 8) is referred to as the trailing edge.

The displacements of the panel will be monitored using linear variable differential transducers (LVDTs) at locations indicated LVDT 1, LVDT 2 and LVDT 3 (see Figure 8). The deformation (v) for the calculation of the lateral/racking displacement is taken as displacement at LVDT 1 minus the displacement at LVDT 2. The displacement at LVDT 3, which represents the vertical displacement or uplift of the panel is reported separately.
The horizontal racking load will be applied at an appropriate rate to ensure the rate of loading achieves 90 % of the maximum racking load (F_{max}) within 300 ± 120 s in accordance with EN 594 [9].
The lateral/racking displacement of the panel is calculated as shown in Eq. (1).

$$v = v_{LVDT1} - v_{LVDT2} \tag{1}$$

Where v_{LVDT1} is the lateral displacement at LVDT 1 and v_{LVDT2} is the lateral displacement at LVDT 2 (see Figure 8). The racking strength of the panel is the maximum load attained during the test as shown in Eq. (2).

$$F = F_{max} \tag{2}$$

where F_{max} = the maximum load attained.
The lateral/racking stiffness (R) is calculated as shown in Eq. (3).

$$R = \frac{F_{40} - F_{10}}{v_{40} - v_{10}} \tag{3}$$

where F_{10} and F_{40} = are the loads corresponding to 10% and 40% of F_{max} and v_{10} and v_{40} = are the displacements corresponding to 10% and 40% of F_{max}. Essentially, the Racking Stiffness (R) is the slope of the line between 10% and 40% of F_{max}.

6 CONCLUSIONS

A finite element model capable of replicating the load-displacement behaviour of wall-floor CLT connections has been presented. The model has been shown to perform well when examining the load-displacement behaviour and lateral stiffness of C24-grade CLT. The model was also utilised to examine the influence of the timber grade used in the manufacture of the CLT. The numerical results have demonstrated the potential for significant lateral/racking resistance to be achieved from CLT panels manufactured from C16 grade material but it was shown that the stiffness of the CLT itself may not be as important as the defined connection behaviour. This finding must be examined experimentally and is a key focus of future work which will further calibrate the connections utilised in the developed model to better predict the behaviour not only under monotonic loading but also under cyclic loading situations. The development of a refined model will allow a series of different configurations of connections to be examined utilising different grades of CLT. The model and its findings will provide designers with confidence in this product and may encourage producers to incorporate C16-grade material into the production of CLT.

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