WCTE
WORLD CONFERENCE ON TIMBER ENGINEERING
2010

TWO-DIMENSIONAL NUMERICAL FRAMEWORK FOR THE NONLINEAR STATIC AND DYNAMIC ANALYSIS OF LIGHT-FRAME WOOD STRUCTURES

Ioannis P. Christovasilis[1] and Andre Filiatrault[2]

ABSTRACT: This paper presents a summary of the development and validation of a new two-dimensional (2D) numerical framework, suitable for nonlinear, inelastic, static and dynamic analysis of 2D vertical light-frame wood building slices that incorporate sheathed woodframe shear walls as a lateral-load-resisting system. The 2D building model is based on a sub-structuring approach that considers each floor diaphragm as a rigid body with three kinematic degrees-of-freedom. A sub-structure model is developed for each individual single-story wall assembly that interacts with the adjacent diaphragms, above and below, and generates the resisting in-plane internal forces. The 2D shear wall model takes explicit consideration of all sheathing-to-framing connections and offers the capability to optionally simulate deformations in the framing members and contact/separation phenomena between framing members and diaphragms, as well as any anchoring equipment (i.e. anchor bolts, holdown devices) typically installed in light-frame shear walls to develop a vertical load path that resists overturning moments. Corotational descriptions are used to solve for displacement fields that satisfy the equilibrium equations in the deformed configuration, accounting for geometric nonlinearity (large rotations-small deformations) and P-Δ effects. To validate the proposed numerical framework, three simulation examples are presented, based on experimental results from single- and two-story full-scale shear wall specimens. These examples demonstrate the capability of the model to simulate accurate load paths in the shear wall assemblies and successfully predict variations in strength, stiffness and energy dissipation characteristics of the lateral-load-resisting system.

KEYWORDS: Wood structures, numerical modelling, nonlinear analysis

1 INTRODUCTION

The multi-component nature of light-frame wood structures and the versatility of the finite element method have led to the development of simplified – sub- or macro-modelling – and detailed – including sub-structuring – numerical approaches to simulate the response of the structural components that assemble a light-frame wood shear wall or a complete light-frame wood building. The level of detail incorporated in numerical models of structural components has been related to the physical size of the prototype considered and the type of analysis pursued, favouring numerically efficient sub-modelling techniques, especially for nonlinear cyclic analysis of two-dimensional (2D) or three-dimensional (3D) structures.

The majority [1, 3-8] of the proposed building numerical models over the last 10 years [1-8] utilize 1D or 2D-planar macro-models with a limited number of internal degrees-of-freedom (DOF) to simulate the in-plane hysteretic response of single-story light-frame shear walls. The mechanical properties of each shear wall macro-model are calibrated based on the lateral cyclic response of a detailed shear wall numerical model [3-8] or a shear wall test specimen with similar structural configuration and nailing schedule [1]. The developed shear wall numerical models vary from simplified formulations that consider rigid, pinned framing members and generalized panel displacements [3, 8], to more detailed approaches that assign beam and shell elements to simulate framing and sheathing components, respectively, retaining pinned framing connections [5, 6] or allowing deformability of the support of vertical framing members [4, 7]. Nevertheless, all the shear wall formulations utilize a sub-model of two independent orthogonal nonlinear single-degree-of-freedom (SDOF) springs to explicitly simulate each sheathing-to-framing connection based on the actual nailing pattern. The mechanical properties of the sheathing-to-framing connection sub-models are calibrated from cyclic connection test data of specimens with similar geometric and material configurations. Various approaches have been adopted for the consideration of horizontal wood diaphragms

[1] Ioannis P. Christovasilis, Department of Civil, Structural and Environmental Engineering, University at Buffalo, State University of New York, Buffalo, NY 14260, USA.
Email: ipc@buffalo.edu
[2] Andre Filiatrault, Department of Civil, Structural and Environmental Engineering, University at Buffalo, State University of New York, Buffalo, NY 14260, USA.
Email: af36@buffalo.edu

and the complete formulation of the numerical building model. The simplest model considers diaphragms as rigid plates with 3 DOF in the horizontal plane [3], while a recent study has included all 6 DOF related to the kinematics of a rigid diaphragm in 3D [8], introducing a shear-bending coupled response of the shear wall macro-models. The more detailed formulations utilize elastic shell elements for each horizontal diaphragm [4, 5 and 7] or allow in-plane deformability [1], while additional intercomponent connections are proposed in [5] to capture the interaction between vertical and horizontal diaphragms. Contrary to the approaches described above, the numerical 3D building model proposed in [2] for nonlinear static analysis is formulated at the nail level assigning beam and shell elements for framing and sheathing components, while each sheathing-to-framing connector is simulated by a mechanics-based sub-structure model that is derived from analogy to an elastoplastic pile in nonlinear foundation. Despite the favourable modelling features, the computational overhead needed to perform a nonlinear cyclic analysis of a complete building at the nail level, with an additional computationally expensive connection sub-structure model, rendered this numerical framework [2] computationally intensive and inefficient, thus, it has not been extended to include dynamics and simulate the seismic response of light-frame wood structures.

2 RESEARCH MOTIVATION

The review of the existing studies has shown that there is lack of detailed, yet computationally efficient, building numerical models at the nail level, even for reduced geometric and structural configurations, such as 2D models of a multi-story vertical slice of a light-frame wood building, as shown in Fig. 1a. This model includes only the structural components considered in the seismic design; typically plywood or Oriented Strand Board (OSB) attached to one side of the wall framing. Despite the numerical efficiency of simulating a shear wall assembly in a building analysis with a reduced DOF calibrated macro-model, the numerical accuracy of the predicted nonlinear response relies on the appropriate modelling of both the structural components and the boundary conditions of the respective wall, in the individual shear wall model. The boundary conditions can be considered to be associated with the motion and the deformations of the horizontal diaphragms above and below the shear wall assembly. Although recent studies have developed detailed finite element shear wall models [4, 5 and 7], the boundary conditions considered are not consistent with the boundary conditions assumed in the calibrated simplified sub-models. Obviously, the simplest hypothesis is to simulate the diaphragms as rigid bodies with 3 DOF in the wall plane, as shown in Fig. 1b. If the shear walls are assumed to deform in a pure shear mode of deformation, similarly to the simplified model described in [3], the rigid diaphragms shall translate horizontally but not rotate, however, if framing axial and bending flexibility is considered, the diaphragms shall translate in both directions and rotate within the wall plane. Since the motion of the diaphragms affects the framing deformations and, thus, the distribution of sheathing-to-framing resisting forces, light-frame wood shear walls act as structural elements with shear, axial and moment interaction, which implies that two identical wall segments within the same inter-story shear wall will not demonstrate identical responses, unless boundary conditions are identical. So, an effective simplified shear wall sub-model, to be used as equivalent of a detailed shear wall model that can accommodate uplifting response, should address the coupling interaction between the 6 boundary DOF and such formulations have not been established in the reviewed existing literature.

These observations have led to the development herein of a numerical framework for the analysis of light-frame wood buildings that considers rigid floor diaphragms as the primary components and addresses the resisting forces generated by the inter-story shear walls through the interaction with the floor diaphragms. The analysis has been limited to 2D vertical building models with single-sided structural panels, as the first fundamental step towards the detailed nonlinear analysis of a complete light-frame wood building.

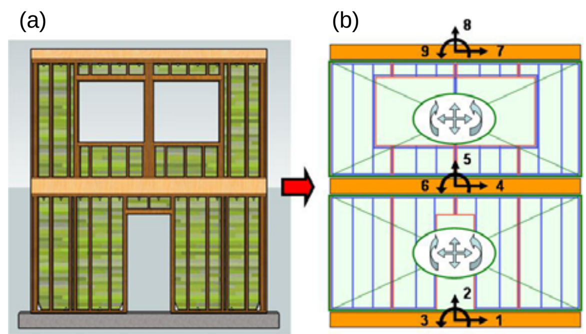

Figure 1: *(a) Typical Exterior Wall of a Two-Story Light-Frame Wood Building; and (b) Illustration of Master DOF of a Building Model and Shear Wall Sub-Structures for the Simulation of Each Story.*

3 NUMERICAL FRAMEWORK

The numerical framework presented in this paper formulates a 2D building model based on a detailed modelling approach of the entire inter-story shear wall assembly of each discrete floor. When considering the global analysis of a complete building, while a great number of DOF is required for the accurate calculation of resisting forces, the DOF needed for the calculation of the inertial forces in the structure can be significantly less. It is, thus, favourable (i) to consider a numerical building model with reduced DOF, called master DOF, which can adequately represent the inertial forces in the global level; and (ii) to use a sub-structuring approach to condense out the numerous DOF of each detailed shear wall model, maintaining only the associated master DOF. An efficient selection of master DOF is those associated with the motion and the deformation of the floor diaphragms. If floor diaphragms are considered to be rigid bodies, then three DOF in the 2D wall plane – two translations and one rotation – are sufficient to describe the equilibrium equations for each body, as shown in Fig. 1b. Utilizing the diaphragms as boundary elements of the sub-structures developed for each inter-story wall assembly allows the simulation of other modes of deformation (i.e. flexural and rocking modes) with due consideration of the interaction effects between shear walls and floor diaphragms.

Regarding the analysis of each shear wall sub-structure, sheathing panels are described with 4 DOF, similarly to [3], while sheathing-to-framing connections are described with two orthogonal independent phenomenological springs that exhibit pinching, strength deterioration and stiffness degradation. Each orthogonal pair of springs representing a single sheathing-to-framing connection is rotated according to the parallel and perpendicular directions of the initial trajectory computed under infinitesimal lateral wall deformation, as initially suggested in [10] and adopted in [7]. The proposed shear wall element enables the analyst to select between a simplified and a detailed formulation to describe the wood framing components. In the former case, referred later as **Pure Shear** formulation, framing is assumed rigid and pin-connected and is considered to be rigidly attached to the floor diaphragms. In the latter case, referred later as **Model** formulation, framing members are represented with linear elastic beam elements with axial and flexural behaviour using centre-line modelling of each individual framing component. Considering a wall segment with no openings, the framing configuration will consist of vertical continuous studs connected to the horizontal continuous sill and top plates and a detailed numerical model of the framing domain can be developed as shown in Fig. 2. The framing domain of the shear wall assembly is considered as three groups of components: the sill plate members, the top plate members and the internal framing members. These components are meshed with 2-noded beam elements assigning different nodes for each group at the interaction surface, which is actually the location of each plate-to-stud connection at the horizontal boundaries of the wall assembly. This requires the development and use of appropriate interface elements to simulate: (i) the interaction between horizontal and vertical framing members; (ii) the interaction between horizontal boundary plates and diaphragms; and (iii) the structural response of anchoring equipment (i.e. anchor bolts, holdowns). Contact elements are introduced along vertical DOF at framing-to-framing connections, as well as at the intersection of the sill plate with the ground. Horizontal DOF at framing-to-framing connections are rigidly constrained, while horizontal forces between horizontal plates and diaphragms are transferred through master nodes assigned at the centre of each independent sill or top plate. This enables modelling of the uplifting response without introducing unrealistically high uplift resistance due to catenary action resulting from the consideration of geometric nonlinearity. Anchoring devices are simulated by introducing nonlinear springs that connect corresponding vertical DOF of the framing with the diaphragms.

Finally, the use of corotational descriptions of the displacement fields of the finite elements implemented in the proposed numerical framework accounts for geometric nonlinearity associated with large rotations and for P-Δ effects due to gravity loads, assuming small deformations of the structural members that remain linear elastic, such as the individual framing members and the sheathing panels. This results in a shear wall element that satisfies equilibrium in the deformed configuration and is applicable for nonlinear analysis up to complete failure of the lateral-load-resisting system and side-sway collapse of the structure. Due to space limitations, the analytical derivations developed within this research study are not presented in this paper. This detailed analytical background can be found in [11, 12].

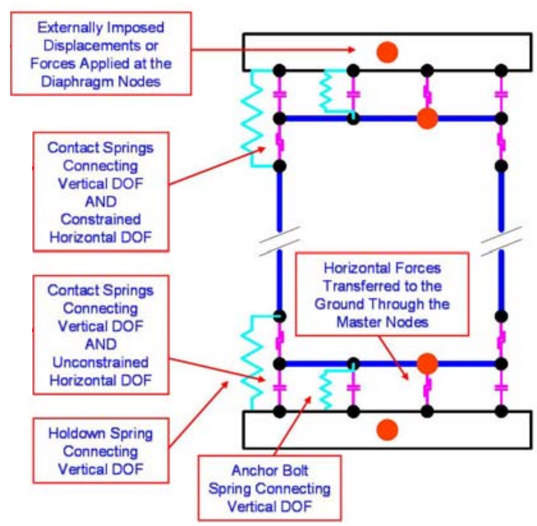

Figure 2: *Detailed Numerical Model of the Framing Domain.*

4 NUMERICAL PREDICTIONS

4.1 VALIDATION AGAINST PSEUDO-STATIC CYCLIC TESTS

This section presents the comparison of global responses – inter-story displacements versus inter-story forces – between experimental and numerical data, generated for one single-story and one two-story configuration of full-scale shear wall specimens. The selected experimental data have been documented in [13], as part of the CUREE-Caltech Woodframe Project.

4.1.1 Single-story specimen

The single-story configuration incorporated a Garage Door (GD) with one wall segment at each end with a relatively high aspect ratio (AR) of 2.5. The wall dimensions were equal to 4.9m long by 2.45m high, as shown in Fig. 3. The framing consisted of nominal 2x4

(38mm-by-89mm) studs spaced at 400mm on centre (o.c.), using Douglas-Fir lumber graded No. 1 or better. The sheathing provided was OSB, 9.5mm thick. Sheathing panels were fastened to the framing with 8d box gun nails, 63.5mm long with 2.9mm diameter. Edge nailing was specified at 75mm o.c. Specific anchorage equipment was installed at the end posts of each full-height shear wall segment, as shown in Fig. 3. More information on the geometric and structural characteristics of the shear wall specimens can be found in [13], while the properties and parameters utilized in the constitutive models are presented in [12]. Two identical specimens were tested under displacement-controlled cyclic loading using the CUREE protocol [14]. The total weight acting at the top of each single-story specimen was estimated at 4.5kN.

Figures 4a and 4b illustrate the cyclic pushover test results for two different test specimens (labels **Test A** and **Test B**) along with the numerical **Model** predictions, for the two wall configurations. In general, the predicted cyclic behaviour is well correlated with the experimental response exhibiting in-cycle and cyclic strength degradation and pinching characteristics, consistent with the test observations. Figures 4c and 4d illustrate the monotonic and cyclic pushover curves and the associated cumulative strain energy dissipation from the cyclic responses, including both numerical predictions. This summarizing figure shows that the **Pure Shear** response predicts not only higher stiffness and strength but also fatter hysteresis loops during pinching response, leading to significant overestimation of the energy dissipation capability. The energy dissipation capability predicted by the **Model** response is consistent with the energy dissipated by the **Test** response and the overall rate of dissipation is reasonably predicted throughout the deformation ranges.

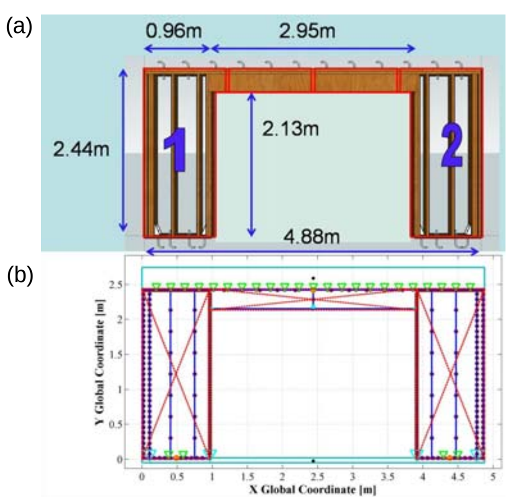

__Figure 3:__ (a) Geometric and Panel Configuration; and (b) Numerical Model of the GD Wall.

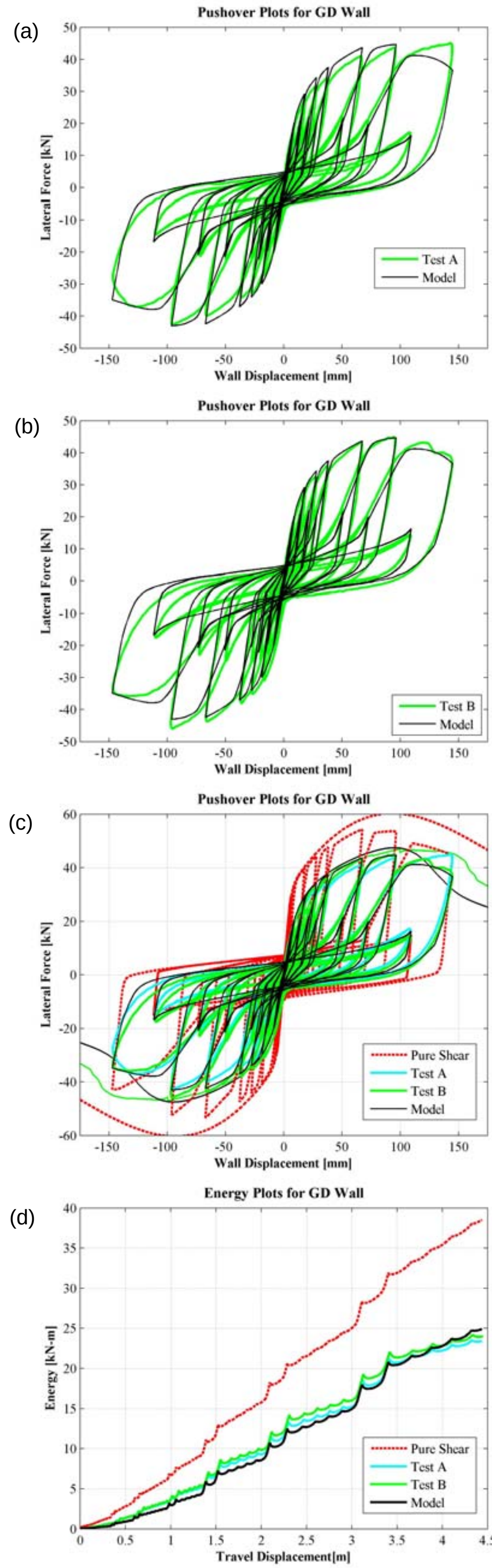

__Figure 4:__ (a, b, c) Comparison of Cyclic Response; and (d) Dissipated Energy for GD Wall.

4.1.2 Two-story specimen

The dimensions of the two-story specimen, denoted as FS2S wall, were equal to 4.9m long by 5.2m high, as shown in Fig. 5. Each story had a clear height of 2.45m, while the diaphragm between the two stories was 0.3m high. Both stories were fully sheathed with four OSB panels. The structural components were the same as the single-story specimens and edge nailing was specified at 150mm o.c. for both stories. The total weight acting at the top of the walls was estimated at 5.8kN and 4.5kN for the first and second story, respectively.

(a)

(b)

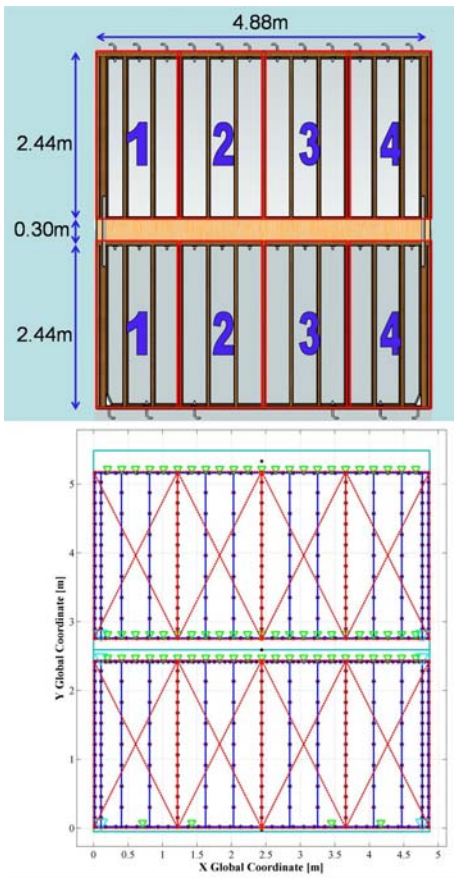

Figure 5: *(a) Geometric and Panel Configuration; and (b) Numerical Model of the FS2S Wall.*

Figure 6 illustrates experimental and predicted hysteretic force-displacement **Model** responses for each story. The **Model** predictions for both stories are in relatively good agreement with the **Test** responses. Figure 7 illustrates global experimental and numerical responses that include both **Model** and **Pure Shear** predictions, as well as the strain energy absorbed in each story. Interestingly, while the **Pure Shear** prediction for the first story is similar to the **Model** prediction and similarly correlated to the **Test** response, the **Pure Shear** predicted second-story forces are fairly higher than the **Model** and **Test** forces.

(a)

(b)

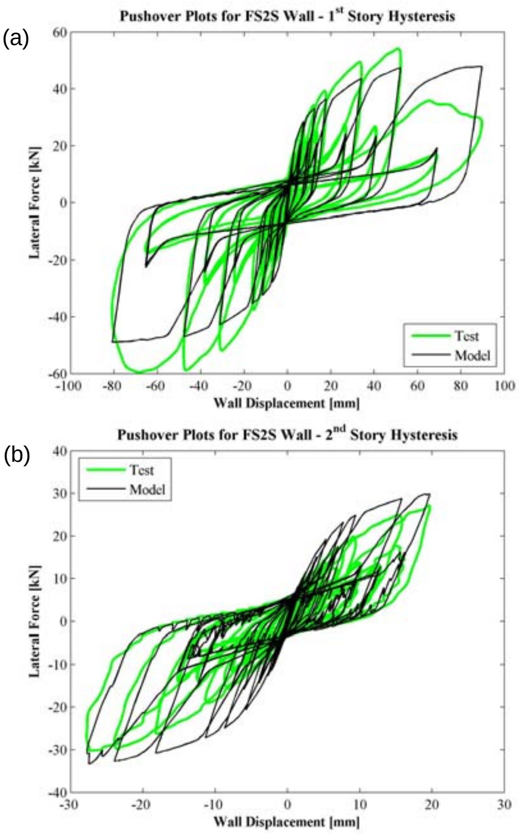

Figure 6: *Comparison of Cyclic Response for (a) the First Story; and (b) the Second Story of FS2S Wall.*

4.2 VALIDATION AGAINST DYNAMIC SHAKE-TABLE TESTS

This section presents the numerical predictions related to the experimental data from a shake-table testing program that has been documented in [15] and was conducted within the NEESWood Project. The test specimen consisted of two single-story shear walls with dimensions equal to 2.4m long by 2.45m high, as shown in Fig. 8a. Since the walls were identical and the deformations recorded were similar, only one side of the specimen was considered in the numerical model using half of the actual seismic weight. The shear wall incorporated a single OSB panel 1.22m-by-2.44m because it served as a benchmark structure for comparison with a retrofitted damper-wall with similar dimensions [15]. The framing consisted of nominal 2x6 (38mm-by-140mm) studs spaced at 400mm o.c., using Spruce-Pine-Fir lumber. The sheathing provided was OSB, 11mm thick. Sheathing panels were fastened to the framing with 8d common nails, 63.5mm long with 3.3mm diameter. Edge nailing was specified at 150mm o.c. The seismic weight acting at the top of each wall was 30kN. More information on the geometric and structural characteristics of the shear wall specimens can be found in [15], while the properties and parameters utilized in the constitutive models are presented in [12].

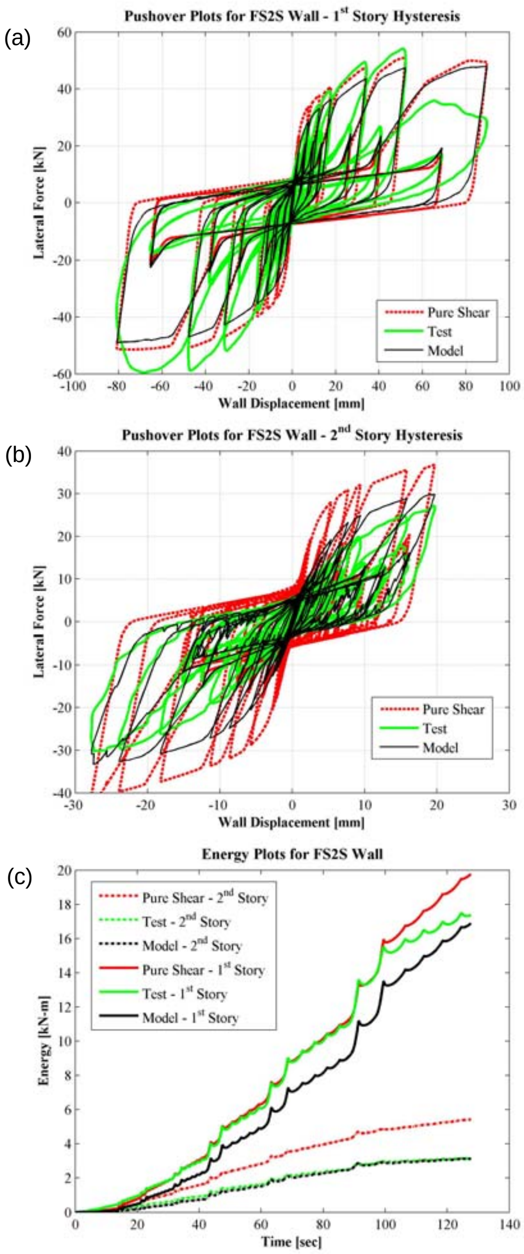

Figure 7: *Comparison of Cyclic Response for (a) the First Story; and (b) the Second Story of FS2S Wall.*

Three shake-table tests with increasing amplitude were conducted using the same test structure. The ground motions were selected from the 1994 Northridge Earthquake. The first two motions were selected from the Canoga Park record with a scale factor of 0.12 and 0.40, respectively. The third ground motion was selected from the Rinaldi record, with a scale factor of 0.4. The acceleration response spectra corresponding to the achieved ground motions recorded during shake-table testing are illustrated in Fig. 9.

The initial natural frequency of the test structure was equal to 3.85Hz [15], which compares well with the **Model** prediction of 3.98Hz and the **Pure Shear** prediction of 4.42Hz.

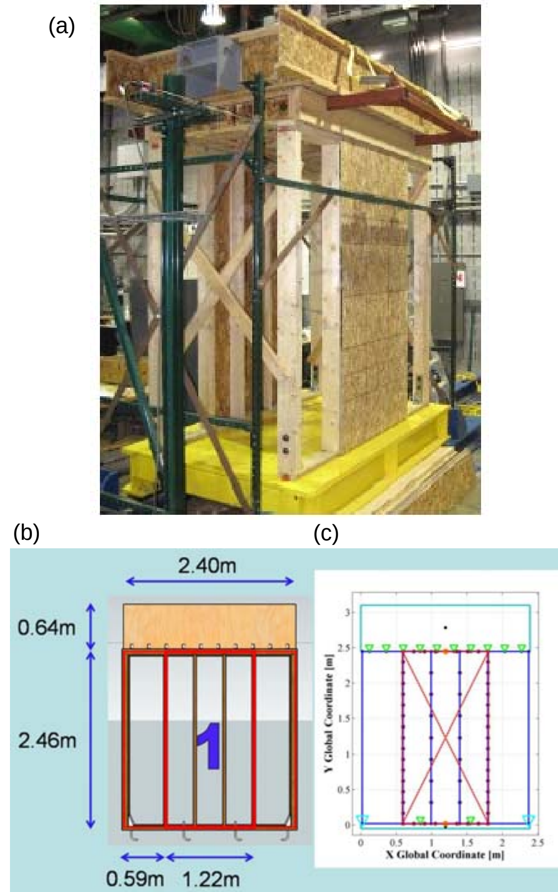

Figure 8: *(a) Photo of the Test Specimen (after [15]) (b) Geometric and Panel Configuration; and (c) Numerical Model of the RPI Wall.*

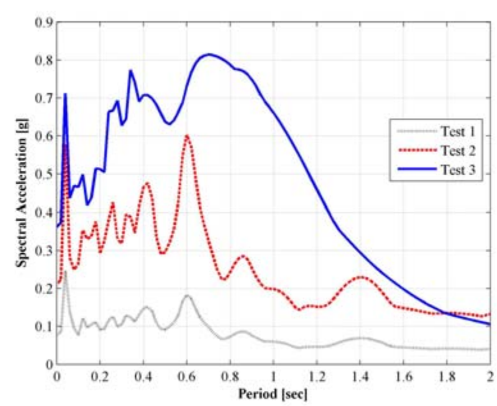

Figure 9: *Acceleration Response Spectra of the Achieved Input Ground Motions*

Stiffness and mass proportional Rayleigh damping was selected such as to provide a damping ratio of 1% of critical for the first (horizontal) mode of vibration and 5% of critical for the second (vertical) mode.

Figure 10 illustrates the recorded and predicted force-displacement responses for each of the three tests. It is observed that the predicted response is generally stiffer than the recorded response but evaluating the overall performance, the **Model** predicted responses correlate relatively well among the three tests.

The time-histories of inter-story-displacements and velocities and the absolute acceleration time-histories at the top of the wall are illustrated in Figs. 11, 12 and 13, respectively, for **Test 2** and **Test 3**. Reasonable correlations can be observed between the numerical predictions and the experimental results.

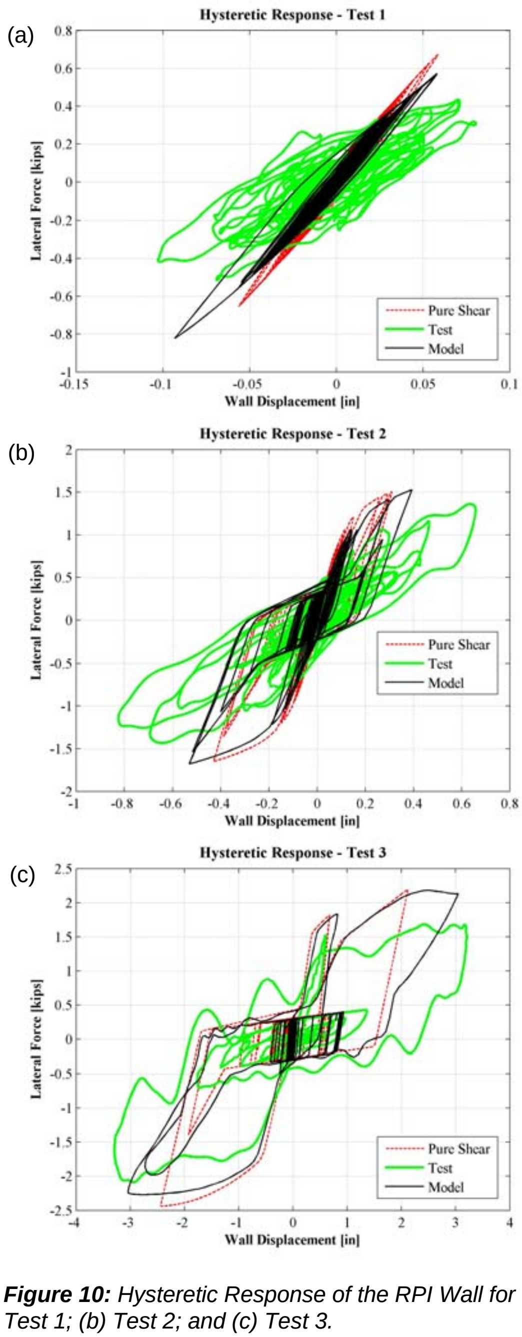

Figure 10: *Hysteretic Response of the RPI Wall for (a) Test 1; (b) Test 2; and (c) Test 3.*

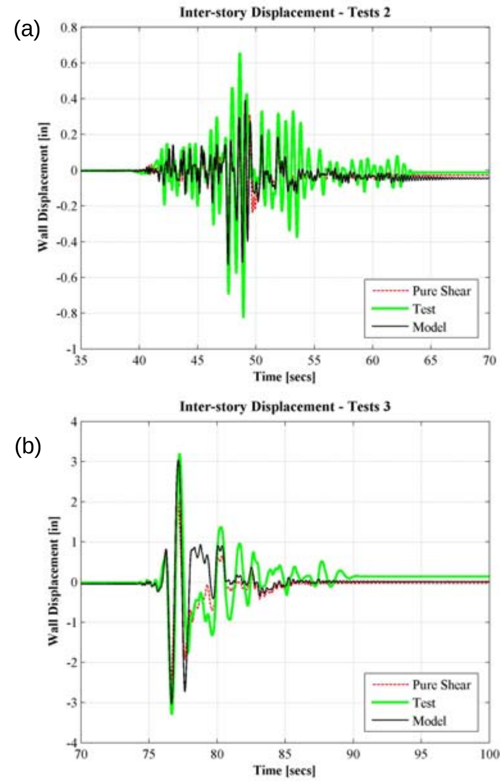

Figure 11: *Inter-story Displacement of the RPI Wall for (a) Test 1; and (b) Test 2.*

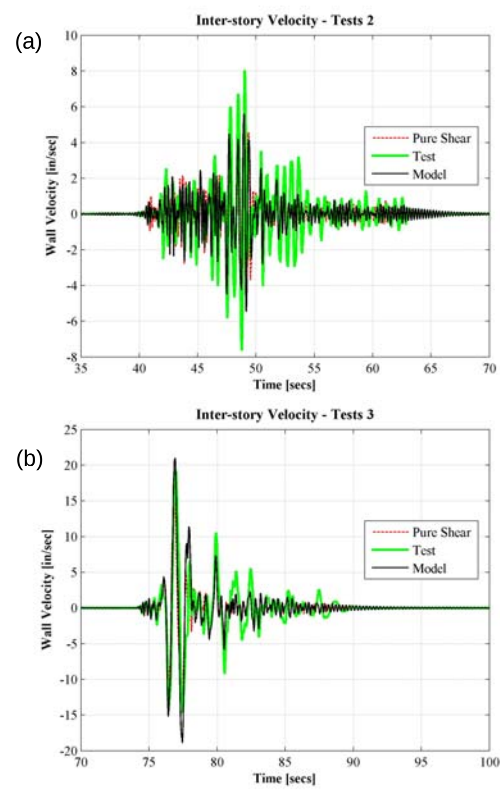

Figure 12: *Inter-story Velocity of the RPI Wall for (a) Test 1; and (b) Test 2.*

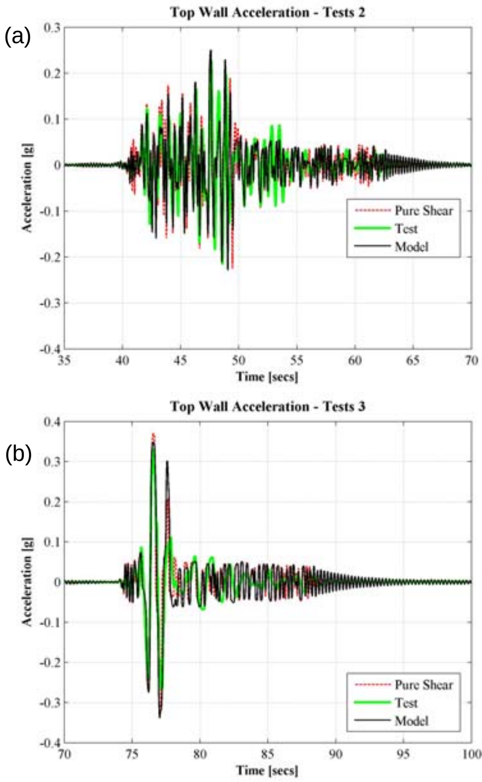

Figure 13: *Top of Wall Acceleration of the RPI Wall for (a) Test 1; and (b) Test 2.*

5 CONCLUSIONS

This paper introduced a novel numerical framework for the detailed nonlinear analysis of two-dimensional light-frame wood structures. The numerical **Model** predictions demonstrated a very good correlation with the equivalent **Test** results from 2 full-scale specimens, while the **Pure Shear** predictions provided an upper bound of the lateral performance characteristics of the shear wall assemblies and were significantly higher for the GD wall and the 2^{nd} story wall. The capability to perform a dynamic analysis of a detailed shear wall model was demonstrated.

ACKNOWLEDGEMENTS

The material presented in this paper is based upon work supported by the National Science Foundation under Grant No. CMMI-0529903 (NEES Research) and CMMI- 0402490 (NEES Operations). Any opinions, findings, and conclusions or recommendations expressed in this material are those of the author(s) and do not necessarily reflect the views of the National Science Foundation.

REFERENCES

[1] Ceccotti, A., Follesa, M. and Karacabeyli, E., 2000. “3D Seismic Analysis of Multi-Storey Wood Frame Construction,” *Proc. of 6th WCTE,* British Columbia, Canada.

[2] He, M., Lam, F. and Foschi, R.O., 2001. “Modeling Three-Dimentional Timber Light-Frame Buildings,” *Journal of Structural Engineering,* ASCE, **127**(8):901-913.

[3] Folz, B. and Filiatrault, A., 2004. “Seismic Analysis of Woodframe Structures I: Model Formulation,” *Journal of Structural Engineering,* ASCE, **130**(9):1353-1360.

[4] Dujic, B. and Zarnic, R., 2004. “Method for Modelling Dynamic Response of Timber Frame Building,” *Proc. of 8th WCTE,* Lahti, Finland.

[5] Collins, M., Kasal, B., Paevere, P. and Foliente, G.C., 2005. “Three-Dimentional Model of Light Frame Wood Buildings I: Model Description,” *Journal of Structural Engineering,* ASCE, **131**(4):676-683.

[6] Ayoub, A., 2007. “Seismic Analysis of Wood Building Structures,” *Engineering Structures,* **29**:213-223.

[7] Xu, J. and Dolan, J.D., 2009. “Development of a Wood-Frame Shear Wall Model in ABAQUS,” *Journal of Structural Engineering,* ASCE, **135**(8):977-484.

[8] van de Lindt, J.W., Pei, S., Liu, H. and Filiatrault, A., 2010. “Three-Dimentional Seismic Response of a Full-Scale Light-Frame Wood Building: Numerical Study,” *Journal of Structural Engineering,* ASCE, **136**(1):56-65.

[9] Filiatrault, A., Christovasilis, I.P., Wanitkorkul, A. and van de Lindt, J.W., 2010. “ Experimental Seismic Response of a Full-Scale Light-Frame Wood Building,” *Journal of Structural Engineering,* ASCE, **136**(3):246-254.

[10] Judd, J.P. and Fonseca, F.S., 2005. “Analytical Model for Sheathing-to-Framing Connections in Wood Shear Walls and Diaphragms,” *Journal of Structural Engineering,* ASCE, **131**(2):345-352.

[11] Christovasilis, I.P. and Filiatrault, A. 2009. “Two-dimensional Numerical Modeling of Light-frame Wood Structures for Seismic Collapse Assessment”, *Proceedings of COMPDYN 2009 – Computational Methods in Structural Dynamics and Earthquake Engineering*, Rhodes, Greece.

[12] Christovasilis, I.P. 2010. “Numerical and Experimental Investigations of the Seismic Response of Light-Frame Wood Structures”, Ph.D. Thesis, University at Buffalo, State University of New York, Buffalo, NY.

[13] Pardoen, G.C., Walman, A., Kazanjy, R.P., Freund, E. and Hamilton, C.H., 2003. “Testing and Analysis of One-Story and Two-Story Shear Walls Under Cyclic Loading,” CUREe Publication No. W-25, CUREE-Caltech Woodframe Project.

[14] Krawinkler, H., Parisi, F., Ibarra, L., Ayoub, A. and Medina, R., 2003. “Development of a Testing Protocol for Woodframe Structures”, CUREe Publication No. W-02, CUREE-Caltech Woodframe Project.

[15] Shinde, J.K. and Symans, M.D., 2010. “Integration of Seismic Protection Systems in Performance-Based Seismic Design of Woodframed Structures,” NEESWood Report No. NW-06, Dept. of Civil and Environmental Engineering, Rensselaer Polytechnic Institute, Troy, New York.