

Prototyping a Passively Self-Centering Cross-Laminated Timber Rocking Wall System: Analytical and Experimental Investigation

Marco Lo Ricco, P.E., S.E., Dr. Al Ghorbanpoor, P.E., Dr. Shiling Pei, P.E., Doug Rammer, P.E., Marshall Begel, P.E., James Bridwell, M.S., and Reid B. Zimmerman, P.E., S.E.

Abstract:

Cross-Laminated Timber (CLT) garners international interest with increasingly taller wood structures built of robust, rigid panels. For seismic resilience of multistory platform construction, this study prototypes a CLT rocking wall system with an elliptical boundary profile. Curvilinear cuts to the load-bearing edges of a rectangular CLT panel produce smoother lateral response to earthquakes, with gravity-driven passive re-centering ability and reduced crushing damage at wall corners. Stiffness and ductility of the rocking wall can be tuned by proportioning the ellipse profile. Connections between the rocking wall and diaphragm can be designed to transfer shear and influence rocking behavior. This paper presents the design and full scale cyclic testing performance of a prototype CLT rocking wall with two different types of connections. Hysteresis plots of the test results and idealized analytical models show good agreement between design assumptions and actual properties of the pendulum rocking wall system.

Keywords: Cross-laminated timber, seismic resiliency, rocking walls, rolling inverted pendulum, slip-friction inverted pendulum

Introduction

Cross-Laminated Timber (CLT) panels, since originating in Europe over two decades ago, have revolutionized the design of modern heavy timber structures worldwide (Pei et al. 2016). CLT production assembles large volumes of lumber, in at least three orthogonal layers, for pressing and adhesion into structural composite panels that work as slab, deck, or wall systems (ANSI/APA PRG 320 2018). As a composite panel resembling precast concrete in size, CLT delivers economic, environmental, and structural performance benefits leading to more effective use of wood, smaller carbon footprints, and weight savings in commercial construction. Nearly a decade ago, Murray Grove, a 9-story apartment building in London, drew international focus on CLT construction with its fully

panelized platform structure rising several stories taller than conventional timber framing (TRADA 2009). Shake-table tests of a 7-story CLT platform assembly, however, called seismic performance into question, after registering peak acceleration of 3.8 g in the upper floor (Ceccotti et al. 2013). Early renditions of CLT platform construction, therefore, need new design approaches before entering earthquake hazardous regions.

CLT construction promises multifaceted revitalization for the western United States — where sizable forest products economies, sustainable building markets, and seismic hazards coincide. Despite inherent lightweight capability to reduce seismic excitation mass, few CLT buildings have been constructed in earthquake regions, because

continued on next page

alternatives to codified structural schemes carry an extra burden of proof (ASCE/SEI 7 2017). Design of the 12-story Framework building earned permitting in Portland, Oregon, for its vertically post-tensioned CLT rocking wall lateral system and flexible mass timber frame, via validation testing and peer review, yet the project remains unrealized for lack of financing (Zimmerman and McDonnell 2017, Zimmerman and McDonnell 2018). In contrast, the 18-story Brock Commons, built with a predominantly mass timber structure in Vancouver, British Columbia, expedited approval of the lateral force-resisting system by pairing the timber gravity system with prequalified reinforced concrete cores (Fast and Jackson 2017). Though currently less expedient, treating CLT as an emerging technology with its own advantages is leading to better lateral systems.

To expand the effectiveness of CLT lateral systems, the “Structural Research Drive” maps seismic development with *code minimum*, *code plus*, and *resilient* hierarchical tiers of performance expectations (Pei et al. 2014, Pei et al. 2016). Determining seismic design parameters for CLT, via rigorous FEMA P695 qualification procedures, fulfills essential code requirements that lay a foundational tier of objectives (Applied Building Technology Council 2009, Amini et al. 2014). CLT systems, however, must target the pinnacle of seismic performance with *resilient* solutions to extend the most compelling approach (Pei et al. 2014, Pei et al. 2016). Recent tests of vertically post-tensioned rocking CLT walls, adapted from pioneering research in precast concrete, offer ways to achieve resilience (Dunbar et al. 2014, Ganey et al. 2017). Shake-table tests of a 2-story timber structure of vertically post-tensioned walls, diaphragms, and glulam framing, produced no significant damage, further demonstrating resilience with an entire assembly (Pei, van de Lindt, et al. 2018). Future shake-table tests will soon examine 10-story timber archetypes featuring various vertically post-tensioned CLT walls (Pei, Berman, et al. 2018).

For vertically post-tensioned rocking wall systems to work effectively, unbonded tendons must bear on top of walls and anchor into foundations to safeguard walls from overturning. Post-tensioned walls generally span full building height to best support all diaphragms laterally. CLT panels usually extend past several floors to minimize horizontal wall joints, and connections between wall panels and intersecting floors must enable rocking without constraint. Rocking walls, moreover, must be slenderly proportioned to take advantage of the counterintuitive “scale effect” of rectangular inverted pendulum blocks, first elucidated by Housner (1963, p. 417). Pivoting on the base corners of rectangular walls, in fact, drives the rocking mechanism and opens gaps that engage vertical tendon restraints. Vertically post-tensioned rocking walls, therefore, typically entail *rectangular* panels and *balloon* construction.

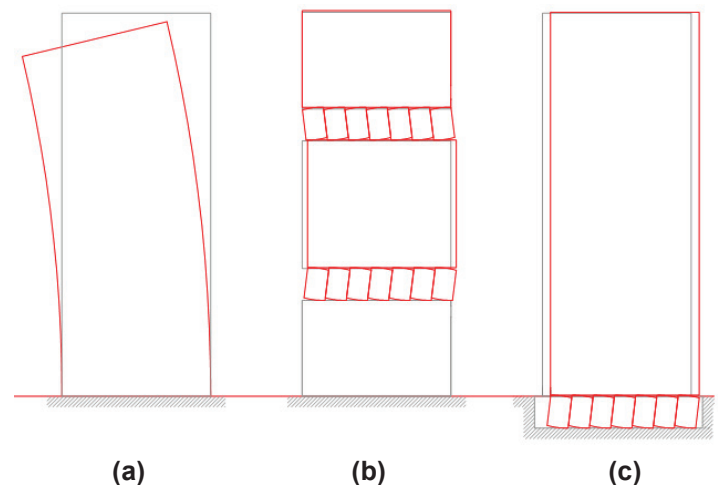


Figure 1: Elevations views of (a) Fixed base, (b) multiple soft-story rocking, and (c) soft-story rocking base schemes.

An alternative *elliptically profiled* rocking wall panel can integrate with *platform* construction to self-center without post-tensioning. Platform assembly, in contrast to balloon, simplifies construction sequencing and connection details with one-story tall modular components. Figure 1 illustrates three building schemes, in elevation views, for

comparison. View (a) cantilevers from the ground and funnels forces to the base, representing how most commonly designed buildings behave under lateral effects. Views (b) and (c) introduce soft-stories designed to flex laterally and self-center under superstructure weight. Soft stories can divide superstructures into multiple blocks, as shown in view (b), or support the entire superstructure above grade level, as shown in view (c). Because soft stories undergo virtually all lateral inter-story drift, superstructure blocks can comprise of any rigid construction that CLT walls can support at the rocking level. The following sections of this article detail mechanics and performance of a one-story tall elliptical rocking wall prototyped in CLT, with focus on timber contact boundaries critical to the pendulum system.

Shaping an Elliptical Rolling Pendulum Made of Timber

Figure 2 illustrates the concept of the passive self-centering platform CLT wall proposed here. Early research of construction assemblies demonstrated that various boundary conditions alter the rocking

response of rectangular CLT shearwall panels (Dujic and Zarnic 2006). While rectangular panels may rock in a platform assembly, the rotated panel of Figure 2 shows corners protruding into ceiling and floor space. Curving load-bearing edges whittles the corners to facilitate panel rotation. Study of circular, elliptical, and parabolic half disks has demonstrated that geometric properties, such as radius or proportions of width and height, control various characteristics of inverted pendulum rocking (Mazzoleni et al. 2015). Within a platform assembly, ellipse eccentricity uniquely leverages superstructure weight to provide restoring moment. This project prototypes elliptical geometry to take advantage of distinctive passively self-centering features.

Elliptical Rolling Pendulum Mechanics

Figure 3 illustrates a panel that fits the middle of an elliptical profile to adapt ellipse geometry to wall construction. The mathematical model, forming the basis of Figure 3, was initially developed to analyze elliptical rolling rods as seismic isolation (Jangid and Londhe 1998, Londhe and Jangid 1999). The

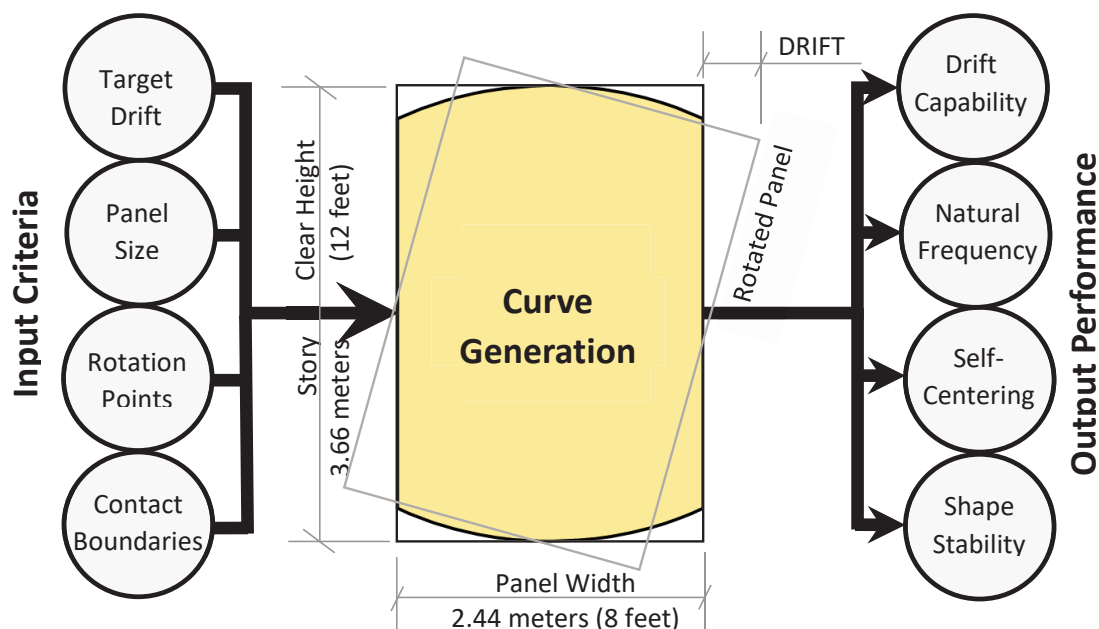


Figure 2: Shaping CLT Panels for Rocking Response to Earthquakes

original application placed a multistory structural frame atop elliptical rolling rods of varying eccentricity, e , quantified by:

$$e = \sqrt{1 - \frac{b^2}{a^2}}$$

where a = semi-major axis length and b = semi-minor axis length of the parent ellipse. Cross-sections of the rolling rods formed complete ellipses, but mechanics of the rolling pendulum still apply to the truncated panels shown in Figure 3.

Two fundamental assumptions establish the rocking wall response diagrammed in Figure 3. First, the floor and ceiling that sandwich the rocking wall should remain horizontal, as rigid bodies throughout the rolling response. Structural design practice commonly fulfills this rigid-body condition with beams and superstructures conforming to

standard serviceability criteria. Following rigid-body constraint, view (a) assigns 3 independent degrees of freedom to the panel, x_r , y_r , and θ_r , and only 2 translational degrees of freedom, x_b and y_b , to the dependent supported mass.

Secondly, elliptical arc length, AA'' or BB'' , equals the respective straight horizontal segment of ceiling, AA' , or floor, BB' , between contact points shown in Figure 3 view (b). Relative slip between the rocking wall and the bounding floor and ceiling should be either eliminated or neutralized. Slippage, if unconstrained, threatens to introduce residual displacements that negate the self-centering benefits of the pendulum system. Because friction between the rocking wall and bounding floor and ceiling presents uncertainty, traction cannot completely enforce non-slip kinematic conditions. To ensure that walls roll without slipping, *displacement restraints* connect the panel to the floor and

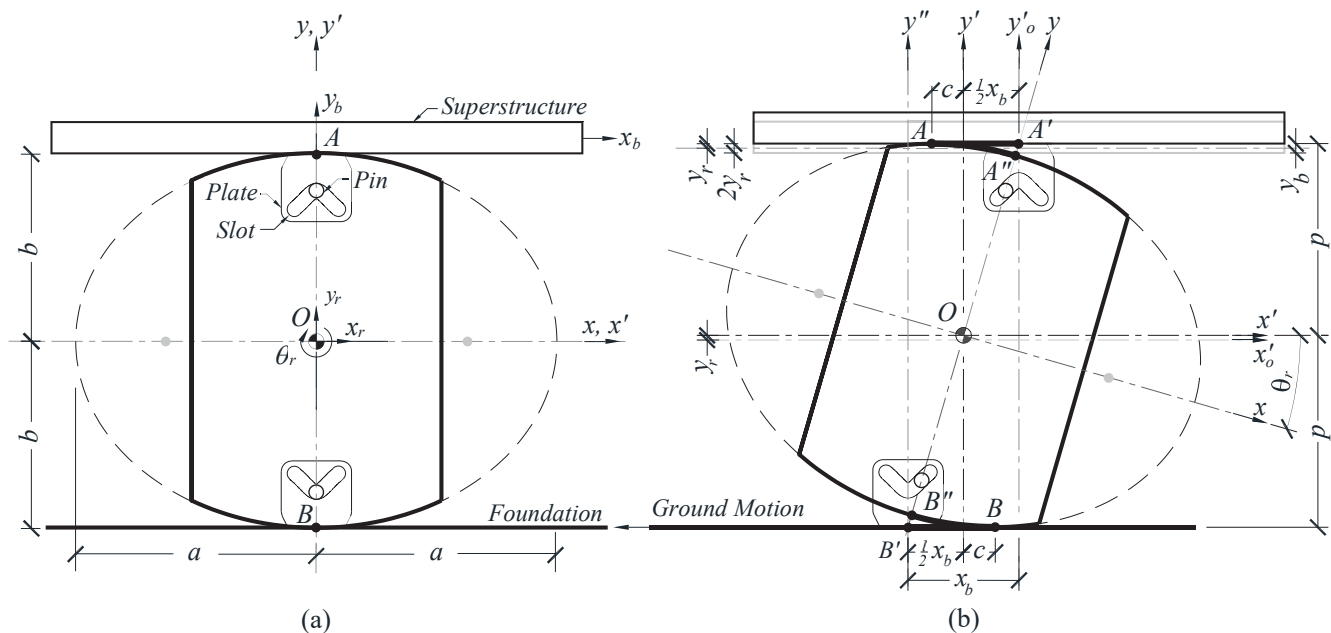


Figure 3: Rolling Elliptical Pendulum Model, Adapted to Load-Bearing Wall Panel from (Jangid and Londhe)

ceiling with pins and slotted plates as depicted in Figure 3. Faceplates connected to ceiling and floor beams guide panels to roll in plane, with slots that direct pin travel. If slip occurs in the Figure 3 configuration, pins bear against all components, panel and faceplates, but with minimal connection constraint. A similar shear key connection concept approximated paths of pin travel for a rectangular rocking timber panel and calculated effects of connection constraint (Loo et al. 2014). The v-shaped slots pictured in Figure 3 take shape by tracing connection points of the rolling ellipse through a desired range of lateral displacements, to minimize constraint.

Figure 4 diagrams main forces on the free-body of a rolling panel configured, according to Figure 3, to transfer shear primarily via traction at load-bearing edges. Summation of moments about point O for

equilibrium arrives at the lateral force estimation, F_L , offered by Jangid and Londhe:

$$F_L = W \left(\mu_r \operatorname{sgn}(\dot{x}_b) + \frac{c}{p} \right)$$

where W is the superstructure weight transferred through the directly supported mass; μ_r is a rolling friction coefficient; the *signum function* specifies the direction of horizontal traction forces, and dimensions c and p respectively indicate half the moment arm distance of the vertical restoring moment and horizontal overturning moment induced by traction. Neglecting panel weight typically results in conservative design or no appreciable effect, because panel weight, w_p , typically adds to the restoring moment and amounts to minute fractions of the total supported weight.

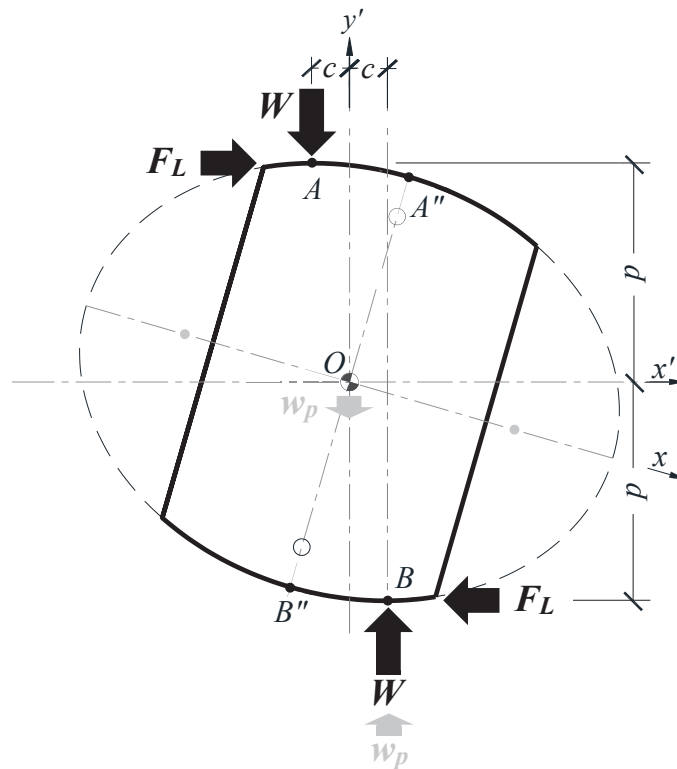


Figure 4: Free-Body Diagram of Edge Contact
(rolling friction and inertial forces not shown for clarity)

Figure 5 plots an idealized hysteresis loop that normalizes superstructure weight, W , with lateral force F_L , expressed as a function of the lateral drift ratio, $x_b/2b$:

$$F_L \left(\frac{x_b}{2b} \right) = W \left(\mu_r \operatorname{sgn}(\dot{x}_b) + \frac{c}{p} \cdot \frac{x_b}{2b} \right).$$

Rolling friction coefficient, μ_r , determines the y-intercepts of the normalized lateral force function and depth of the hysteresis loop. Because rolling friction dissipates little energy, Figure 5 charts a narrow hysteresis loop estimating the magnitude of rolling friction at 1% of overburden weight. The c/p ratio of dimensions, labeled in Figures 3 and 4, charts stiffness that behaves nonlinearly but only slightly so within the practical range plotted by the solid lines. A circle, or ellipse of zero eccentricity, acts as a roller providing virtually no lateral stiffness. An ellipse of maximum eccentricity,

approaching unity, essentially matches the stiffness of a rectangular panel. Figure 5 presents an ellipse of eccentricity, e , equal to 0.73, to demonstrate properties of a prototype panel with high lateral drift capacity and low stiffness, relative to a conventionally anchored shearwall.

Elliptical Slip-Friction Pendulum Mechanics

Figure 6 illustrates a vertically slotted connection plate that transfers story shear primarily through horizontal pin constraints, instead of relying on traction. Horizontal constraint forces the panel to rotate about the pins and contact surfaces to slip along distances, x_c , dimensioned in Figure 6. Lateral displacement of the newly constrained configuration, x_p , relates to the previous rolling mechanism via equation: $x_p = x_b - 2x_c$. Though the net lateral displacement of x_p is less than x_b , for a given panel rotation θ_r , the path effectively traveled by the constrained system measures $2x_c$ greater than x_b .

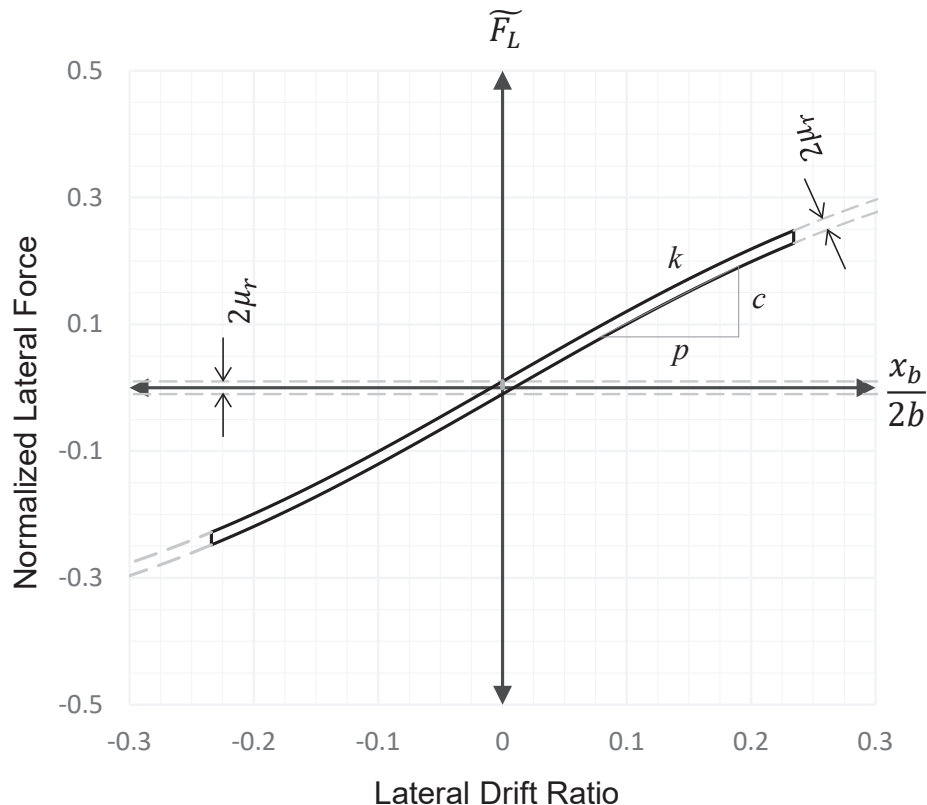


Figure 5: Normalized Lateral Stiffness Plot for No-Slip Rolling Elliptical Panels of Eccentricity, $e = 0.73$

continued on next page

Proportioning similar triangles expresses the cumulative path of travel as:

$$x_b + 2x_c = \left(\frac{2p}{d \cos \theta_r} - 1 \right) x_p.$$

Pushing panels with greater force through the same rotation, θ_r , but with less translation, increases effective lateral stiffness. Sliding, furthermore, creates frictional forces, F_s , as shown in Figure 7. In addition to friction between timber and steel, steel-to-steel contact between pins and face plates generates sliding friction, f_p , and pins spinning within pipe bushings embedded in the timber panel generate rotational friction. Globally, the model does not consider the force couple produced by f_p as restorative but aggregates sources of frictional damping into a sum of rolling and sliding, μ_r and μ_s , coefficients. Pins counteract sliding forces and transfer story shears on a moment arm reduced from $2p$ between edge contact points to $2d \cos \theta_r$ between pin centers. Considering the effective travel and moment arm between pin centers, yields

pin force, F_p , expressed as a function of net lateral displacement, x_p :

$$F_p(x_p) = W \left((\mu_r + \mu_s) \operatorname{sgn}(\dot{x}_p) + \frac{c}{d \cos \theta_r} \left(\frac{2p}{d \cos \theta_r} - 1 \right) x_p \right).$$

Figure 8 normalizes lateral force, F_p , and plots the relationship with respect to lateral drift ratio, $x_p/2b$. Comparisons of Figure 8 with the no-slip traction of Figure 5 reveal significant differences. Sliding friction increases the y-intercept of Figure 8 so that the sum of frictional coefficients totals 10% raising the rocking threshold and adding hysteretic damping. Connection constraint, furthermore, significantly reduces the range of lateral displacements. Within a practical range, the slip-friction system behaves nearly linearly, but the dashed trends of Figure 8 indicate a stiffening effect. Pin contact with vertical limits of the slots should, by design, stiffen and restrain the system from venturing beyond the practical, solid-line regions of the mathematical functions.

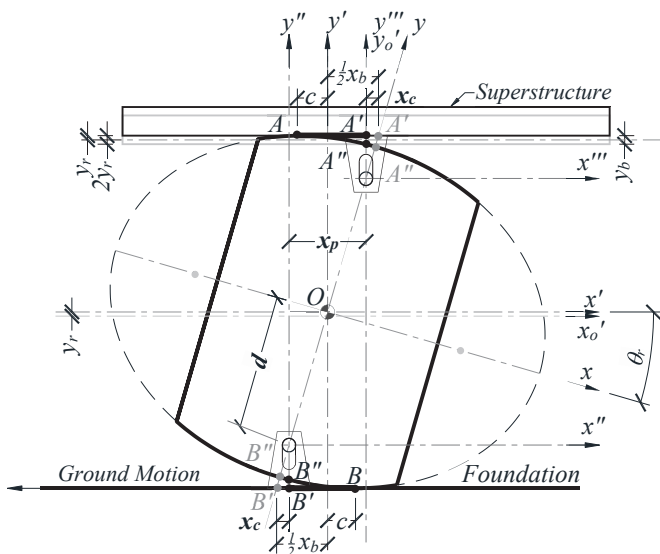


Figure 6: Slip-Friction Pendulum Model

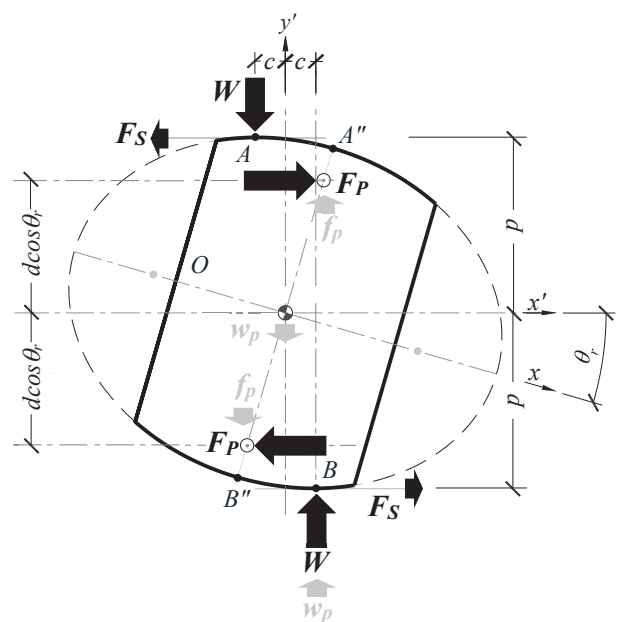


Figure 7: Free-Body Diagram Including Pin Contact (rolling friction of the panel, rotational friction of the pins, and inertial forces not shown for clarity)

continued on next page

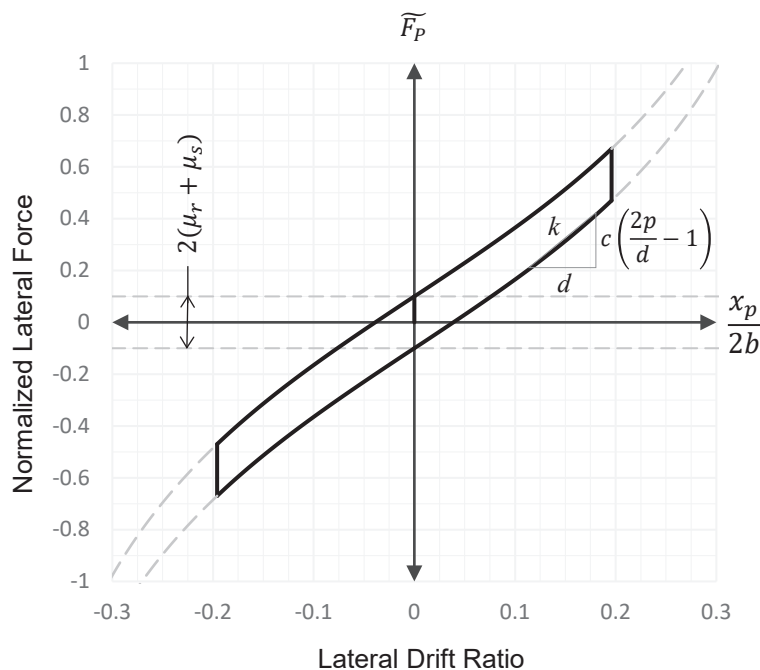


Figure 8: Normalized Lateral Stiffness for Slip-Friction Rolling Elliptical Panels of Eccentricity, $e = 0.73$

Prototype Performance

The *No-Slip Traction* and *Slip-Friction* rocking models hypothesize that:

- Rigid-body mechanics can estimate lateral stiffness properties of full-scale CLT panels with reasonable accuracy,
- Slot shape of the faceplate connections determines the rocking mode, by varying connection constraint, and
- CLT can endure concentrated, high-pressure compression bearing contact and shear induced by friction, with only localized damage while maintaining overall structural stiffness.

Prototype Testing

Figure 9 renders laboratory test apparatus around a full-scale, 5-ply CLT prototype—measuring 2.44 m by 3.66 m by 0.170 m (8 ft by 12 ft by 6.625 in.) in respective width, height, and thickness—and conforming to North American manufacturing standards for grade V2M1 (ANSI/ APA PRG 320 2018). CNC fabrication cut the CLT panel to an elliptical profile of semi-major axis width, a , and semi-minor axis height, b , respectively equaling 2667 mm (105 in.) and 1829 mm (72 in.). CNC also bored holes through the CLT at pin locations specified by digital drawings and similarly processed steel to plasma-cut the slotted connections diagrammed in

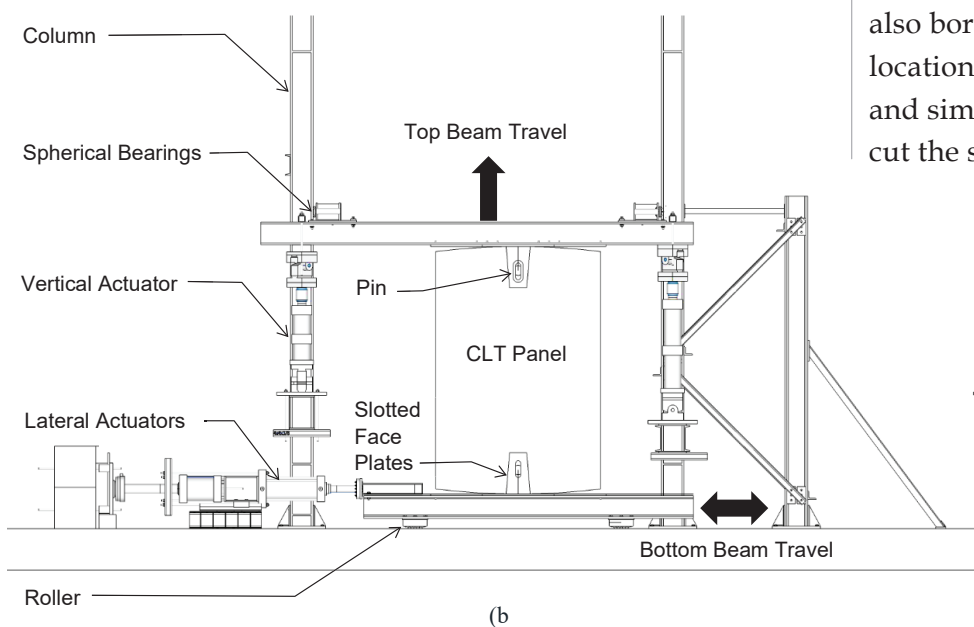


Figure 9: Full-Scale Prototype Test Specimen and Apparatus Elevation View

Figures 3 and 6. A steel support structure framed CLT specimens with fixed and moving parts to create boundary conditions of a rocking soft story. Horizontally oriented actuators applied lateral force to the bottom beam, which glided on industrial rollers within a channel track. Vertically oriented actuators simulated superstructure weight through a feedback process that kept the top beam level and maintained a constant sum of load, while simultaneously adjusting to the moving panel contact point. Accommodating rocking yet preventing overturning, the top assembly of 3 steel beams traveled strictly vertically, gliding on spherical bearings rolling on plates fixed to the column flanges. Precautionary out-of-plane bracing typically did not engage, because wall panels rolled within tolerances set by gaps between braces and panel faces.

Table 1: Cyclic Prototype Test Protocol, adapted from Chapter 17 of ASCE/SEI 7-16

Displacement Step	Number of Cycles
$0.0025h_s$	20
$0.25D_M$	3
$0.50D_M$	3
$0.67D_M$	3
$1.00D_M$	6
$0.75D_M$	$\frac{30S_{M1}}{S_{MS}B_M} = \mathbf{18} \geq 10$

h_s = story clear height = 3.66 m (12 ft)
 D_M = in-plane maximum lateral displacement, limited by panel geometry or external connection limits.
 S_{M1} = the $MCE_{R, 5\%}$ damped, spectral response acceleration parameter at a period of 1s adjusted for site class effects
 S_{MS} = the $MCE_{R, 5\%}$ damped, spectral response acceleration parameter at short periods adjusted for site class effects
 B_M = Numerical coefficient depending on effective damping, β_M , percentage of critical.

Because rocking walls undergo displacement excursions much larger than the the lateral displacement capacities of conventional shearwalls, test protocols emulated quasi-static prototyping procedures for seismic isolation (ASCE/SEI 7 2017). The test protocol of Table 1 generally follows code for seismic isolation but requires simplifying assumptions for first-iteration prototypes. Cycles at the first displacement step represent wind drift limits. Subsequent displacement steps and cycles assess seismic demands.

The same CLT panel specimen was subjected to the Table 1 displacement protocol under two different configurations:

1. *No-Slip Traction Rolling* under simulated superstructure gravity load, or overburden, of 400 kN (90 kips), and
2. *Slip-Friction Rocking* under a low overburden of 133 kN (30 kips).

Synchronized load cells and displacement transducers of the hydraulic actuator system supplied the primary data used to assess properties of the rocking wall system.

Hysteretic Performance

Table 2 summarizes key results of the two test configurations. Figures 10 and 11 respectively plot lateral load versus lateral displacement for *No-Slip Traction Rolling* and *Slip-Friction Rocking* of a one-story tall, 5-layer CLT panel at each seismic displacement milestone of the Table 1 cyclic protocol. In the *No-Slip Traction Rolling* case, two horizontal actuators stroked in series to provide fully reversed cycles approaching ± 432 mm (± 17 in.) and unidirectional cycles up to 864 mm (34 in.), utilizing full lateral displacement capacity of the apparatus. Geometric relationships of the rolling elliptical pendulum, corresponding to Figures 3 through 5, independently shaped the idealized hysteresis superposed on plots of the Figure 10 test data. Estimating the depth of the idealized hysteresis, however, required readings at the onset of lateral displacement to calibrate frictional effects.

Table 2: Summary of Lateral Stiffness Derived from Cyclic Tests of 5-Layer CLT Panel with $e = 0.73$

Connection Configuration	Overburden W		Hysteretic Friction Coeff. μ	Stiffness*				Maximum Tested Lateral Displacement D_M	
				<i>Measured</i>		<i>Normalized to unit of overburden force</i>			
	<i>(kN)</i>	<i>(kips)</i>		<i>(kN/mm)</i>	<i>(kips/in.)</i>	<i>$\times 10^{-4}$ (kN/mm)</i>	<i>$\times 10^{-3}$ (kips/in.)</i>	<i>(mm)</i>	<i>(in.)</i>
No-Slip Traction	400	90	1	0.120	0.68	3.00	7.56	864	34.0
Slip-friction	133	30	10	0.095	0.54	7.14	18.00	717	28.2

*Values derived from test data using secant tangent stiffness at $0.5D_M$

A rolling friction coefficient of μ_r , equal to 1% of the overburden weight estimated frictional damping reasonably well through displacements up to half of the maximum tested limit.

Figure 10 plots (a) and (b) indicate close agreement between theoretical model and experimental results. Slope of actual hysteresis loops over the range of fully reversed cycles matched the mathematical stiffness model, and uniform spacing between top and bottom segments of the hysteresis exhibited constant damping. Recessed steel floor anchorages, not quite flush with the concrete surface, and continuous track mechanisms of industrial rollers supporting the bottom beam contributed minor and regular blips to the recorded data, noise attributable to imperfections of the setup rather than the prototype.

Beginning with the $0.67D_M$ step in Figure 10 plot (c), however, unidirectional cycles inflated hysteresis loops. Local splitting of timber in bearing contact zones, steel yielding along slots of pin bearing, and friction of moving test apparatus all likely added damping that the idealized model did not track. Beyond lateral displacement of $0.75D_M$, the idealized hysteresis model, furthermore, did not account for an observed stiffness increase. Contact of panel corners could have caused the apparent stiffness

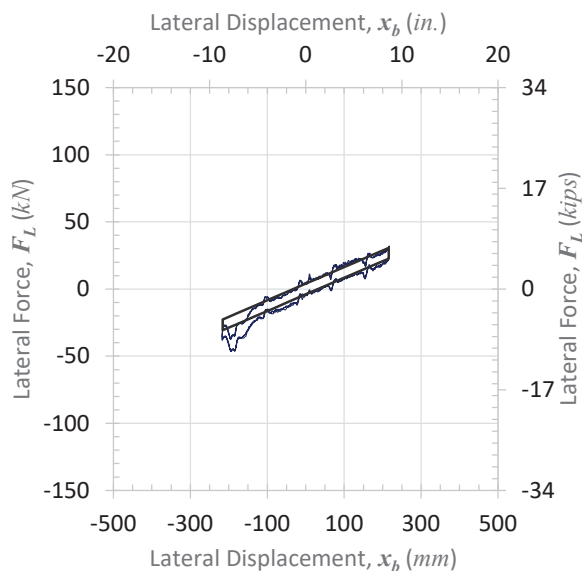
to increase near extreme lateral drift. Throughout the entire tested range, actual hysteresis loops essentially retraced paths, indicating no appreciable damage.

Figure 11 plots the hysteretic performance of the same panel, subsequently configured according to the slip-friction model presented in Figures 6 through 8. The specimen, prior to testing in slip-friction, had experienced the load history of traction rolling charted by Figure 10, which preconditioned the load-bearing edges, but without significant damage to the CLT panel. Peak lateral force, F_P , registered at maximum lateral displacement D_M increased by 8% in comparison with the maximum F_L of Figure 10, while D_M decreased by 17% because of added horizontal connection constraint. Despite higher peak lateral force, the panel configuration of Figure 11 supported only a third of the overburden, 133 kN (30 kips), relative to the previous setup. Sliding added frictional damping forces amounting to 9% of the overburden. The idealized hysteresis of Figure 11, therefore, cumulatively considered rolling and sliding friction with a summation of coefficients:

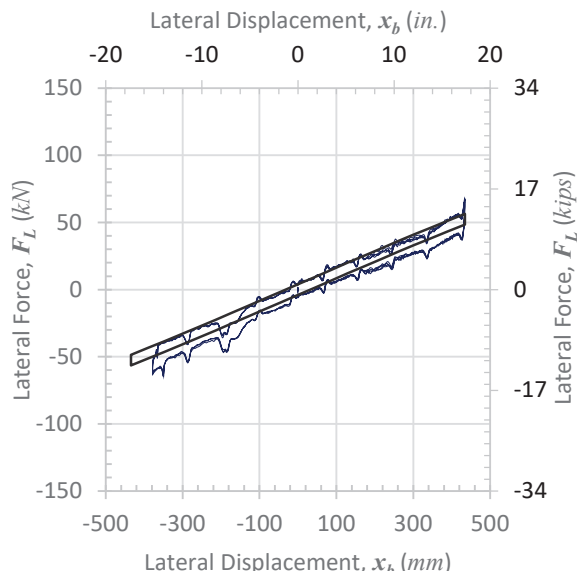
$$\mu = \mu_r + \mu_s = 0.01 + 0.09 = 0.10$$

to model total frictional damping and set the depth of the idealized hysteresis loop.

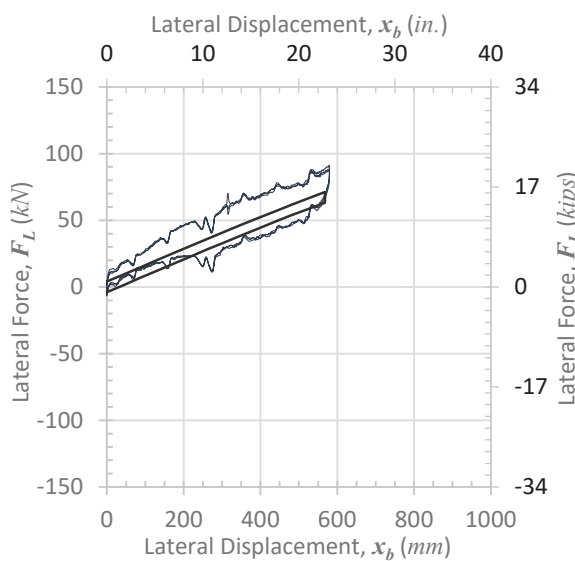
continued on next page



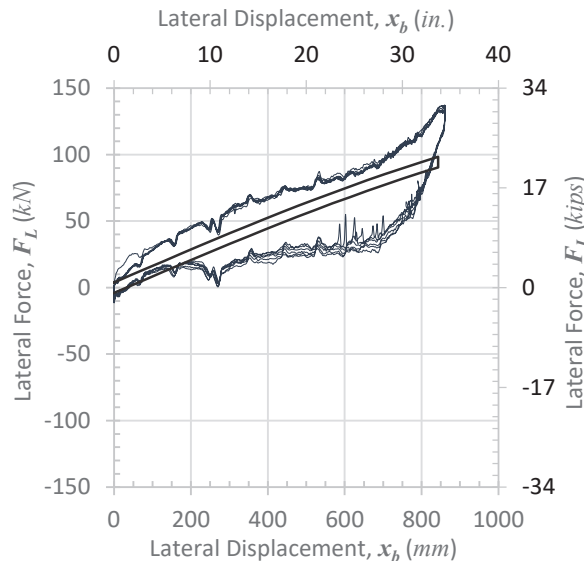
(a) Rolling Pendulum at $0.25D_M$



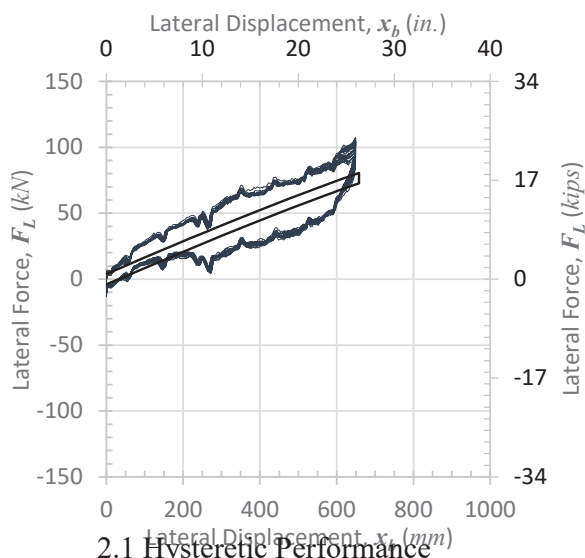
(b) Rolling Pendulum at $0.5D_M$



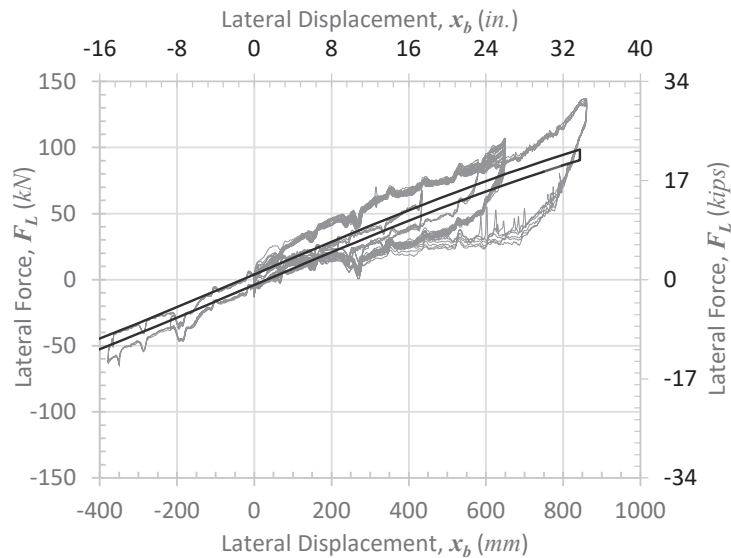
(c) Displacement Step $0.67D_M$



(d) Displacement Step $1.0D_M$



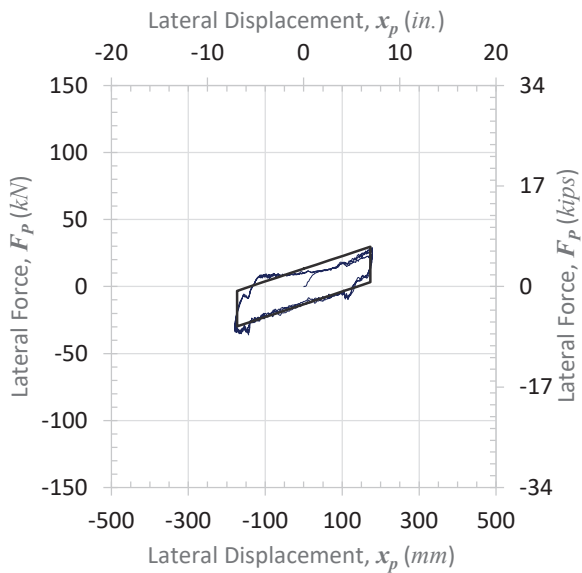
2.1 Hysteretic Performance
(e) Displacement Step $0.75D_M$



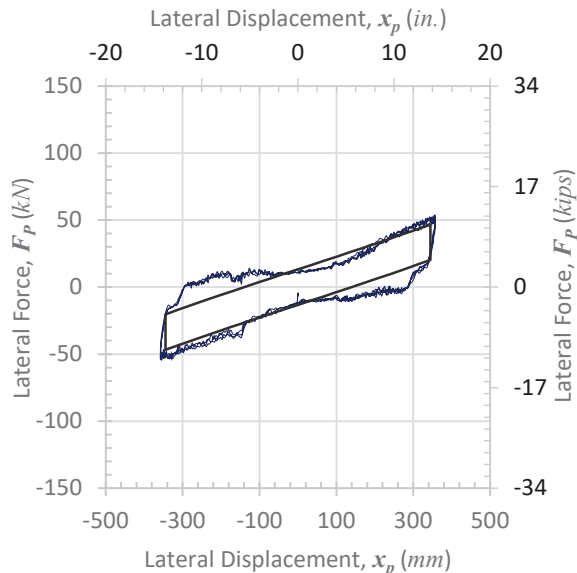
(f) Superposed Displacement Steps

**Figure 10: Hysteresis Plots of No-Slip Traction Configuration of $e = 0.73$
Panel under High Overburden of 400 kN (90 kips)**

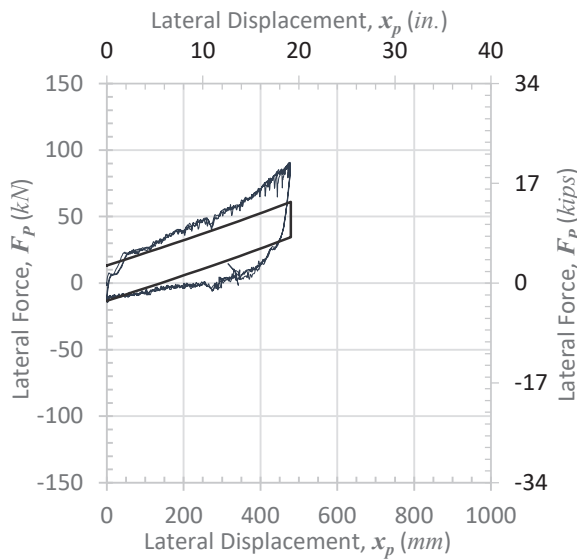
continued on next page



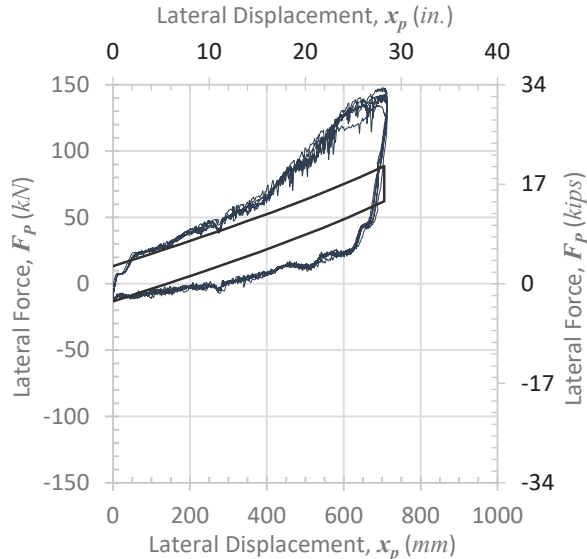
(a) Rolling Pendulum at $0.25D_M$



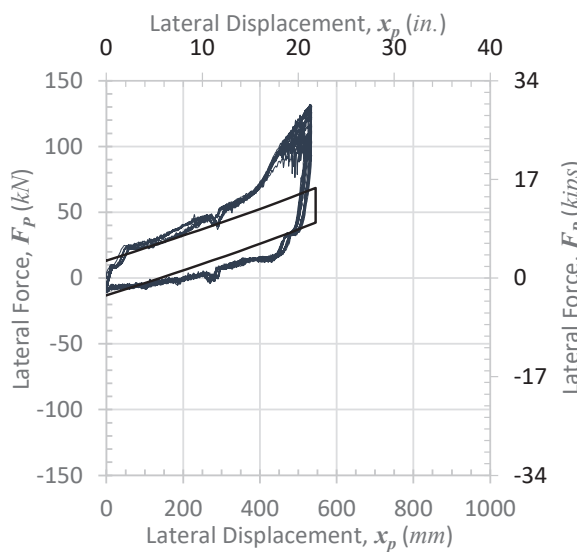
(b) Rolling Pendulum at $0.5D_M$



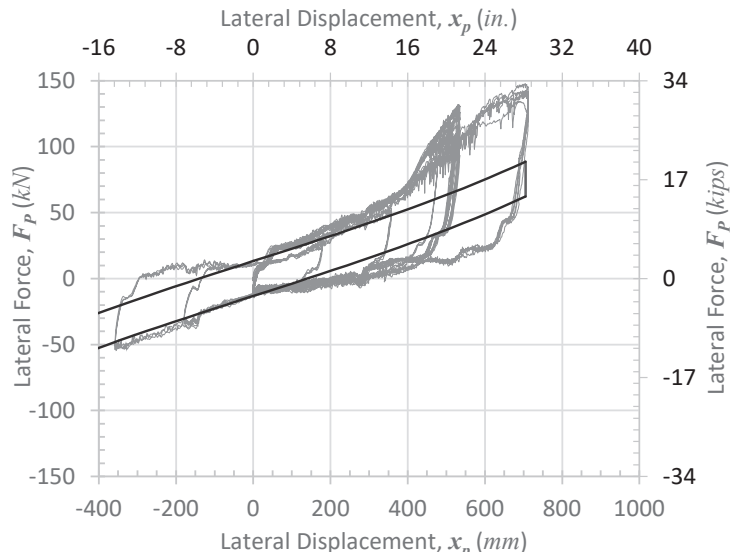
(c) Displacement Step $0.67D_M$



(d) Displacement Step $1.0D_M$



(e) Displacement Step $0.75D_M$



(f) Superposed Displacement Steps

**Figure 11: Hysteresis Plots of Slip-Friction Configuration of $e = 0.73$
Panel under Low Overburden of 133 kN (30 kips)**

continued on next page

Theoretical and empirically measured hysteresis loops indicate reasonable agreement through the ± 356 mm (± 14 in.) range of fully reversed cycles. Plot (a) shows the top right loading branch of the empirical hysteresis dipping below the theoretical model, but the remaining segments behaved predictably. Plot (b) shows a similar dip, but the loading branch of the lower left quadrant charted as expected. Stiffness of the panel in plots (a) and (b) matched slopes of the idealized hysteresis loop through the reversed cycles. Plots of the unidirectional half cycles, however, show that stiffness of the slip-friction system rose more rapidly at outer reaches of lateral displacement than the gradual increase predicted by mathematical modeling.

Observed Damage

Figure 12 photographed the test specimen charted in Figure 10 (d) near extreme lateral displacement at approximately 13° of panel rotation. Figure 13 exhibits staining of the bottom panel edge photographed immediately following unidirectional cyclic tests in the *No-Slip Traction Rolling* configuration. Staining on the left half indicated travel of the contact point. Fully reversed cycles of previous tests left barely discernable signs of contact on the right half. Figure 14 provides a closeup view of the local crushing of timber end grain observed in the longitudinal CLT laminations at the outer reach of the contact. Laminations split when the component of contact bearing force oriented perpendicular to grain exceeded tension strength of the timber. Both traction forces and eccentricity of the wood grain, with respect to gravity loads, grew as the panel rotated. Longitudinal laminations bore the brunt of the damage, because virtually all axial compression in the CLT panel transfers parallel to grain. Cross-laminations, however, restrain splits in the longitudinal laminations.

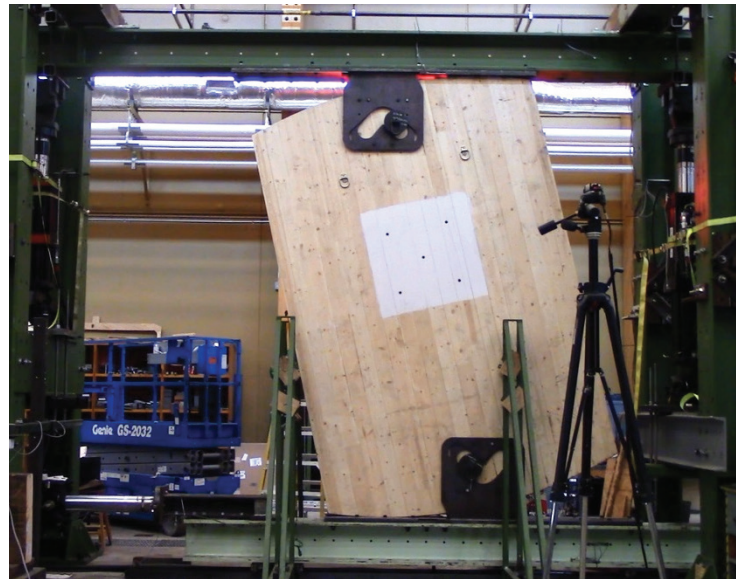


Figure 12: Rolling of $e = 0.73$ Panel at Max Lateral Displacement and Overburden of 400 kN (90 kips)



Figure 13: Bottom Edge of $e = 0.73$ Panel after Unidirectional Rolling Tests



Figure 14: Local Damage of Bottom Edge of $e = 0.73$ Panel after Unidirectional Rolling Tests

Figure 15 photographed the same specimen and panel rotation but in *Slip-Friction Rocking* configuration corresponding to plot (d) of Figure 11. Figure 16 enlarged the view of the top right corner to show where gaps between and splits within laminations opened and closed because of sliding. Video captured movements of laminations as the contact point traveled along the top edge and accompanying audible pops. Sliding produced noticeably louder creaking than the crackles recorded during rolling configuration tests. Figure 17 photographed splits and scrapes of the top panel edge immediately following the unidirectional tests charted by Figure 11. Sliding exacerbated splits in the longitudinal laminations and at only a third of the overburden load that caused lesser damage in rolling. Figure 11 plots (c) through (f) captured some effects of the stick-slip frictional behavior in noise of loading segments. Stick-slip friction posed little consequence to overall stiffness of the CLT panel but caused noticeably higher damage to panel edges. Reverberating booms, furthermore, dynamically impacted and eventually dislodged pipe bushings embedded in the CLT in later tests at higher overburden.

The local damage observed in both rolling and slip-friction configurations could be mitigated by adding cross-laminations to the faces of the 5-ply panel. Outer cross layers may serve as both protective cover for the longitudinal plies and transverse reinforcement counteracting cracks induced by traction or sliding. Though the panel from the *No-Slip Traction Rolling* test was reused for slip-friction rocking tests, a new panel tested in *Slip-Friction Rocking* would likely sustain similar damage. Design modifications therefore should address concentrations of force in the timber near the pinned connections.

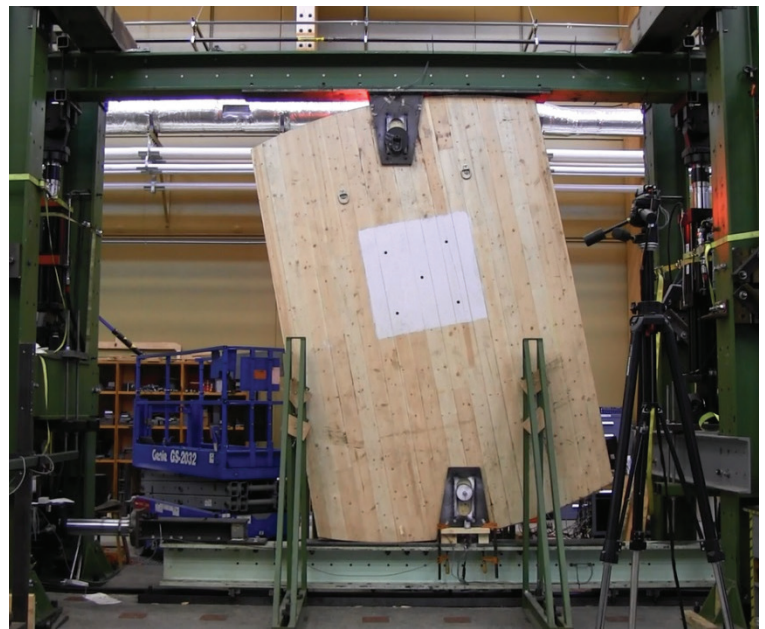


Figure 15: Slip-Friction of $e = 0.73$ Panel at Max Lateral Displacement and Overburden of 133 kN (30 kips)

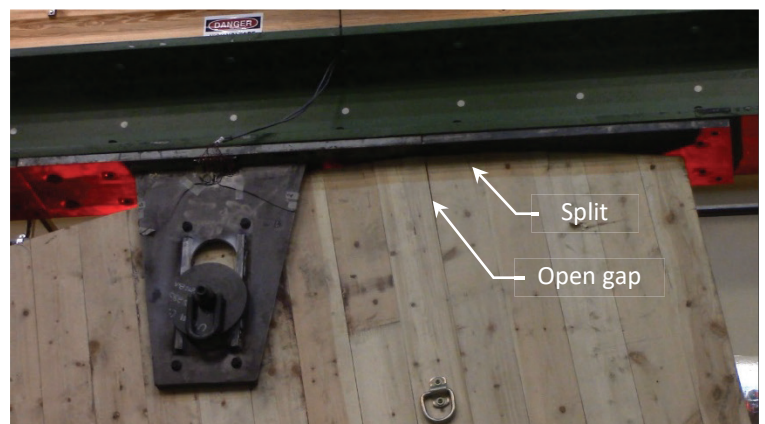


Figure 16: Splits and Gaps Open during Slip-Friction Sliding at Top Contact Bearing Zone.

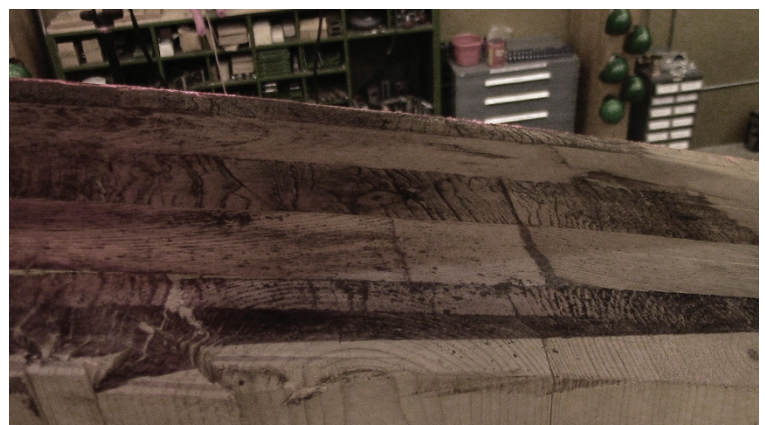


Figure 17: Local Damage of Top Edge of Panel after Unidirectional Slip-Friction Tests

continued on next page

Conclusions

- As a rigid body, the CLT prototype panel generally behaved predictably, according to principles of mechanics.
- Elliptical eccentricity provided the key feature for passive self-centering within a platform construction scheme.
- Connection constraints dictated the modes of horizontal shear transfer and rocking.
- Curvilinear pin slots shaped to act only as *displacement restraints* imposed minimal constraint, which enabled the elliptical panel to rock as a *rolling pendulum*.
- Vertical slots imposed horizontal connection constraint, which forced the panel into a *slip-friction* mechanism.
- Idealized hysteresis models enveloped the observed behaviors of the physical prototype with reasonable accuracy through lateral displacements ranging between:
 - ◇ ± 432 mm (17.0 in.) in *No-Slip Traction Rolling*, and
 - ◇ ± 356 mm (14.0 in.) in *Slip-Friction Rocking*.
- Idealized rigid-body models underestimated both damping and stiffness at extreme lateral displacements.
 - ◇ Simplifications in the *slip-friction rocking* model, which assume an aggregate and constant sliding friction force throughout the motion, do not keep pace with the rapid stiffness and damping increases observed at larger lateral displacements.
- *No-Slip Traction Rolling*, compared to *Slip-Friction Rocking*, demonstrated:
 - ◇ More than 3 times greater capacity for carrying superstructure weight on the same panel specimen, before damage to critical timber laminations begins.

- ◇ 17% greater lateral displacement capacity for the same panel rotation limit, and
- ◇ 2 times lesser lateral stiffness for the same overburden.

- Rolling produced less damage than sliding at the timber contact zones along panel edges.

Key takeaways from this first iteration of analytical and physical prototyping suggest that *rolling* is more resilient than the *slip-friction* pendulum. Frictional energy dissipation increased wear of CLT panel, lateral stiffness of the system, and concentrated forces at pin connections. Resilience of the *slip-friction* configuration, however, can likely improve with simple strategies. Adding cross-plyies to the outer wall faces, for example, would confine and reinforce the axial-load-carrying laminations. Shoes fit to the edges of the CLT panel could alternatively produce smoother sliding and distribute bearing contact forces over a greater area of timber. Refining details of the slip-friction panel mechanism, furthermore, would enhance versatility of the elliptical inverted pendulum concept. While the *rolling* configuration provides better seismic isolation properties in the direction of the wall plane, the slip-friction configuration offers several benefits other than better energy dissipation, such as:

- Universal connections that are interchangeable with a variety of elliptical profiles,
- Reduced risk of residual lateral displacements, and
- Higher thresholds to rocking, which improves everyday resistance to wind.

To better characterize the *slip-friction rocking* mechanism, an analytical model based on mechanical work equations is being developed. Multiple sources of friction, including frictional bearing contact of the pin connections, likely contribute to the nonlinear effects most pronounced by the hysteresis data at extreme lateral displacements. In conclusion, both *rolling* and *slip-friction* pendulum configurations merit further refinements because each showed

potential to improve the resiliency, versatility, and economic competitiveness of mass timber platform construction in regions of high seismicity.

Acknowledgement

The U.S. Forest Service in cooperation with the University of Wisconsin-Milwaukee funded this research through the Wood Innovations Program (Grant Number: 2016-DG-11420004-170). The U.S.D.A. Forest Products Laboratory-Engineering Mechanics and Remote Sensing Laboratory (Madison, Wisconsin) performed full-scale testing. The findings and opinions of this article represent views of the authors, which do not necessarily represent those of the sponsors.

References

- Amini, M.O., J.W. van de Lindt, S. Pei, D.R. Rammer, P. Line and M. Popovski (2014). Overview of a project to quantify seismic performance factors for cross laminated timber structures in the United States. In: Materials and joints in timber structures. S. Aicher, H.-W. Reinhardt and H. Garrecht, Springer, Stuttgart, Germany: 531-541.
- ANSI/ APA PRG 320 (2018). Standard for Performance-Rated Cross-Laminated Timber. Tacoma, Washington, APA – The Engineered Wood Association: 1-40.
- Applied Building Technology Council (2009). Quantification of Building Seismic Performance Factors, FEMA P695. Washington D.C., Federal Emergency Management Agency (FEMA).
- ASCE/SEI 7 (2017). Chapter 12: Seismic Design Requirements for Building Structures. In: Minimum Design Loads and Associated Criteria for Buildings and Other Structures (ASCE/SEI 7-16), American Society of Civil Engineers, Reston, VA, USA: 89 - 119.
- ASCE/SEI 7 (2017). Chapter 17: Seismic Design Requirements for Seismically Isolated Structures. In: Minimum Design Loads for Buildings and Other Structures (ASCE/SEI 7-16), American Society of Civil Engineers, Reston, VA, USA: 168-179.
- Ceccotti, A., C. Sandhaas, M. Okabe, M. Yasumura, C. Minowa and N. Kawai (2013). SOFIE project – 3D shaking table test on a seven-storey full-scale cross-laminated timber building. In: Earthquake Engineering & Structural Dynamics. 42: 2003-2021.
- Dujic, B. and R. Zarnic (2006). Study of lateral resistance of massive X-lam wooden wall system subjected to horizontal loads. International Workshop on Earthquake Engineering on Timber Structures. Coimbra, Portugal, COST E29- European Cooperation in the Field of Scientific and Technical Research: 97-104.
- Dunbar, A.J.M., S. Pampanin and A.H. Buchanan (2014). Seismic performance of core-walls for multi-storey timber buildings.
- Fast, P. and R. Jackson (2017). Brock Commons: A Case Study In Tall Timber. STRUCTURE magazine. Reedsburg, Wisconsin, C3 Ink. 24: 50 - 52.
- Ganey, R., J. Berman, T. Akbas, S. Loftus, J. Daniel Dolan, R. Sause, J. Ricles, S. Pei, J.W. van de Lindt and H.-E. Blomgren (2017). Experimental Investigation of Self-Centering Cross-Laminated Timber Walls. In: Journal of Structural Engineering. 143.
- Housner, G.W. (1963). The behavior of inverted pendulum structures during earthquakes. In: Bulletin of the seismological society of America. 53: 403-417.
- Jangid, R.S. and Y.B. Londhe (1998). Effectiveness of Elliptical Rolling Rods for Base Isolation. In: Journal of Structural Engineering. 124: 469-472.

Londhe, Y.B. and R.S. Jangid (1999). Dynamic response of structures supported on elliptical rolling rods. In: Doboku Gakkai Ronbunshu. 612: 11-20.

Loo, W.Y., C. Kun, P. Quenneville and N. Chouw (2014). Experimental testing of a rocking timber shear wall with slip-friction connectors. In: Earthquake Engineering & Structural Dynamics. 43: 1621-1639.

Mazzoleni, M.J., M.B. Krone and B.P. Mann (2015). Dynamics of Rocking Semicircular, Parabolic, and Semi-Elliptical Disks: Equilibria, Stability, and Natural Frequencies. In: Journal of Vibration and Acoustics. 137: 041017-041017.

Pei, S., J. Berman, K. Ryan, J. Ricles, R. Sause, J.D. Dolan and J.W. van de Lindt (2018). Natural Hazards Engineering Research infrastructure (NHERI) Tall Wood Project. Golden, Colorado, Colorado School of Mines. 2018.

Pei, S., J. Berman, J.D. Dolan, J.W. van de Lindt, J. Ricles, R. Sause, H.-E. Blomgren, M. Popovski and D.R. Rammer (2014). Progress on the development of seismic resilient Tall CLT Buildings in the Pacific Northwest. Proceedings of the 2014 World Conference on Timber Engineering, Quebec City, Canada.

Pei, S., J.W. van de Lindt, M. Popovski, J.W. Berman, J.D. Dolan, J. Ricles, R. Sause, H.-E. Blomgren and D.R. Rammer (2016). Cross-Laminated Timber for Seismic Regions: Progress and Challenges for Research and Implementation. In: Journal of Structural Engineering. 142: E2514001.

Pei, S., J.W. van de Lindt, A. Barbosa, J. Berman, E. McDonnell, J.D. Dolan, R.B. Zimmerman, R. Sause, J. Ricles and K. Ryan (2018). Full-Scale Shake Table Test Of Mass-Timber Building With Resilient Post-Tensioned Rocking Walls. World Conference on Timber Engineering. Seoul, Republic of Korea.

TRADA (2009). Stadthaus, Murray Grove.

Zimmerman, R.B. and E. McDonnell (2017). Framework - A tall re-centering mass timber building in the United States. Joint 2017 New Zealand Society for Earthquake Engineering Annual Conference and the Anti-Seismic Systems International Society's 15th World Conference. Wellington, New Zealand.

Zimmerman, R.B. and E. McDonnell (2018). Framework - Innovation in Re-Centering Mass Timber Wall Buildings. 11th National Conference in Earthquake Engineering. Los Angeles, CA.

Marco Lo Ricco, P.E., S.E. - Ph.D. Candidate, Department of Civil and Environmental Engineering, University of Wisconsin-Milwaukee, P.O. Box 784, Milwaukee, WI 53201, mloricco@uwm.edu

Dr. Al Ghorbanpoor, P.E. - Professor, University of Wisconsin-Milwaukee, P.O. Box 784, Milwaukee, WI 53201, algh@uwm.edu

Dr. Shiling Pei, P.E. - Associate Professor, Department of Civil and Environmental Engineering, Colorado School of Mines, 1500 Illinois St., Golden, CO 80401, spei@mines.edu

Doug Rammer, P.E. - Research General Engineer, USDA/US Forest Service Forest Products Laboratory, One Gifford Pinchot Drive, Madison, WI 53726, drammer@fs.fed.us

Marshall Begel, P.E. - General Engineer, USDA/US Forest Service Forest Products Laboratory, One Gifford Pinchot Drive, Madison, WI 53726, mbegel@fs.fed.us

James Bridwell, M.S. - General Engineer, USDA/US Forest Service Forest Products Laboratory, One Gifford Pinchot Drive, Madison, WI 53726, jbridwell@fs.fed.us

Reid B. Zimmerman, P.E., S.E. - Associate, KPFF Portland, 111 SW Fifth Avenue, Suite 2500, Portland, OR 97204, Reid.Zimmerman@kpff.com

