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SEISMIC PERFORMANCE OF STRUCTURAL FRAMES AND JOINTS IN WOOD TALL BUILDINGS

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There has been a growing interest in using wood in tall buildings by employing cross-laminated timber (CLT) in a structure's moment frame system. However, still much needs to be learned about the performance of CLT frame systems subject to lateral loads such as earthquakes. The ductility needed in CLT frames in seismic load environments is primarily provided through connections. Thus, a proper connection detail is required to ensure an adequate seismic response, especially at extreme loads. Current strength specifications of connectors are mainly applicable in cases of pure tension or shear; while in real applications, connections are subjected to a combination of these loads. This paper is part of analytical and simulation studies on seismic performance of wood structural frames being conducted at Illinois Institute of Technology. These studies include simulations to investigate: (1) Moment-resistance capacity of CLT panels; and (2) Resistance capacity of typical joints used in CLT frame systems. Limited experimental data available on a 6-story wood frame system are considered as a means of verification of the aforementioned analytical investigations. Focus of this paper is specifically on the significance of interaction between tension and shear as reported in literature. Several interaction equations as implemented in current analytical models for pushover analysis of CLT frames are explained and their adequacy to predict the nonlinear behavior of CLT panels under lateral loads is discussed.

Keywords: Pushover analysis, CLT, Interaction equations, Failure criteria, Connectors, Nonlinear behavior.

1 INTRODUCTION

Currently, there is a growing interest in using wood as a primary material for tall buildings (Mohammadi and Ling 2017). Tall buildings made solely of timber materials are those in the 8 to the 9-story range. In combination with other materials, taller constructions are also possible and have been considered by structural engineers and architects both in real applications and as concept designs (e.g., Timber Tower Research Project, *Skidmore, Owings, and Merrill*, Chicago, IL, 2013). The interest in using wood as an alternative material for tall buildings is due to several unique properties of wood including (1) large strength to density ratio; (2) flexibility in architectural design; and (3) favorable resistance to fire.

Multi-storey timber buildings belong to two main construction types - one is the conventional light-frame constructions that consist of combinations of sheathing and framing elements connected with nails or staples; the second type is the mass timber constructions such as cross-laminated timber (CLT or Xlam). CLT is an engineered wood made of layers of lumber boards that are stacked crosswise (usually at 90-degree angle) and glued together. CLT is mostly used for mid and high rise buildings due to its dimensional stability, stiffness, and rigidity. Moreover, CLT possesses attractive features such as lightweight, low cost and favorable thermal and vibrational performance that make it an alternative material for tall building construction as a stand-alone material or in combination with steel and concrete. Experimental and analytical investigations have shown that CLT structures have a remarkable seismic performance with little or no residual deformations and minor damage. Entire CLT buildings and assemblies under monotonic and cyclic loads have also been tested (Popovski *et al.* 2010, Tomasi and Smith 2014, Gavric *et al.* 2015a). Moreover, uniaxial and 3D shake table tests were performed on CLT structures and substructures to simulate the ground motion of an earthquake (Ceccotti and Follesa 2006, Dujic *et al.* 2010, Hristovski *et al.* 2012, Ceccotti *et al.* 2013). These studies showed that the main factors that affect the seismic performance of CLT panels are: (1) the mechanical properties number and type of connectors (2) aspect ratio of the panels (3) boundary conditions (4) the presence of vertical joints or openings in the substructures (5) vertical load. It is noted that the seismic performance of timber structures is not solely derived from the performance of CLT only; an appropriate design for the structural component and details is required to ensure sufficient performance (Sustersic *et al.* 2015, Casagrande *et al.* 2017, Reynolds *et al.* 2017). In addition, several modeling techniques have been proposed to simulating the behavior of CLT structures and sub-assemblies subject to seismic loads (Filiatrault and Folz 2002, Hummel 2017).

2 MECHANICAL MODELS USED FOR CLT PANELS

CLT panels and walls have a significant in-plane stiffness, thus the predominant kinematic behavior is rocking, sliding or combination of both, while shear and bending deformations are significantly lower (Gavric *et al.* 2015a). Therefore, the energy dissipation is mainly achieved through joints connecting structural members. In general, the behavior of the CLT wall and connection is nonlinear. Consequently, to predict the force-displacement of a CLT wall system, a force-displacement relationship for each connector (angle brackets, hold-downs, screws) must be selected. In our studies, a trilinear force-displacement relationship is adopted for each connector (Figure 1) and calibrated using CLT connections test results based on EN 12512 provisions (Gavric *et al.* 2012, 2015b).

Figure 2 shows the kinematic model adopted for CLT wall. This model neglects any in-plane deformations of the CLT panel and ignores any interaction between rocking and sliding.

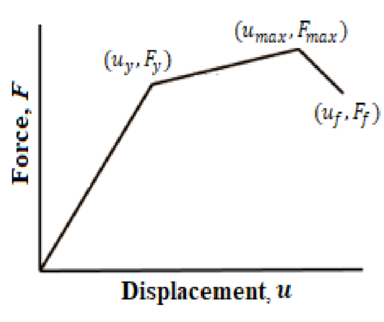


Figure 1. Trilinear force-displacement relationship that is chosen to model the CLT connectors (Gavric *et al.* 2015a).

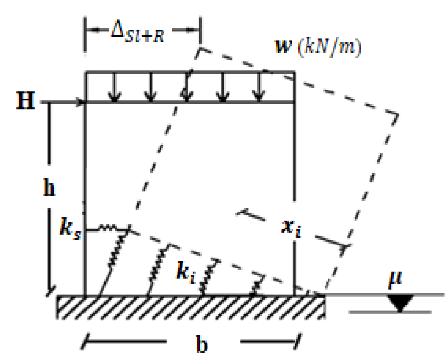


Figure 2. Kinematic model adopted for CLT wall subjected to lateral and vertical loading.

The rocking component of the displacement, Δ_R is a function of the wall rotation and height, $\Delta_R = h\theta$. Considering the force in the spring, i to be linear elastic, it can be obtained from the wall rotation and distance from the rotation point, $F_i = x_i\theta$. Solving the equilibrium at the lower right corner results in:

$$Hh - w\frac{b^2}{2} - F_i x_i = 0 \tag{1}$$

From Eq. (1) the rocking lateral displacement component is equal to (Eq. (2)):

$$\Delta_R = \frac{\left(Hh - \frac{wb^2}{2}\right)h}{\sum_{i=1}^{n} k_i x_i^2} \tag{2}$$

Applying a simple friction model, where the wall starts sliding when the horizontal force exceeds the force of static friction, the friction force equals:

$$F_\mu = \mu\, w\, b \tag{3}$$

The sliding component H is obtained from Eq. (4):

$$H - k_s\Delta_{Sl} - F_\mu = 0 \tag{4}$$

From Eq. (4), the sliding lateral displacement component Δ_{Sl} equals (Eq. (5)):

$$\Delta_{Sl} = \frac{H - F_\mu}{k_s} \tag{5}$$

Considering the connector force in tension, $F_i(d_i)$ and in shear, $F_{si}(d_i)$, the nonlinear wall resistance as a function of displacement, H (D) equals to the minimum of Eq. (6a) and Eq. (6b):

$$H(D) = \sum_{i=1}^{n} \frac{F_i(d_i)x_i}{h} + \frac{wb^2}{2h} \tag{6a}$$

$$H(D) = \sum_{i=1}^{n} F_{si}(d_i) + F_\mu \tag{6b}$$

3 FAILURE CRITERIA

Since the behavior of CLT walls subjected to a lateral load is predominantly consists of rocking and sliding, a two-directional loading will require connection elements such as hold-downs and angle brackets. And as such, criterion frequently used to predict the ultimate capacity and failure is based on maximum normal stress and maximum shear stress (Steeve and Wingate 2012):

$$\sigma_{\max} = \frac{\sigma}{2} + \sqrt{\left(\frac{\sigma}{2}\right)^2 + \tau^2} \tag{7}$$

$$\tau_{\max} = \sqrt{\left(\frac{\sigma}{2}\right)^2 + \tau^2} \tag{8}$$

Dividing Eq. (7) and Eq. (8) by the maximum normal and shear strength respectively yield to Eq. (9) and Eq. (10):

$$\frac{R_T}{2} + \sqrt{\left(\frac{R_T}{2}\right)^2 + (KR_S)^2} = 1 \tag{9}$$

$$\sqrt{\left(\frac{R_T}{2K}\right)^2 + (R_S)^2} = 1 \tag{10}$$

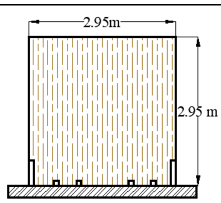
Where $R_T = {}^{\sigma}/_{\sigma_{max}}$, $R_S = {}^{\tau}/_{\tau_{max}}$ and $K = {}^{\tau_{max}}/_{\sigma_{max}}$

To study the influence of different *K* values in the analytical model to predict the force-displacement relationship of CLT wall, several models are investigated:

- Model 1: Maximum uncoupled tension and shear stress (no tension-shear interaction).
- Model 2: *K*=0.5, this value is commonly used in interaction equations for members loaded in combined tension and compression.
- Model 3: Maximum normal stress criterion with *K*=1/5 as an interaction equation for hold-downs and maximum shear stress criterion with *K*=1.5 as an interaction equation for the bracket angles. It is reported in Gavric *et al.* (2015b) that the tensile strength of the hold downs is almost five times greater than its shear strength. However, the average uplift resistance of angle brackets was 63% of the shear resistance for wall panel connections and 87% for a wall-foundation connection.
- Model 4: Maximum uncoupled tension stress of hold-downs and maximum shear stress criterion with *K*= 1.5.

The accuracy of the above models to predict the actual behavior of CLT walls under lateral loads has been verified by comparing the models' predictions of the maximum lateral resistance and the ultimate displacement capacity to experimental data (Gavric *et al.* 2015a). The ratio between experimentally measured data and the analytical predictions are displayed in Table 1.

Table 1. The maximum lateral resistance (F_{max}) and the ultimate displacement capapacity (Δ_f) of tested CLT shear walls to that of the analytical models (Gavric *et al.* 2015a).

Wall panel test configuration	**Test Description**	**Model.1**		**Model.2**		**Model.3**		**Model.4**	
		$F_{\text{max-exp}}$ / $_{\text{max-ana}}$	$\Delta_{\text{f-exp}}$ / $\Delta_{\text{f-ana}}$	$F_{\text{max-exp}}$ / $F_{\text{max-ana}}$	$\Delta_{\text{f-exp}}$ / $\Delta_{\text{f-ana}}$	$F_{\text{max-exp}}$ / $F_{\text{max-ana}}$	$\Delta_{\text{f-exp}}$ / $\Delta_{\text{f-ana}}$	$F_{\text{max-exp}}$ / $F_{\text{max-ana}}$	$\Delta_{\text{f-exp}}$ / $\Delta_{\text{f-ana}}$
 HTT22 hold-downs and BMF 90×48×3×116 mm angle brackets.	Test.1: n=2, m=4, w= 18.5kN/m	1.00	1.1	1.28	1.15	1.07	1.1	1.01	1.1
	Test.2: n=2, m=4, w= 18.5kN/m	1.02	0.9	1.3	0.94	1.09	0.92	1.03	0.9
	Test.3: n=2, m=2, w= 18.5kN/m	0.84	1.07	1.07	0.91	0.88	1.03	0.87	0.87
	Test.4: n=2, m=4, w= 9.25kN/m	1.08	1.08	1.4	1.1	1.2	1.09	1.08	1.08
Number of hold-downs = n Number of angle brackets = m	$\bar{x}$	0.99	1.04	1.26	1.03	1.06	1.04	1.00	0.99
	COV %	10.4	8.9	11.0	11.5	12.5	8.0	9.0	12.1

Model 1 and Model 4 (Table 1) predicted the CLT maximum resistance and ultimate capacity relatively well. However, in Test 3 both models underestimated the maximum lateral resistance reaching almost a 15% difference. Model 2 significantly underestimated the maximum lateral resistance and showed about 26% lower values than those from experimental results. These underestimated values are rather remarkable and reported to be due to the predicted premature failure of hold-downs in shear as opposed to the real performance where plasticization does not simultaneously apply in tension and shear. Model 3 predictions are relatively good, and calibrating K based on boundary conditions and connections type could lead to better performance of this model.

4 DISCUSSIONS AND CONCLUSIONS

Our study in understanding the seismic behavior of frames made up of CLT panels and walls is in progress. Currently, our research is focusing on: (1) providing an insight into lateral load resistance capability of CLT panels when used in multi-story frame systems; (2) identifying the type of structural panels and joints that offer favorable performance for use in tall buildings; and (3) identifying other areas where one may pursue using wood, as sole material or in combination with other materials, in tall buildings. The interaction models for tension and shear reported in the literature and briefly reviewed in this paper are especially useful in modeling wood frame systems in nonlinear push-over analysis and being pursued in our studies. Limited experimental data available on 6-story wood frame system is also being considered as a means of verification for the results from our analytical investigations (Van de Lindt *et al.* 2010).

The ductility needed for frames can be achieved through a proper design for connections and their components (such as hold-downs and angle brackets). As reported in literature, the traditional interaction equation of shear and tension should not be used for hold-downs since it predicts its premature failure and underestimates the behavior. The literature reports that it is a good practice to include the contribution of angle brackets to resist uplift.

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