World Conference on Timber Engineering Oslo 2023

DEVELOPMENT OF A NUMERICAL MODEL TO CONSIDER THE FOUNDATION FLEXIBILITY EFFECTS IN CLT ROCKING WALLS

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ABSTRACT: This work developed a numerical model to consider the effects of foundation flexibility in cross laminated timber rocking wall structures. The model was validated against the test results of a quasi-static cyclic and dynamic tests. In both tests, the foundation behaved as a flexible member with significant yielding. A multi-spring model was used in the study and two different approaches for solid and steel section beam foundations were employed to calculate the axial stiffness of contact springs. The proposed techniques were found to be not only able to consider the foundation flexibility but to also omit the need to use empirical relationships or calibration with experiments to calculate plastic hinge length to be used for calculation of axial stiffness for the base springs. Following the validation of the numerical model, a parametric study was performed to study the effects of foundation flexibility on the shear-drift response of rocking wall structures.

KEYWORDS: Cross laminated timber, Rocking wall, Wall-foundation interaction, Numerical modeling

1 INTRODUCTION

Recently, there has been an increasing focus on the use of sustainable materials in construction. A global effort to reduce carbon emissions has instigated renewed interest in the use of engineered timber like cross laminated timber (CLT) and mass plywood for building structures, which promise a green construction material with higher strength-to-weight ratio, comparable stiffness and ductility, faster installation, and improved thermal performance. CLT rocking wall structures have been in a development phase as a resilient and sustainable alternative to conventional concrete and steel structures. Rocking structures promise damage avoidance design with minimal residual drift and decent energy dissipation.

The U.S. National Science Foundation funded NHERI Tall wood project is a major research initiative [1]. As an initial step, a series of CLT rocking walls with varying design parameters like area and initial force of post-tensioning, single and coupled walls, and varying flexibility of foundation were used [2]. A uniaxial shake table test of a 2-story CLT rocking wall structure followed [3], and a 10-story rocking wall structure is being tested on the NHERI shake table at the University of California-San Diego [4]. During the testing of the 2-story structure, it was observed that the base beam, used as a foundation for the rocking wall, yielded with significant permanent deformation. This unintended inelastic behavior of foundation increased the fundamental period of the building [5], resulting in lower-than-expected damage in the structural members especially at the wall toes; however, the displacement demand on members increased. Based on these results, it is important to explore the effect of foundation flexibility on the overall lateral behavior of rocking wall structures and develop guidelines to achieve optimum solutions with minimal damage and manageable drift demands. To achieve this, numerical models capable of incorporating the effects of foundation flexibility are required, which can then be used to further investigate different aspects of this rocking wall-foundation interaction.

This study is focused on the development of a numerical model to account for the foundation flexibility effects in rocking wall structures. Two different modeling approaches are proposed in this study to model two different tests from literature where the foundation of rocking wall behaved nonlinearly with significant yielding/crushing, one was a static [2] while the other was a dynamic test [3]. A multi-spring model was used in this study to model the rocking behavior along with two different approaches to model contact stiffness at the interface of rocking wall and flexible foundation. A parametric study was then performed to understand the effects of foundation flexibility on the seismic force and displacement demands against two levels of initial post-tensioning.

2 CASE STUDY STRUCTURES

Ganey [2] tested single and coupled CLT rocking walls using quasi-static cyclic lateral loading and the response of these rocking walls were numerically modelled in several studies [2,6,7]. However, to the best of the authors' knowledge, Specimen 4 from the testing has been

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https://doi.org/10.52202/069179-0345

modeled in the past studies. Specimen 4 consisted of a single CLT rocking wall resting on a CLT foundation. Since the rocking wall rested on the weaker axis of the CLT foundation, the nonlinear behavior was confined to crushing of the foundation only, resulting in a flexible foundation condition. The CLT rocking wall consisted of two Hem-Fir 5-ply CLT panels joined by steel plates using SDS screws. Cross-sectional dimensions were approximately 1.22 m by 0.17 m while the wall height was 4.43 m. Lateral loading was applied at a height of 4.1m. A 31.5 mm rod with an initial post-tensioning force of $0.4f_{pt,u}$ was used, where $f_{pt,u}$ was the ultimate stress of PT rod. The material properties of CLT and PT are shown in Table 1 while the testing setup for Specimen 4 is shown in Figure 1.

Table 1: *Properties of CLT and PT used for Specimen 4*

CLT Property (Units)	Value	PT Property (Units)	Value
E_1 (MPa)	3042	E_{pt} (GPa)	220
E_2 (MPa)	900	$f_{pt,y}$ (MPa)	929
E_3 (MPa)	2080	$f_{pt,u}$ (MPa)	1091
G_1, G_2, G_3 (MPa)	345	$F_{pt,i}$ (kN)	338
ν_1, ν_2, ν_3	0.3		
f_y (Direction 1*) (MPa) (For rocking wall)	24.8		
f_y (Direction 2*) (MPa) (For CLT foundation)	5.93		

*Directions 1, 2 and 3 coincide with the height, length, and thickness of the CLT wall panel

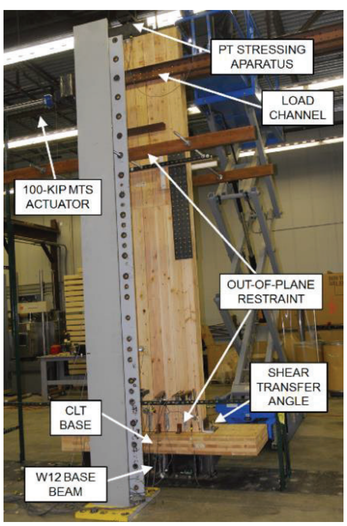
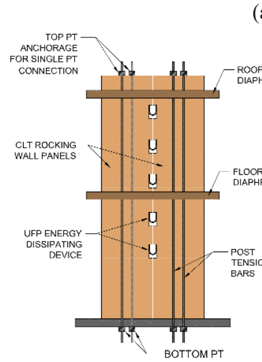
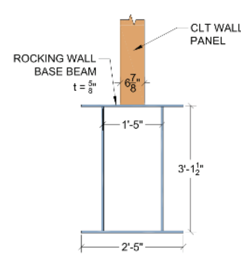


Figure 1: *Testing setup for Specimen 4 (CLT wall) resting on CLT foundation* [2]

The dynamic test considered was the full-scale 2-story CLT rocking wall structure with two rocking shear walls in the direction of shaking [3]. The rocking wall panels were 5-ply having grade E2-M1 CLT. The modulus of elasticity, shear modulus and yielding stress for CLT were 8536 MPa, 552 MPa and 25 MPa, respectively. For each wall, two balloon framed CLT panels were used. The CLT panels in each wall were joined by five equally spaced U-shaped flexural plates along the height of the wall to provide a minimum energy dissipation ratio of 0.3. Modulus of elasticity and yielding stress of each U-shaped plate were 2×10^5 MPa and 414 MPa. Post-tensioning was provided by using four 19 mm bars in each wall with an average initial post-tensioning force equal to 40% of yield strength (53 KN for each bar). Each wall had a height, length, and thickness of 7.32 m, 1.52 m, and 0.17 m, respectively. Shear transfer angles were used at the wall ends to transfer shear and avoid lateral slip. Wall-to-diaphragm shear transfer connections were used to transfer lateral demands while out-of-plane movement was constrained using bracing. As explained earlier, the steel foundation of the 2-story building, which was expected to behave as rigid, exhibited nonlinear behavior with permanent deformations right below the wall ends (i.e., at the center of rotation during the rocking motion.) A total of 14 excitations were used for the dynamic testing of the structure, with effective PGA for the excitations varying from 0.13g to 0.85g representing service level earthquake (SLE), design basis earthquake (DBE) and maximum considered earthquake (MCE). Details of the test structure, rocking wall and foundation are shown in Figures 2 and 3. Further information about the structure, testing setup and dynamic testing can be found in [8].

(a)





(b) (c)

Figure 2: *(a) Full-scale 2-story structure (b) CLT rocking wall (c) Steel base beam [8]*

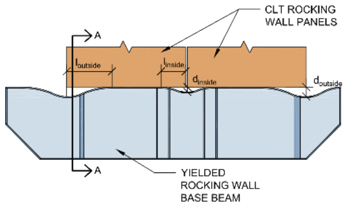


Figure 3: *Full-scale 2-story structure Steel base beam after yielding* [8]

3 PROPOSED NUMERICAL MODELS

Several techniques have been used to model the behavior of rocking walls including monolithic beam analogy, lumped plasticity model, multi-spring model, fiber model, and finite element model. Except for finite element models, all the above-mentioned models need iterative procedures or calibration using experimental results to determine the compression zone/plastic hinge length, and do not consider foundation flexibility. In the current study, a multi-spring (MS) model has been used, as shown in Figure 4, for the 2-story coupled rocking wall with energy dissipation mechanisms, that was tested on the shake table. The MS model for the quasi-static test on single rocking wall without any external energy dissipation mechanism was also based on the same principles for the modeling of the wall, PT steel, contact modeling, placement of rigid links, etc., but it is not shown here for brevity. The rocking wall was modeled using a beam-column element, the post-tensioning was modeled using axial springs with prestress force, and the contact was modeled using 50 compression-only springs at the base. Adjusting the stiffness of the contact springs is an active area of research. Several empirical formulas have been proposed in the past to determine the length of the plastic hinge (L_e), which is used to find the contact stiffness by equating it to EA/L_e where, E and A are modulus of elasticity of wall and contact area. [9,10] proposed empirical relationships to find out the plastic hinge length for rocking timber walls. A plastic hinge length of two times the wall thickness has also been proposed and used in the past [5,11] while for concrete rocking connections, a plastic hinge length equal to the half the length of the walls has been proposed [12,13]. All these empirical relationships try to a provide a measure for the axial stiffness of the rocking wall assuming the foundation to act as rigid, however, both the case study structures selected in the current work showed a permanent deformation due to yielding in the foundation, rendering the available empirical formulas unsuitable. Therefore, different techniques were proposed in this study.

The stiffness of the contact springs was calculated using principles of contact mechanics. [14] argued that the volume with dimension of the contact length in all three spatial dimensions represents the maximum stress concentration region and should be used to find different parameters of contact. It has been shown that although it's an assumption, it gives results within a 10% margin of error [14]. If the indentation at contact interface of a length $2a$ against a force F is d, then axial stress and axial strain are $\frac{F}{(2a)^2}$ and $\frac{d}{2a}$, respectively. Solving for the equation $\sigma = E\varepsilon$ results in

$$F = 2aEd \tag{1}$$

where, $2aE$ represents contact stiffness and E is the modulus of elasticity of elastic half space. However, this formulation is valid for solid foundations only and a different technique is used to model the 2-story rocking wall test on shake table.

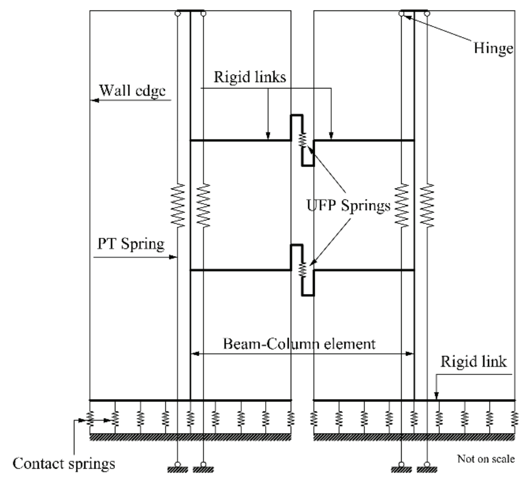


Figure 4: *Multi-spring model*

A steel beam was used as foundation in the dynamic test which showed yielding and permanent deformation. To quantify the contact stiffness at wall-foundation interface, a finite element model for the steel base beam resting on the ground was developed in ABAQUS [15]. Solid 8-noded linear brick elements (C3D8R) with reduced integration were used to model the base beam. A mesh sensitivity study was conducted before choosing the mesh size. Self-weight of the base beam was ignored. Yielding stress for the steel beam was assumed to be 414 MPa. To simulate contact behavior during rocking motion, a triangular force pattern was applied at the expected contact area on the steel beam. The problem was considered as a two-dimensional and the axial stiffness for the contact springs was determined by dividing the force in the tributary area of each spring divided by the average deformation in that region. The finite element model, and the stress distribution in the steel base beam after applying the loading is shown in Figure 5. It is important to mention that the finite element model showed yielding in the base beam even for the loading corresponding to the service level earthquakes. Using the MS model described earlier with contact stiffness from the results of finite element analysis, all 14 ground motions were applied to the 2-story rocking wall in the sequence used during the experiments and the yielding observed in any test in the contact springs was used as initial condition for those springs in the next

 https://doi.org/10.52202/069179-0345

tests. However, it was observed that assuming an undamaged base beam for each test did not alter the analysis results significantly as the beam started yielding at the corners at the start of the test, and the condition of the beam against the maximum response time span was almost same.

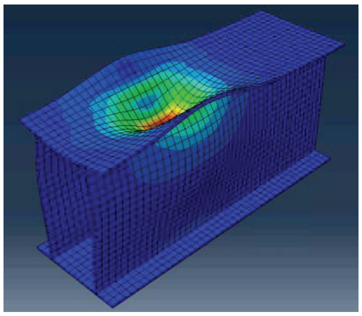


Figure 5: *3D Finite element model of base beam after applying loading*

4 NUMERICAL MODEL VALIDATION

Loss in PT force against increasing drift demand was observed in the test for Specimen 4; however, the model used here cannot account for it. Therefore, a maximum drift of 4% is considered for the comparison purpose as the losses in PT steel during the test were within reasonable range up to this drift. Equation 1 was used to model the stiffness of the foundation which is considered to dominate the contact behavior at the wall-foundation interface because of a much lower modulus of elasticity of CLT in transverse direction of the foundation when compared to the in-plane direction in wall. This stiffness is, therefore, used as axial stiffness for contact springs. The experimental results for only one 5-ply CLT panel was available in the out of plane direction; therefore, an average value of yield stress for four 3-ply and one 5-ply CLT panels in the direction of interest were used to adjust the yielding stress of the CLT foundation.

Wall Specimen 4 was modeled using the MS model explained in earlier sections. Initially, a single layer of axial springs representing a flexible foundation was used. However, two layers of springs can also be used representing the axial stiffness of wall and transverse stiffness of foundation. An important point in this regard is the use of a plastic hinge length for the springs representing the rocking wall. Equating Equation (1) with the expression EA/L_e gives an effective length of the wall equal to the thickness of wall. It's important to mention here that several past studies have proposed an effective length of two times the wall thickness for CLT walls [7, 11, 16]. Results with one and two layers exhibited almost identical results in this study and are not shown here for brevity.

Comparison of force-drift behavior from the experimental results and numerical model is shown in Figure 6. The CLT rocking wall showed slightly different behavior in positive and negative direction of motion. As the first cycle of lateral deformation was provided in positive direction, the comparison in the positive direction of response is considered and discussed here. The numerical model was found to model the initial stiffness accurately. Since the yielding stress for the 5-ply CLT in the transverse direction was taken as average of different samples, a monotonic response considering the CLT wall to be linear elastic is shown. The nonlinear behavior in monotonic response is purely due to geometric nonlinearity. Comparing these two responses shows that the CLT wall showed yielding before gap opening behavior which also caused a softening of post gap-opening stiffness. The numerical model was able to predict this response accurately and the assumed average yield stress value was found to be reasonable. The hysteretic behavior, on the other hand, showed some mismatch with the numerical model showing lower energy dissipation than the experimental results. It's important to mention here that this is due to unavailability of a reliable hysteretic model for the transverse direction response of CLT and future works need to focus on this. Single and coupled rocking walls tested by Ganey were also modeled using this approach and results showed a similar or better accuracy compared to some of the earlier works that calculated plastic hinge length by using trial-and-error approach; however, these walls had rigid-like footing condition and their results are not presented here for brevity and for the reason that this work is focused on the response of rocking structures on flexible foundations.

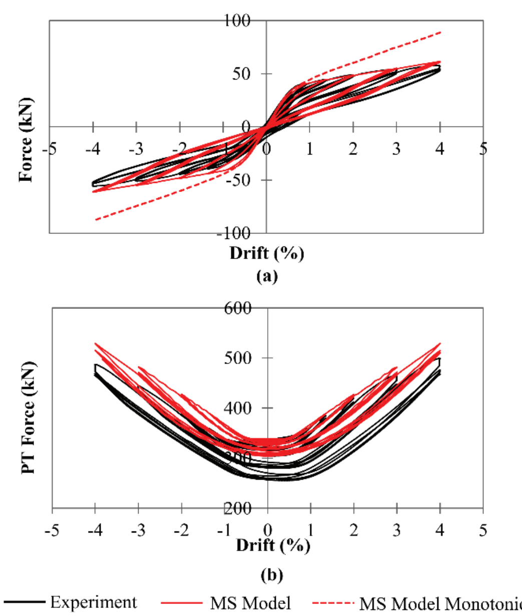


Figure 6: *Comparison of (a) force-drift and (b) PT force-drift response from experiment and MS model for Specimen 4*

As explained earlier, the stiffness of the base springs for the dynamic test was calibrated using a finite element model. All the other aspects of the MS model were same except for the calculation of contact stiffness as described earlier. Results from two SLE (Tests 2 and 4), one DBE (Test 6) and one MCE (Test 12) level ground motions are shown in Figure 7. The MS model was able to predict the displacement demands in terms of pattern and maximum values with significant accuracy. Barring some under-prediction in a few cycles, the numerical model was found to be efficient in simulating the displacement results of rocking wall resting on flexible foundation. Although not shown here, the results of roof acceleration and base shear, however, showed underprediction for most tests. Two different techniques were used in the current study to model the foundation flexibility effects in solid and steel section beam foundations, and the proposed techniques have shown promising results. Further verification of the proposed techniques is needed in the future to develop a more generalized approach for modeling contact behavior in rocking walls with or without rigid foundations, omitting the need to calibrate the numerical models with experimental results.

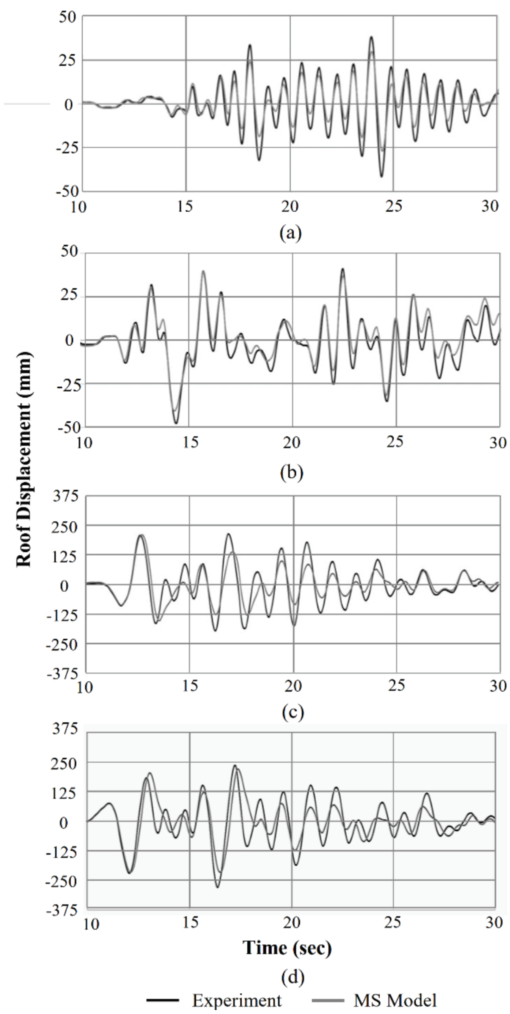


Fig 7: *Comparison of roof displacements from experiment and MS model for (a) Test 2, (b) Test 4, (c) Test 6 and (d) Test 12*

As the proposed MS model can include the effects related to foundation flexibility, a parametric study was conducted to further explore these effects on the base shear demands of rocking walls against different levels of foundation flexibility.

5 PARAMETRIC STUDY

To further explore the effects of foundation flexibility on the lateral behavior of rocking walls, the validated numerical model was used to simulate different combinations of wall and foundation stiffness. Ganey used two levels of initial post-tensioning for different specimens (i.e., 10% and 40% of ultimate strength of PT steel.) In this part, the Ganey wall was used with different foundation stiffness values, ranging from rigid to a stiffness value equal to 10% of the axial stiffness of the wall against two levels of initial post-tensioning, as shown in Table 2. Axial springs representing stiffness of wall and foundation were considered to be acting in series to calculate equivalent interacting stiffness. All the other parameters were kept constant for all the iterations. In this part, both the wall and the foundation were assumed to be linear elastic with geometric nonlinearity acting as sole contributor for inelastic behavior. As the foundation flexibility increased,

the decompression rotation increased while the forces against any particular drift decreased, as shown in Figure 8. When the foundation stiffness was one tenth of the axial stiffness of the wall, the maximum force demands against 3% drift reduced by 30% and 27% for the cases with initial PT force equal to 10% and 40% of the ultimate PT force, respectively. There was also a visible decrement in initial stiffness of the rocking system which will change the modal properties of the system with a lengthened fundamental period, modified mass participation and changes in modal force patterns along the height.

Table 2: *Variables used in parametric investigation*

Initial PT	Parameter studied	Parameter variations				
$f_{pt,i} = 0.1f_{pt,}$	Ratio of foundation to wall stiffness	Rigid	1	0.5	0.25	0.1
$f_{pt,i} = 0.4f_{pt,}$	Ratio of foundation to wall stiffness	Rigid	1	0.5	0.25	0.1

Although the modeling techniques proposed in the current work were focused on the timber rocking walls, future works need to ascertain their general applicability by using them for the modeling of concrete and steel rocking structures. Furthermore, the proposed contact mechanics technique for solid foundations suggests that a concrete rocking wall resting on a concrete foundation with a similar value of modulus of elasticity, as is usually the case, should not be considered rigid as the real contact stiffness value would be lower than the axial stiffness of the wall in the contact region.

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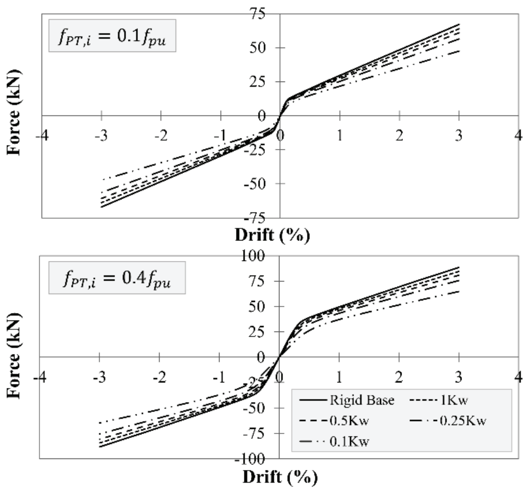

Figure 8: *Effect of foundation stiffness on the force-drift response.*

6 CONCLUSIONS

This study was focused on the development of a simple numerical model to account for the foundation flexibility effects in rocking wall structures. Results from a static and a dynamic test of rocking walls with flexible foundations were used in the current study. Two different approaches for the modeling of contact behavior at wall-flexible foundation interface were proposed. For the solid flexible foundation, a technique based on the principles of contact mechanics was used. Albeit some mismatch in the hysteretic response, the proposed technique was found to predict the force and displacement responses of the case study rocking wall resting on a yielding foundation with significant accuracy. On the other hand, using the results of foundation stiffness from a finite element model to adjust the contact stiffness in the multi-spring model proved to be an effective technique, and accurately predicted the displacement responses of the structure against varying levels of seismic intensity. A decrease in foundation stiffness is found to reduce the initial stiffness and the force demand in rocking structures. Future works are needed to check the general applicability of proposed techniques by modeling the concrete and steel rocking walls.

ACKNOWLEDGEMENT

Authors would like to acknowledge the Fulbright Commission, USA for provide the Visiting Scholar Fellowship to the first author.

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