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A Comparison of Punching Shear Design Approaches for Point Supported CLT Panels

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ABSTRACT: The following study examines different methods of punching shear design for point supported Cross Laminated timber (CLT) panels. A literature review was completed, summarizing CLT rolling shear capacity, as determined in both European and North America research as well as code and material standard documents. Analysis methods for calculating internal stresses in CLT slabs were then reviewed, with an emphasis on simplified design equations. Several design checks were then completed by comparing predicted capacity to tested strength for CLT panels from existing experimental programs. Predictions from each method were compared to actual panel capacities from testing, and it was found the methods used predicted failure at approximately 3x the input load or higher for the specific panel examined. A parametric study was completed where a number of panels were analysed using the different analysis methods outlined. The parameters studied include panel number of spans, length, width, grade, thickness, and support size. It was shown that simplified analysis methods make predictions that were comparable to FEM predictions, but FEM results varied more with respect to these parameters. It was also shown that the support stiffness was a significant parameter on the distribution of shear forces and the peak model forces observed.

KEYWORDS: Tall wood buildings, CLT Floors, Point-supported floors, two-way floor action, Punching Shear, Rolling Shear.

1 INTRODUCTION

Recent changes to the National Building Code of Canada [1] and International Building [2] have increased height limitations on tall timber buildings up to 12 and 18 storeys respectively. These tall timber provisions accommodate a quickly growing segment of the market, one with many benefits such as decreased embodied carbon in buildings and shorter construction times. One structural system gaining popularity is point supported Cross Laminated Timber (CLT) a gravity system where panels are supported directly on columns without intermediate beams. Buildings using the point supported CLT structural system minimize floor assembly depth. They also reduce material volumes and construction timelines due to the limited components.

The force distribution of point supported CLT differs from typical CLT on beams, and shear forces near the supports are much higher. As such, this type of system is more frequently governed by shear design instead of bending or deflection. While CLT has been studied significantly over the past 20 years, there is minimal code guidance for the design of CLT in this condition. One prominent building in North America using point supported CLT panels is TallWood House at the University of British Columbia [3], where physical testing of full scale CLT panels [4] was used to validate the design of the point supported structural system.

Since the UBC TallWood House was completed, a number of research programs have studied point supported CLT, evaluating both the rolling shear strength of CLT and analysis methods to assess internal stresses. However, few studies to date have compared the predictions made by different design methods. To help address this knowledge gap, the following paper reviews various methods to determine the capacity and demands for CLT punching shear, and compares their predicted results. The analysis and capacity methods examined were applied to a design example where a number of design cases are considered on an experimentally tested panel. Finally, a parametric study was used to examine the effect a wide range of variables on panel response.

2 CLT PUNCHING SHEAR CAPACITY

There are two significant parameters for point supported CLT: the rolling shear strength (f_s), which describes the strength of the base CLT fibre; and the punching shear amplification factor (k_s), which increases the base strength of CLT near columns to it's punching shear strength (f_p). A summary of the rolling and punching shear amplification of CLT is outlined in the following section.

2.1 ROLLING SHEAR STRENGTH

In North American material standards and CLT fabrication standards "specified strength in rolling shear, fs, is taken as approximately 1/3 of the specified strength in shear, f_v, for the corresponding species combination" [5,6]. The rolling shear strengths specified in the fabrication standards are consistent with this rule instead

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of being developed based on test results in material studies.

The rolling shear strength of CLT can be determined experimentally using either inclined direct shear test or four point bending test. Inclined shear tests load a small specimen of CLT in shear directly through a single layer oriented for rolling shear. Many tests on different European species of wood were completed by Erhert et al. [7,8]. It was noted that the strength of CLT was influenced by the aspect ratio of the boards used. Other studies using inclined shear tests include Wang et al. [9], Mestek [10], and Muster [11]. Four-point bend tests load a CLT specimen in bending to failure. The internal stress is indirectly determined using an analytic or numerical assessment of the CLT beam based on the known shear failure load and the CLT layup. Several studies used this method on European [12], Asian [13], Oceanic [14] CLT. While the experimentally measured rolling shear strength varies, impacted by parameters such as species, width-to-depth ratio of the lamella, etc., strengths reported for softwood have been shown within the range of 0.8MPa to 2.0MPa. Table 3 summarizes the 5th percentile strengths observed in different studies and compares them to the characteristic rolling shear strength specified by the material standard CSA O86. To be compared with the short-term test results with the CSA O86 [5] values have a short term load duration factor of 1.15 applied to the standards specified value.

Table 1: *Comparison of 5th percentile strength with CSA 086 code values.*

Reference	f_s (MPa)
CSA O86 SPF [6]	0.50
Wang et al. [9]	0.76
Ehrhart (2015) [7]	1.51
Mestek (2011) [10]	1.47

2.2 PUNCHING SHEAR STRENGTH

Point supported CLT is loaded with a more complex load path than those used in rolling shear tests. The punching shear strength of CLT can be predicted using direct punching tests where a point load is applied to a CLT panel supported or 3 or 4 sides. Based on the ultimate strength observed in testing, the internal shear stresses are determined using analytical methods or numerical methods such as FEM models. Recent studies [10,11,15] have shown that the stress predicted at failure corresponds to a higher shear capacity than the observed base rolling shear strength. Mestek [10] noted that point supported CLT specimens failed at internal stress 1.6 times the predicted capacity associated with base rolling shear strength. Similar increases in strength were noted by Bogensberger et al. [15] and Muster [11].

Possible factors contributing to the observed strength increase include additional confinement from compression, the restraining effect of top and bottom layers, and nonlinear redistribution of forces at failure. Bogensberger proposed a nonlinear strength model of CLT model that could account for the observed higher shear stress [15]. The effect of compression on rolling shear strength was observed by Mestek [10], where the shear strength observed where compression was applied to inclined shear specimens was consistently higher.

Based on the observed data, a draft Eurocode document [16] proposed a factor, k_p=1.6, to increase the base rolling shear strength for punching shear in CLT. In a later study, Muster [11] proposed decreasing k_p to 1.3 at supports at panel edges.

3 POINT SUPPORTED CLT ANALYSIS

Internal stresses can be estimated using ether finite element models, or simplified design equations. The following sections highlight existing methods used to predict demands in CLT panels.

3.1 FINITE ELEMENT MODELS

Three types of Finite element models (FEM) have primarily been used for analysis and design of CLT in punching shear:

- Solid 3D element models.
- Shell 2D element models.
- Beam models using the shear analogy.

Solid 3D models are the most detailed analysis approach of the above three. Each layer of wood is modelled with interaction properties between layers defined. These models can show complex behaviour such as the effect of edge glue, the contact stresses between layers or supports, and the interaction between bending and shear forces. Internal stresses are generally read directly from FEM models. Solid models have been used in a number of studies [10,11,15] to predict internal panel stress, or compare to other design methods in research.

Shell element 2D models represented the CLT with either an orthotropic shell element with equivalent section properties, or a layered shell element. In layered shell elements, stresses can be extracted from layers directly, and some software products, such as Dulbal's RFEM, have layered shell representations of CLT [17]. Stresses through the depth of the panel are then calculated based on established analytical models such as beam shear models. If orthotropic shell elements are used, then stresses are calculated using a section analysis.

Muster [11] presents a modelling approach where 2D shell provided similar stresses to 3D finite element models. In this approach, the supports are modelled as springs with a spring stiffness tuned so that bending stresses were similar between the two models. The simplicity of 2D shell element modelling makes this method attractive to practicing designers.

A grid of beam elements in conjunction with the shear analogy method can also be used to model point supported CLT. This method was used by Mestek [10] to determine the distribution of shear forces on column supports, in lieu of the solid 3D model or a calibrated 2D shell element model. Girder-girder models avoid the extreme stress concentrations that can occur in solid or shell element models.

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3.2 SIMPLIFIED MODELS

In addition to FEM, a number of simplified analysis methods have been proposed. Simplified analysis methods typically first determine shear forces on each face of a column, then find internal stress using those forces. The accuracy of these methods is generally determined by comparing to FEM analysis. Three simplified analysis methods were considered in this study:

- Tributary Area simplified method [3].
- Mestek's simplified method [10].
- Muster's simplified method [8].

The Tributary Area simplified analysis method was proposed in a study for the UBC TallWood House [3]. This method determines column face forces (V_i) using associated tributary area. Internal stresses, (τ_i) are then evaluated based by evenly distributing force across the width of the effective support, then using beam bending equation as described in equation 1. The effective depth (d_i) is taken as the height of CLT between layers longitudinal to CLT face being evaluated. The effective width of the support ($b_{i,eff}$) is as half the effective depth (d_i) from the respective face of the column.

$$\tau_i = \frac{1.5V_i}{b_{i,eff}d_i} \qquad (1)$$

Mestek's simplified analysis method was developed and calibrated from a parametric study using FEM grid-beam CLT models used to determine forces at the face of column supports. From the support forces, the internal stress across the effective support width and through the depth of the panel are based on equations from DIN 1052 [18]. Conditions for supports at corners of panels, and another for supports in the centre of a panel were considered.

To determine the distribution of forces, a FEM based parametric study was implemented considering:

- Three ratios of strong-to-weak axis lamella thickness, 1, 1.5, and 2
- depths of CLT from 100mm to 220mm with 5 to 11 symmetrical layers depending on thickness and layup
- A single span with a span length-to-width ratio between 1-3

Support size and bay size were not considered in the parametric study. The study also assumes no narrow side edge gluing. Based on outputs from these parametric studies, equations for calculating the force on the strong axis face (V_x) and the force on the weak axis face (V_y) were determined using regression equations. The study found that the distribution of force on each support face had a strong correlation with both number of CLT layers, and the ratio between CLT layer thicknesses. Two equations are developed for both columns at edges and columns away from any panel edge.

Given the support shear force(s), an equation simplified from the German design code [18] is used to calculate peak stress (τ_i) as described in equation 1. This equation is based on the lamella thicknesses in each direction (t_x, t_y), effective support width ($b_{i,eff}$) and two modifications factors: a stress concentration factor (k_A) and a pre-calculated modification factor (k_R). The effective support widths ($b_{i,eff}$) was taken as 35° from the face of column support.

$$\tau_i = \frac{K_{A,i}1.5V_i}{k_{R,i}b_{i,eff}(t_x + t_y)} \qquad (2)$$

The stress concentration factor (k_A) accounts for the ratio between the peak stress and average stress across the effective support width in the FEM model for panels supported at their edge. The factor ranges from 1.35 to 1.6 depending on effective support width and panel thickness, but is taken as 1.0 for panels supported away from their edge. The factor k_R is a coefficient precalculated from the section analysis method presented in DIN. Table 2 summarizes values of k_R for a corner support.

Table 2: *Values for K_R in the x and y direction for a corner support.*

Factor	Number of Lamella			
Orientation	5	7	9	11
$k_{R,x}$	2	2.5	3.3	3.89
$k_{R,y}$	1	2	2.5	3.3

Mestek also compared the proposed simplified equation to the FEM results of the parametric study. The predictions from the simplified equations were typically higher than the FEM results, with most in the range of 20% higher. One exception was the weak axis CLT shear stress for 5ply CLT, which was 40-70% higher than the FEM results.

Muster also developed a simplified analysis method, where column face forces (V_i) were calculated using a strip method modified from Hillerborg [19]. Once column face forces were determined, beam bending equations on an effective section were used to determine internal stresses, as shown in equation 3. Two k factors were included in this equation. The stress concentration (k_A) was taken from Mestek and accounts for the ratio between the peak and stress. The stress concentration factor (k_{edge}) was derived from comparisons with FEM data, and represent additional stress concentrations resulting from openings through the panel at supports at panel edges. The effective width is calculated by taking a combination of 45° and 15° angles from the column support for through parallel and perpendicular to grain shear loading respectively.

$$\tau_i = \frac{k_A K_{edge}1.5V_i}{b_{i,eff}d} \qquad (3)$$

Muster compared the simplified equation to 3D FEM models, and it was found that both models had similar trends, but a large scatter between the Muster's simplified analysis method and 3D solid FEM stresses.

4 POINT SUPPORTED DESIGN METHODS

The following section compares several design cases for point supported CLT in a design case study using test data from the UBC TallWood House building. Each design case was completed by choosing an approach for calculating panel capacity as presented in Section 2, to internal stresses predicted by an analysis method from Section 3.

4.1 OVERVIEW OF CONSIDERED DESIGN METHODS

The analysis methods chosen for use in each design case were a mix of FEM and simplified methods. For the simplified analysis methods, both the Tributary Area and Muster simplified analysis method from Section 3.2 were chosen because of their simplicity and flexibility. While the tributary method does not account for centre span hogging, it has no limitations on panel configuration and column locations. Muster's method accounts for hogging of centre columns and panel stiffness, and can analyse any number of panel spans. Mestek's simplified analysis method was excluded because it is limited to single span panels with columns at corner, or columns located at the middle of panel.

For the FEM analysis methods, only the 2D shell analysis were examined due to the complexity associated with making 3D solid modelling and grid-beam models. Several hybrid analysis methods are also examined, where the FEM models are used to calculate V_i on each column face, then simplified analysis equations are used to calculate internal stresses.

Three types of capacity methods were used in the design cases considered: the base code strength, the code strength amplified by k_p, and 5th percentile characteristic strength from test data presented in Table 1. The code strength represents the most conservative assumption, while using test data with k_p is the least. The design cases considered are summarized in Table 3, which describes the methods used to both analyse and determine the capacity of each panel.

Table 3: *Design methods considered.*

Design Case	Analysis Method	Capacity Method
D1	Tributary Area	PRG
D2	Muster's simplified method	PRG & k_p
D3	Hybrid approach, FEM with Muster simplified	PRG & k_p
D4	FEM RF-Laminate	PRG & k_p
D5	Hybrid approach, FEM with Muster simplified (support stiffness x10)	PRG & k_p
D6	FEM RF-Laminate (support stiffness x10)	PRG & k_p
D7	Hybrid approach, FEM with Muster simplified	Test data & k_p

Design case D1 uses the Tributary Area simplified method compared against O86/PRG rolling shear values without punching amplification factors. Case D2 uses the Muster simplified analysis method and compares peak stresses against O86/PRG rolling shear values amplified by the Muster punching shear amplification factors. Case D3 is a proposed hybridized method, which used a FEM 2D shell model of the system with spring supports as presented by Muster to predict column face forces (Vi) [11]. The equations from Muster's simplified method were used to predict peak stress in the system, and compared with the amplified O86/PRG capacities. Case D4 is a full FEM approach using a similar model to the one used in D3. Peak stresses in individual lamella from the RF-Laminate module were extracted from the model and compared with amplified O86/PRG capacities. Cases D5 and D6 used similar models and capacities to D3 and D4 respectively, but with support spring stiffness increased by a factor of 10 to evaluate the sensitivity of the model to stiffness. Finally, design case D7 used a similar model to that in D3, with unchanged spring stiffness, but compares the peak stresses against a 5th percentile base rolling shear strength of 0.75Mpa, along with Muster amplification factors. The base strength value for this case more closely aligning with results from testing [9],

4.2 COMPARISON WITH PREVIOUS TESTING FOR UBC TALLWOOD

The design cases presented in Table 3 were evaluated by comparing their design strength to internal stress for point supported panels in the UBC TallWood House testing [3]. For this evaluation, the panel from Manufacturer III was used, which was a two-span continuous 5ply panel, 169mm thick. Failure occurred in punching shear at the middle support in the strong axis. An average ultimate load of 402kN (201kN per bay) was applied to the system. Figure 1 shows the geometry of the panel tested.

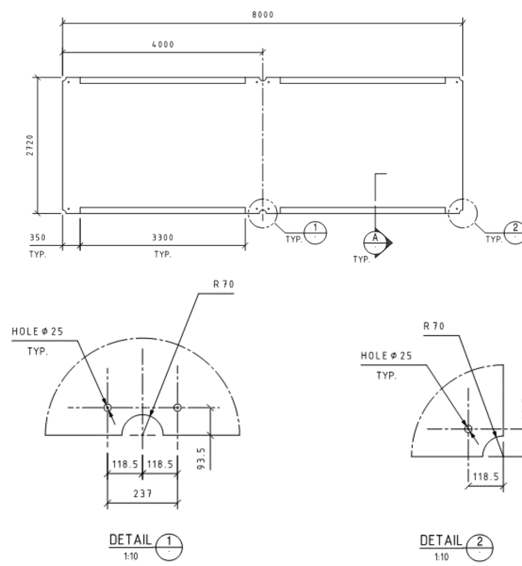

Figure 1: *Geometry of panel considered for Testing.*

For each design case from Table 3, the CLT panel from experimental was analysed using it's failure load to determine the stress demands at failure. For the case D1 and D2, the load was simplified as an equivalent uniform load. For the remaining series the loads were applied as discrete area loads to match the true test setup as shown

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in [3]. Table 4 summarizes predicted column face shear for the middle the column, where V_x is the maximum force on the left and right face (i.e. face perpendicular to primary span), and V_y is the shear on the top or bottom face (i.e. face parallel to primary span).

The column face forces predicted by each design case was similar with a few exceptions. In D1 the force in the weak axis was smaller than all other design cases. There was also a notable difference between FEM models with 10x support stiffness used, where stiffer models (D5/D6) attracted more force in the weak axis.

A significant spread is observed in the stress predicted by each design case. Case D1 predicted lower shear stresses than other methods, likely because of not including any stress concentration factors. Comparing design cases that used hybrid analysis (D3, D5) with design cases that used pure FEM (D4, D6), the hybrid methods predicted higher stresses for the baseline model, but lower stresses when the support stiffness was increased. When similar design cases that used different support stiffnesses (D4 vs. D6, D5 vs. D7) were compared, it was found that the spring stiffness significantly impacts the load distribution between column faces, with stiffer supports leading to more force in the y axis and higher stress peaks.

Table 4: *Design Demands*

Design Case	V_x (kN)	V_y (kN)	τ_x (Mpa)	τ_y (Mpa)
D1	32	33	1.55	1.3
D2	33	55	2.81	1.5
D3	34	60	2.94	1.65
D4	34	60	2.28	1.8
D5	26	71	2.25	1.98
D6	26	71	3.05	2.24
D7	34	60	2.94	1.65

Table 5 summarizes the shear strength(s) considered in each design method (D1-D7) for the strong and weak axis faces. The characteristic strength used was multiplied by a phi factor of 0.9 from CSA O86 to determine the design rolling shear strength (f_s'). The punching shear strengths in each direction (f_{px}, f_{py}) were determined based on amplifying the design rolling shear strength by punching factors where applicable. An Overstrength Factor (OF), defined as the ratio between the punching shear stress at failure and the capacity, is also shown. Lastly the predicted failure axis, where the predicted shear stress and the punching capacity the closest, is presented.

Table 5: *Capacity and Overstrength Factor (OF) for design cases considered.*

Design Case	f_s' (Mpa)	f_{px} (Mpa)	f_{py} (Mpa)	OF	Governing Axis
D1	0.45	0.45	0.45	3.44	strong
D2	0.45	0.585	0.72	4.79	strong
D3	0.45	0.585	0.72	4.96	strong
D4	0.45	0.585	0.72	3.90	strong
D5	0.45	0.585	0.72	3.85	strong
D6	0.45	0.585	0.72	5.21	strong
D7	0.68	0.884	1.088	3.33	strong

In all design cases, the strong axis governed the shear capacity, which matched observations from testing [3]. In addition, all design cases using PRG based rolling shear strength had an OF above 3.4, with the series D1 showing the lowest predicted value. For series D7, where a larger rolling shear strength was used, there was still a significant margin of safety in the system, with an overstrength of 3.33.

5 PARAMETRIC STUDIES

To further understand how the analysis methods presented in Section 3 compare, a parametric study was completed. The output column face shear forces from simplified analysis methods were compared with outputs from FEM. The impact of different parameters on outputs of the shell FEM approach presented in Muster were also examined.

5.1 STUDY OVERVIEW

For simplified methods, the parametric study uses 175mm thick CLT panels 5 layers of 35mm thick laminates, 1950f-1.7E SPF in the strong axis and No.1/No.2 SPF in the weak axis. The parameters considered include:

- number of spans (1,2,3)
- span length (3.6m – 4.4m, increments of .2m)
- span width (2.7m – 3.5m, increments of .2m)

For FEM models, the panel was modelled using the 2D shell elements and support conditions as specified by Muster. Near supports a mesh refinement was used. The following parameters were varied in addition to those above in each trial analysis:

- Effective width of support along the weak axis face (0.3m-0.5m, increments of 0.05m, strong axis face is half this value.)
- Spring stiffness (0.1x, 1x, 10x the stiffness equation proposed by Muster)
- CLT panel type (139, 175, 191, 245)
- Grade (E and V)

The 175mm and 245mm thick panels had 5 and 7 layers of 35mm. These panels used 2100f -1.7E SPF in the strong axis and No.1/No.2 SPF in the weak axis for E rated panels, while V rated panels used No.1/No.2 SPF in both directions. The 139mm and 191mm had layers 35mm thick in the strong axis and 18mm thick in the weak axis. These panels used 2100f-1.7E SPF in the strong axis and No.3 and better SPF in the weak axis for E rated panels, while V rated panels used No.1/No.2 SPF in the strong axis, and No.3 and better SPF in the weak axis.

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All panels have an edge cantilever of 0.3m on either side of the first and last column. In FEM, the effective dimension of the strong axis face is taken to be half the width of the weak axis face. All panels were loaded with a unit 1kPa load, and results were presented in kN/kPa. A total of 150 trials were run for each simplified method panel, and 2250 trials for each FEM model.

5.2 SIMPLIFIED METHODS COMPARISON

The column face shear forces were compared for the Tributary Area analysis method, Muster simplified analysis method, and FEM models. Because there were multiple columns, each plot shows the shear force for the worst-case column. Figure 2 shows the strong axis column face forces (V_x), and Figure 3 shows the weak axis column face forces (V_y). Width had a small effect on V_x and similarly, changes in length had a small effect on V_y. For this method number of spans had a moderate effect on V_x, but a large effect on V_y.

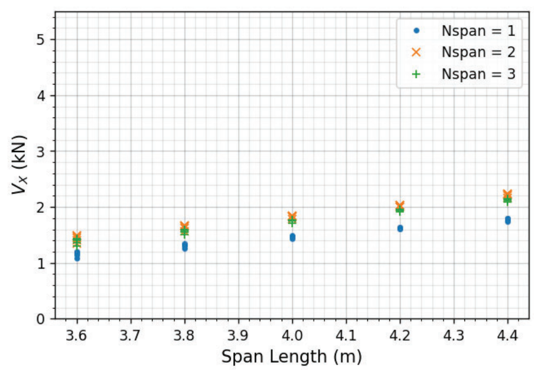

Figure 2: *Shear on X column face for Muster Simplified method.*

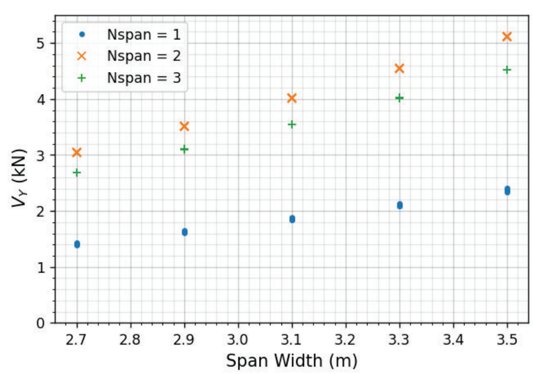

Figure 3: *Shear on Y column face for Muster Simplified method.*

Figure 4 and Figure 5 show the column face forces for the Tributary Area analysis method. Like the Muster analysis method, changes in width had a small effect on V_x and changes to length had no effect on V_y. Similarly, the number of spans had no effect on V_x, and only effects V_y, going from one to two spans. As such, results are only presented for two span panels.

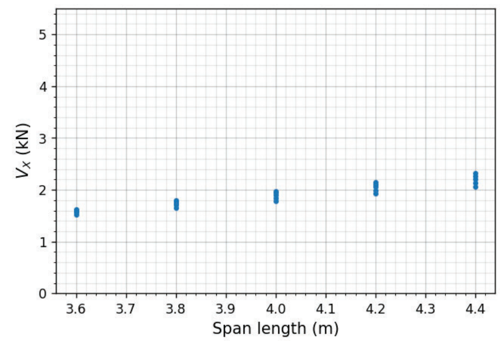

Figure 4*: Shear on X column faces for Trib. Area Method.*

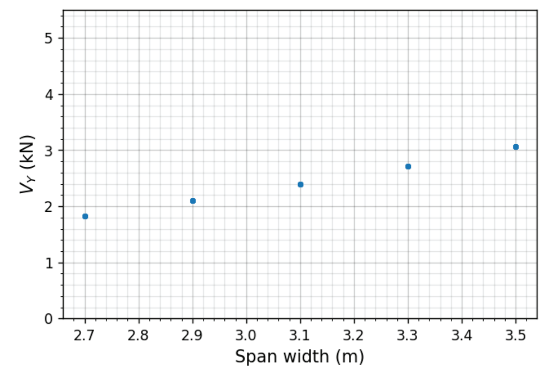

Figure 5: *Shear on Y column faces for Trib. Area Method.*

Results for the Tributary Area and Muster simplified analysis methods were also compared to results from FEM models. The shear force at each column face was extracted from the model for each combination of input variables, and the largest value from all columns was reported. For the FEM models, there was a much larger spread in results, indicating that factors like the out of plane grid dimension (width in this case) or support size had a larger impact on the distribution of forces. Table 6 summarizes the spread in V_x observed, where length is fixed at 3.6m and 4.4m, the number of spans is fixed at two, three or four, and all other parameters were allowed to vary. For the spread of values at each length/span combination, the smallest (min) and largest (max) values from all other combination of parameter were reported, as well as the average value observed. For the Tributary Area simplified analysis method, values were the same for every number of spans, and so not separated out.

Table 6: *Comparison of V_x from simplified methods and FEM*

L (m)	Analysis Method	N_{span}	V_x (kN)		
			Min.	Avg.	Max.
3.6	Muster	2	1.09	1.16	1.20
		3	1.35	1.44	1.49
		4	1.35	1.38	1.43
	Trib. Area	-	1.52	1.58	1.62
	FEM	2	1.10	1.29	1.45
		3	1.39	1.65	1.97
		4	1.39	1.58	1.87

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4.4	Muster	2	1.74	1.78	1.80
		3	2.17	2.21	2.23
		4	2.17	2.13	2.15
	Trib. Area	-	2.06	2.20	2.32
	FEM	2	1.37	1.64	1.86
		3	1.75	2.07	2.46
		4	1.75	1.98	2.31

5.3 EFFECT OF PARAMETERS ON SHEAR FORCE

Data from the parametric study was also used to understand how the studied variables change the shear force predicted by FE models. As with the data from section 5.2, it was found that there was a large spread in predicted shear force when comparing applied shear with the span length in that direction (V_x vs. span length, V_y vs. span width). Correlations on the dataset were calculated to highlight which variables are most significant when predicting shear force. Table 7 summarizes the correlations coefficient determined for span length (L), span width (w), number of spans (N_{span}), effective support dimensions (d_{sup}), lamination grades (V or E Grade), and support stiffness (k_s) on the output shear (V_x, V_y).

In Table 7 the shear force generally correlated strongly with the span direction perpendicular to the support face, but there was also a strong correlation with the opposite span length as well. Grade and support size had less of an effect on the distribution of shear force. Strong correlations were also noted for support stiffness and number of spans.

Table 7: *Correlations of various parameters on Shear forces.*

Panel	Correlation Coefficient with V_{xh}					
	L	w	N_{span}	d_{sup}	Grade	k_s
139	0.23	0.08	0.11	-0.11	-0.05	-0.51
175	0.25	0.21	0.22	-0.06	-0.06	-0.35
191	0.27	0.24	0.28	-0.07	-0.06	-0.31
245	0.25	0.27	0.32	0.02	-0.07	0.00
Panel	**Correlation Coefficient with V_y**					
	L	w	N_{span}	d_{sup}	Grade	k_s
139	0.15	0.35	0.49	0.02	0.01	0.41
175	0.19	0.32	0.53	0.00	0.02	0.34
191	0.18	0.31	0.50	0.03	0.02	0.35
245	0.23	0.30	0.57	-0.01	0.02	0.18

The strong correlation of V_x with support stiffness, but not support width was highlighted below in Figure 6, where a 175 V rated panel with two spans and a 4.2mx2.9m grid has been examined. In this small subset of the data, a large difference between the shear force for different support stiffnesses values (x0.1, x1, x10) was shown. However, shear force was relatively constant for different values of effective support width.

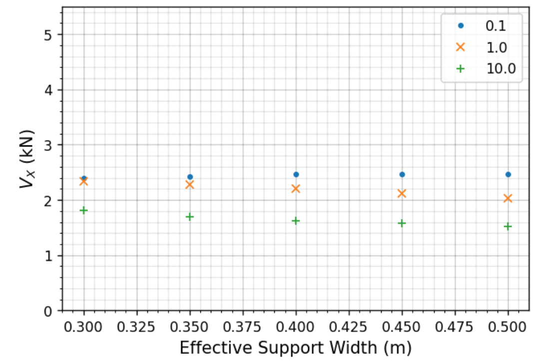


Figure 6: *V_x vs. support width for a trace of data on a 175 V rated panel, and different factors applied ot the support stiffness.*

5.4 FEM MODEL MEASURED K_A

Another quantity measured in the parametric study was the ratio between the peak and average shear force observed in FEM models across a wide variety of panels. In Figure 7-Figure 10, these values were compared to the baseline K_A value proposed by Mestek. The parameters of the parametric study were as presented in Section 5.2.

For each of the panels considered, two series were presented: the baseline formula for spring stiffness from Muster, and x10 the spring stiffness proposed from Muster. For all FEM models the peak value in V_x/m was extracted from the model, and compared to the average value of V_x/m. A large spread in peak/average can be observed that for some series, with thinner panels having a larger distribution in stresses. This may be because Muster's formula for the spring stiffness changes at a negative third power of the panel thickness, so the base stiffness for each panel can be significantly different. The difference in stress ratio suggests significantly different behaviour at supports between the 139mm panels and 245mm panels. Despite the large differences observed between different panels, K_A was lower than the values proposed by Mestek for many panels.

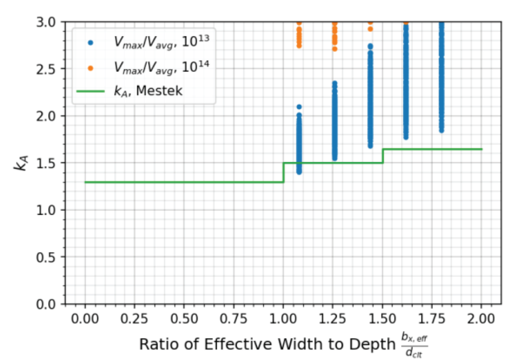


Figure 7: *Calculated K_A for 139 panels.*

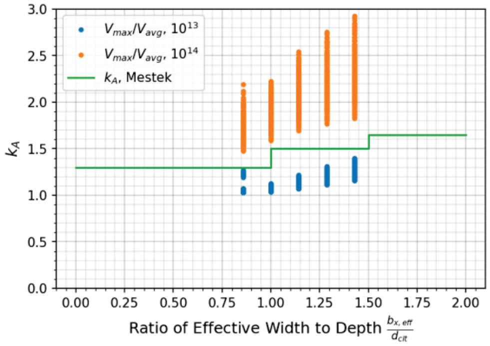


Figure 8: *Calculated K_A for 175 panels.*

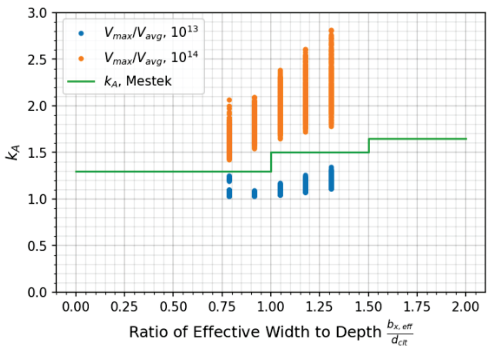


Figure 9: *Calculated K_A for 191 panels.*

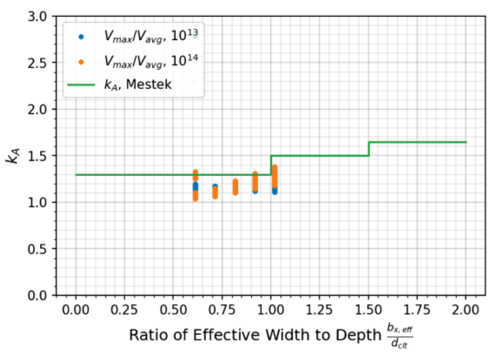
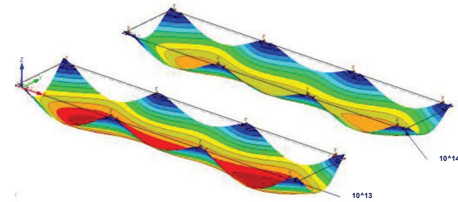
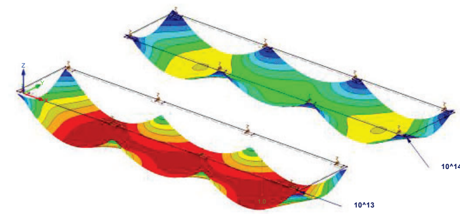


Figure 10: *Calculated K_A for 245 panels.*

The effect of support stiffness as predicted by Muster [11] can also be qualitatively observed in Figure 11 for a single data point. Here a set of four panels is shown where the top pair was 139E rated, the bottom pair was 245E rated, and all other variables are the same between panels. For each pair, the bottom panel uses the baseline formula's constant of 10^{13} (1x stiffness), and the top panel uses 10^{14} (10x stiffness). For these panels, there is a significantly different deformation profile over column supports for the baseline model of 245E vs. 139E, with the 245 behaving much more flexibly.



139E



245E

Figure 11: *Deformation over supports for 139E and 245E panels, colour indicates vertical deflection with red being the highest and blue the lowest.*

6 CONCLUSIONS

Reviewing the code and North American material standards for determining capacity of point supported CLT, it was observed that PRG specified shear strength values were generally lower than characteristic strengths determined in experimental test. Several simplified analysis methods were reviewed, with the method presented by Muster [11] being the most flexible.

The capacity and analysis methods were applied to CLT panel tests in a number of design cases. In addition to the simplified analysis methods examined, a Hybrid analysis method was also examined, that takes forces from column faces in the FEM model, and the stress formula from the Muster simplified analysis method. When compared to test data from the UBC TallWood House testing program, the design cases assessed had a significant margin of safety when using current strength specified values in the PRG 320, with ratios of strength to capacity in the range of 3.4-5.2. Even using strength values from experiment, the ratio between failure load and predicted capacity was 3.3 for the Muster Simplified method.

In both the sample design and parametric study, it was noted that the stiffness of column supports had a significant impact on the FEM predicted forces and stresses. In the parametric study it was noted simplified methods predicted column face shears that were comparable to FEM predictions, however, it was observed that FEM predictions had a larger spread in data. This indicates that parameters like the out of span width affect results more than simplified methods. In general FEM results were more sensitive to the span perpendicular to load, e.g. width on V_x, than the simplified models.

The parametric study also showed that the support stiffness was the parameter that had the most influence on support shear forces. The length, width, and number of spans also had a significant effect on shear force, while the type of panel used and support size did not. The parametric study showed that the support stiffness also

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had a significant effect on the peak model shear forces of all studied parameters. The ratio of peak to average stress varied significantly for different panel layups and support stiffnesses. A large difference in behaviour at the supports was observed between different panels in the parametric study. Because support stiffness was observed to have a significant influence on shear stress distribution in FEM models, further study is recommended to refine this stiffness for a wider set of parameters and ensure its interaction with factors such as k_A is appropriate.

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