A Simulation-based Investigation of Adhesive Construction to Enhance Hazard Resilience of Wood Frame Residential Building

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ABSTRACT

The objective of the research is to study the state-of-the-art modelling methods for wooden frame building construction. These modeling methods are at different levels of variations and complexities. The numerical modelling tools are categorized into academic tools and commercial tools and the modeling methods are classified based on the structural systems (i.e., shear walls and the whole building structures) and applied loads (i.e., wind loading and seismic loading). The academic tools were mainly developed for seismic research purpose with specific objectives such as defining the behavior of wooden frame shear walls, hysteretic behavior of connections between the sheathing and framing members under seismic loading. Models created using commercial tools, on the other hands, are generally used to predict structural responses under seismic and wind loadings and are usually validated using experimental results. Two of the commercial tools widely used for creating wood structural numerical models are ABAQUS/CAE and SAP2000. The simplified modelling method including inbuilt SAP functions was studied from the literature and the detailed modelling process was developed and presented. Both linear and nonlinear analysis of wooden frame structure was carried out considering wind and seismic loading conditions. Lastly, recommendations for future research are provided.
ACKNOWLEDGEMENT

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CHAPTER 1: INTRODUCTION

1.1 Wooden Frame structures and its significance

USA has been a country with dynamic environment with various hazards such as hurricanes, tornados, storms earthquakes, etc. Due to better performance in the dynamic environment and physically being light, replaceable and hollow, about 80% of the total building stock in the US and well over 90% of the residential building in North America are light frame wood building construction. On one hand, high degree of life safety is achieved during moderate and light earthquakes. On the other hand, these structures are vulnerable to costly damage at small loads of deformation. As more than 90% of the US population lives areas that are seismic active and prone to extreme wind hazards, it is imperative to design and build safe wood buildings to not only protect occupants and households, but also decrease damage loss when such hazards occur. Figure 1-1 (b) illustrates a typical wood frame building construction with load transferring path, along which shear wall (see Figure 1-1 (a)) is considered to be main force resisting element that needs to be able to safely carry both lateral and vertical loads induced by the hazardous loadings.

1.2 Research objective

The objective of this project is to study the state-of-the-art modelling methods in wood frame building construction under hazardous loadings. In addition, detailed modelling procedure of wood
buildings and components subjected to wind and seismic loading using the identified numerical modeling methods are presented which can be applied in future research for numerical investigation of various structural strengthening approaches.

1.3 Scope of current effort

This project is intended to identify the suitable simulation-based investigation method that is to be used in the development of a wood frame building model with enhanced strength and stiffness. Specifically, the following research tasks are performed and presented in this report:

1. Identify and study existing numerical modeling tools and the state-of-the-art modeling methods for wood structural responses simulation subject to wind and seismic loadings

2. Establish models of wood frame building structure and shear wall assembly utilizing the selected commercial tools and modeling methods and analyze their responses subject to uplift wind and lateral seismic loadings, respectively.
CHAPTER 2: LITERATURE REVIEW

2.1 Introduction
Various numerical modeling methods were developed for the design and analysis of wooden frame structural system under normal working loads and hazardous loadings. These numerical modeling methods have different levels of variations and complexities. Modern computers were also utilized to ease the procedure of numerical modelling of the wood structures and these computer modeling tools can be roughly categorized into academic tools mainly for research purposes and commercial tools mainly for engineering practices. This section is intended to provide a comprehensive literature review on these numerical modeling tools for wood frame structures that were developed within the last two decades. The numerical modelling tools under academic and commercial categories will be further classified based on the structural system of the models (i.e., shear walls and the whole building structures) and the applied loads (i.e., wind loading and seismic loading).

2.2 Academic Tools
This section introduces the numerical modeling tools that have been developed by researchers in the academic community mainly for research purposes. From literature, it was found out that the two major types of wood structural models considered in seismic analysis are shear walls and the whole building frame, that are separately discussed next.

2.2.1 Shear wall under seismic loads
Numerical tools developed to simulate the seismic response of wood shear walls are introduced, including structure modelling techniques, seismic loading that are considered and simulated responses using these tools. The validation of these numerical tools/methods is generally made through comparing the simulated responses with the experimental results, that is also presented herein.

The CUREE-Caltech wood frame project was funded by the Federal Emergency Management Agency (FEMA), with an objective to investigate and implement engineering features of the wooden frame structure to reduce seismic loss. The project consisted of five elements which
included testing and analysis, field Investigation, building codes and standards development, economic loss estimate due to seismic event and education and outreaching. To understand wood frame seismic behavior, the CUREE-Caltech project conducted full scale shake table tests to realistically replicate the dynamic response of wood frame structures of various construction configurations. Other tasks included in the CUREE-Caltech project together with their respective objectives are presented listed in Table 2-1, where the development of seismic analysis software for wood frame construction is listed as Task 1.5.1. Numerical simulation programs were therefore developed under this task and theses programs formed the bases of the academic wood structure simulation tools that are discussed in this section.
<table>
<thead>
<tr>
<th>Tasks Number</th>
<th>Name of Task</th>
<th>Description of Tasks</th>
<th>Objective</th>
</tr>
</thead>
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<tr>
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<td>Shake Table Test</td>
<td>A full-scale two-story single family house</td>
<td>To measure and quantify dynamic response of wooden frame building’s construction configuration and document force distribution.</td>
</tr>
<tr>
<td>1.1.2.</td>
<td>Organization of an International Benchmark</td>
<td>A full-scale Multi-story Apartment Building with Tuck-under Parking</td>
<td>Shake table tests of structure following 1960’s engineering practice and improve its seismic performance using special moment resisting frame in open front of first story.</td>
</tr>
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<td>1.1.3</td>
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<td></td>
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</tr>
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<td>Static and dynamic tests on wood framed shear walls</td>
<td>To compare results from different protocol and load rates.</td>
</tr>
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<td>To develop understanding on behavior of diaphragm-to-foundation connections.</td>
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</tr>
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<td>Develop understandings of damage characteristics and relate seismic response to wall repair cost.</td>
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</tr>
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<td>Modelling of uncertainty analysis, response variability analysis and selection of reliability-based approach to shear wall design/selection.</td>
</tr>
</tbody>
</table>
2.2.1.1 *Cyclic Analysis of Wood Shear Walls (CASHEW)*

Folz et al. (2001) developed a numerical model that predicted the load-displacement response and energy dissipation characteristics of wooden shear walls. The prediction was made under the general quasit-static cyclic loading. The shear wall was composed of three structural components which were rigid framing members, linear elastic sheathing panels and non-linear sheathing-to-framing connectors. The model is considered to be simple and efficient that has minimum number of path-following rules capable of reproducing the response of the connector under the general cyclic loading. The hysteretic behavior of the sheathing to framing connector is defined using 10 parameters as shown in Figure 2-1. **Error! Not a valid bookmark self-reference.** gives the definition of these 10 parameters used in the model. These parameters were determined by fitting the hysteretic response obtained from the numerical model with the experimental results. This model was then incorporated into a computer program named “Cyclic Analysis of Shear Walls” (CASHEW).

![Figure 2-1: Force-displacement response of sheathing-to-framing connector under monotonic and cyclic loading (Folz et al., 2001)](image_url)
<table>
<thead>
<tr>
<th>Parameter</th>
<th>Definition</th>
</tr>
</thead>
<tbody>
<tr>
<td>$K_0$(kN/mm)</td>
<td>Initial stiffness of sheathing to framing connector</td>
</tr>
<tr>
<td>$r_1k_o$</td>
<td>Asymptotic stiffness of sheathing-to-framing connector under monotonic load</td>
</tr>
<tr>
<td>$r_2k_o$</td>
<td>Post ultimate strength stiffness of sheathing-to-framing connector under monotonic load</td>
</tr>
<tr>
<td>$r_3k_o$</td>
<td>Unloading stiffness of sheathing-to-framing connector</td>
</tr>
<tr>
<td>$r_4k_o$</td>
<td>Re-loading pinched stiffness of sheathing-to-framing connector</td>
</tr>
<tr>
<td>$F_o$(kN)</td>
<td>Force intercept of the asymptotic line for the sheathing-to-framing connector</td>
</tr>
<tr>
<td>$F_1$(kN)</td>
<td>Zero-displacement load intercept of sheathing-framing connector</td>
</tr>
<tr>
<td>$\Delta_u$(mm)</td>
<td>Displacement developed in the connectors corresponding to ultimate load</td>
</tr>
<tr>
<td>$\alpha$</td>
<td>Hysteric model parameter that determine the degree of stiffness degradation</td>
</tr>
<tr>
<td>$\beta$</td>
<td>Hysteric model parameter</td>
</tr>
</tbody>
</table>

The ability of the model to predict the response was validated through comparison to the results obtained from the full-scale monotonic and cyclic tests of shear walls performed at the University of British Columbia, as shown in Figure 2-2 (a and b). In addition, the model was verified based on its ability to predict the energy dissipation. The comparison plot of the energy absorbed during cyclic shear wall tests and determined using the CASHEW model is shown in Figure 2-3. As can be observed from the figures, the CASHEW model is capable of predicting load displacement and the energy dissipation of the wooden shear walls under cyclic loading. This tool set up the foundation for the development of advanced, generic and less complex analytical models. The CASHEW model was incorporated in the cyclic load and earthquake time history analysis of complete 3D wood-framed buildings (i.e., SAPWood). Also, the cyclic test data of sheathing to framing connectors can be used to calibrate the 10 parameters of the model which can be used to represent equivalent hysteretic element of the shear wall’s global lateral loading resistance.
Figure 2-2: (a) Experimental monotonic and cyclic Shear wall tests (b) CASHEW predictions of monotonic and cyclic shear wall tests (Folz et al., 2001)

Figure 2-3: Comparison plot for energy dissipation (Folz et al., 2001)
2.2.1.2 Evolutionary Parametric Hysteretic Model for Wood Shear Walls (EPHMS)

The CASHEW model was proved to be simple to use and an attractive tool for the numerical analysis. Pang et al. (2007) incorporated more features into the CASHEW model to improve its prediction capability, named as the Evolutionary Parametric Hysteretic Model for Wood Shear Walls (EPHMS). A series of exponential functions that defines the backbone curve, loading and unloading path are used to construct the EPHMS model. The backbone curve for the model is shown in Figure 2-4. And the EPHMS model, as shown in Figure 2-5, requires 17 parameters to capture the response of the shear walls. The seven parameters listed in Table 2-3 are used in the two exponential functions with evolutionary parameters that define the ascending and descending envelopes for the force - displacement relation of the walls. The backbone function is estimated using the least-square regression or visual inspection of the actual hysteresis loops as all the parameters have physical interpretations associated with it.

![Figure 2-4: Shear wall backbone curve (Pang et al., 2007)](image)

**Table 2-3: Seven Static Parameter Defining the Shear Wall Backbone Curve**

<table>
<thead>
<tr>
<th>Parameters</th>
<th>Definitions</th>
</tr>
</thead>
<tbody>
<tr>
<td>$K_o$</td>
<td>Initial tangent stiffness of the backbone curve</td>
</tr>
<tr>
<td>$F_o$</td>
<td>Restoring Force</td>
</tr>
<tr>
<td>$K_d$</td>
<td>Tangent stiffness degradation</td>
</tr>
<tr>
<td>$\delta_u$</td>
<td>Displacement associated with the maximum force</td>
</tr>
<tr>
<td>$K_x$</td>
<td>Maximum tangent negative stiffness</td>
</tr>
<tr>
<td>$\delta_x$</td>
<td>Deflection where maximum decay rate occurs</td>
</tr>
<tr>
<td>$F_x$</td>
<td>Upper force asymptote</td>
</tr>
</tbody>
</table>
Figure 2-5: EPHM shear wall backbone, loading and unloading curves (Pang et al., 2007)

These curves are modified according to the updates made on the evolutionary parameters. As shown in the Figure 2-5, the force intercept $F_i(K_i)$ and the shape parameter $\lambda_u$ (mm$^{-1}$) are the evolutionary parameters. Evolutionary parameters are updated by the ten parameters of the degradation function as shown in Figure 2-5. The evolutionary parameters are derived from the multiple damage tracking indices used for modelling cumulative damage of wood shear walls.
After the development of all the backbone curves, loading and unloading paths, the EPHM is evaluated using the cyclic test results of eight full-scale shear walls with varying wall configurations and different cyclic loading protocols. All the seventeen parameters are extracted based on the testing results using an extraction program with a graphical user-interface developed in MATLAB. The advantage of the improvement in predicting the hysteretic model was examined by comparing the simulated results between the EPHM and the CUREE model. The factors that were used in determining the accuracy of the model were cumulative energy dissipation, cumulative force error (CFE) and cumulative energy error (CEE), as illustrated in Figure 2-7. It is observed that the cumulative energy error of the EPHM model and the actual test wall is less than that of CUREE model. The accuracy of the energy dissipation in EPHM model in maintaining large displacement makes EPHM a choice for performance-based design application.
2.2.1.3 *Matlab- Cyclic Analysis of Wood Shear Walls 2 (M-CASHEW2)*

The wooden shear walls models, such as CASHEW and SAPWood described on section 2.2.2.1, assumed that the framing members are rigid and the bottom plates are anchored fully to the foundation. Such models are, however, unable to accurately predict the response of the wooden shear walls that considers the significant uplift of the end stud. In terms of the deformation theories, CASHEW and SAPWood only consider small deformation and neglect the P-Δ effects. To overcome this limitation, a model was developed specifically for nonlinear collapse analysis of wood shear walls where large deformation is expected.

The frame element is modeled as a two-node beam elements which is based on the Lagrangian formulation. The node consists of three degrees of freedom which are two translations and one rotation. Similarly, sheathing elements are modeled as rectangular panel element with five degrees of freedom. The five degrees of freedom are one rigid body rotation, two rigid body translations
and two in-plane shear deformation. The assumption that the sheathing panels have sufficient stiffness is adopted which provides privilege of ignoring the out-of-plane deformation.

There are three types of connection elements employed in this shear wall model, which are panel-to-frame, frame-to-frame, and panel-to-panel connections. Figure 2-8 shows these types of connection between the elements, where, $d_{cf}$ is the displacement of the frame elements and $d_{cp}$ is the displacement of the panel elements. A brief description about the connection elements are given below.

![Connection elements](image)

**Figure 2-8: Connection elements (Pang et al., 2007)**

1. **Panel-to-frame connection (P2F):** A zero-length element with three uncoupled springs (i.e., one rotational and two transitional springs) represents the connecting elements. A two-node panel-to-frame connection is employed in which one node represents the location of the nail head on the panel and the other node represents the location of the nail shank in the frame.

2. **Frame-to-frame connection (F2F):** The F2F connection represents five types of connections in the wooden shear wall structure.
   a. Double end studs or top plates connection
   b. Framing members put together at 90-degree angle
   c. Phenomena of contact and separation between framing members
   d. Hold down for anchoring end studs to foundation
e. Anchor bolts for anchoring bottom plates to foundation

3. **Panel-to-Panel connection (P2P):** A hysteretic contact spring element is used to model the contact and the gap between the shear wall components which is an interface between frames, sheathing panels, frames and foundation and between panels and foundation.

A computer program is coded and named as M-CASHEW2, which accounts for all the features, assumption and the modelling components as discussed above. The evaluation of this modelling method was carried out on the basis of sensitivity studies under monotonic and cyclic loadings. Figure 2-9 shows the deformed shear wall in simulated using the M-CASHEW2 modeling method.

![Figure 2-9: Shear wall view in M-CASHEW2 (deformed) (Pang et al., 2007)](image)

![Figure 2-10: Shear wall back bone curves Pang et al., 2007)](image)
Figure 2-10 shows the prediction of shear wall backbone curve under monotonic loads with and without hold downs (HD). The graph shows increase in the peak backbone force in the presence of the hold down devices which verifies that the modelling process of the shear walls is reasonable. Similarly, Figure 2-11 shows the hysteresis loop for the shear walls with non-oriented and oriented panel to frame spring pair models. The results show that the oriented spring pair model increases the pinching effect in the hysteresis loop. The oriented panel to frame (P2F) spring pair model exhibited the over pinching effect as expected and was studied.

Figure 2-11: Hysteresis loops for the shear walls (Pang et al., 2007)

2.2.1.4 LightFrame3D

Different analytical tools were developed to represent the load-displacement relationship and the hysteresis behavior of wood shear walls. However, some of these hysteresis model is unable to provide satisfactory results when variations in material properties and joint configuration exist. To overcome this limitation, a mechanics-based approach for modelling the connection was proposed. In this approach, the nail connection is considered as an elastic plastic beam modeled as a non-linear medium acting in compression which also permits the formation of gaps between the beam and the medium. LightFrame3D has the capability to predict structural performance under varying material, structural and load combinations. The input history is either load controlled or displacement controlled. One of the important ability of this modeling method is that P-Δ behavior of a panel and frame members due to axial compressive load is taken into account.
Basically, three types of elements are used in this modeling method which are frame, panel and connection element. Panel elements are thin plate elements in 3D plane. Frame elements are 3D beam elements with inelastic material property and the nonlinear spring elements defines the nail connections. The configuration of the model elements is illustrated in Figure 2-12.

![Figure 2-12: Structural configuration (Lam and Foschi, 2001)](image)

Cyclic loading consisting of three identical cycles with 30, 50 and 80% of the nominal yield displacement was applied on the experimental model. Similarly, the numerical prediction was compared with the experimental result which shows good agreement, as shown in Figure 2-13. The result obtained from the numerical analysis demonstrates the confidence of using the numerical model to investigate structural responses under different loadings without the need of physical experiments. In addition, this modelling method can be employed to analyze a complete 3D wood frame building as well.
Figure 2-13: Prediction of LightFrame3D vs experimental results (Lam and Foschi, 2001)

2.2.2 Wooden Frame Buildings Under Seismic Loads

It was noted by B. Folz and A. Filiatrault (2004) that the structural components of the wooden frame buildings namely framing, sheathing and connection have a standard process that are coded for the construction purpose but lacks analysis tools. The absence of tools for analysis causes lack in evaluation of the full performance of the buildings under lateral loads. Based on the modelling techniques of the wooden shear wall, there are different numerical tools that have been developed for the numerical modelling of wooden frame buildings. This section will describe the tools that has been developed for the structure modelling techniques. Types of loading considered, the prediction made using these tools and then validate against the experimental results are also presented in this section.

2.2.2.1 Seismic analysis Package for wood frame structures (SAPWood)

SAPWood developed by Van de Lindt et al. (2010) is a numerical tool that was developed to predict the dynamic response of the light-frame wood buildings. SAPwood consists of both research and analysis tools and is comprehensive in simulation of both structural and nonstructural systems’ seismic performance. The three main structural components modelled in SAPWood are the nonlinear hysteretic springs for the shear wall elements, non-symmetric linear springs for uplift restraint and compression studs/struts and rigid diaphragms for the roofs and floors. The EPHMS model for shear wall modeling is utilized in the SAPWood to create a numerical model for non-linear hysteretic spring element of a shear wall. The hysteretic parameters of the shear wall elements were determined using the nail pattern (NP) analysis module.
The hysteretic parameters of the shear wall elements were determined using the nail pattern (NP) analysis module as shown in Figure 2-14. The NP analysis module enables modeling stud and sheathing panel as a rigid member and nail as a hysteretic spring. Similarly, based on the CASHEW model, the framing members are assumed to be pin-connected. One of the features of the NP model is that the wall elements could be set up based on the wall geometry and the hysteretic parameters of the wall could be obtained. The hysteretic parameters are determined by the process of numerical reversed cyclic push-over analysis using the predefined displacement protocol. Group of equivalent non-symmetric linear springs in series represents the vertical elements that connect the floor diaphragms in the vertical direction so their participation in resisting the out of plane diaphragm rotation is considered. After acquiring both hysteretic parameters for all shear walls and the vertical spring, the building system-level models can be built.
Figure 2-15: Wall model considered in numerical model (van de Lindt et al., 2010)

The SAPWood’s capability of simulating shear wall responses was demonstrated by comparing to the test results of the test building at different phases of construction. Test buildings in different construction phases were developed in SAPWood. The recorded shake table accelerations were used as an input in the numerical verification. The comparison between the simulation and the test results of the peak inter-story drift at the centroid of the floor diaphragms is shown in Figure 2-16. The results showed good agreement between the numerical prediction and the experimental results at various construction phases, including structure with dry wall, wood-only building and complete structure with stucco finishes.
The wood frame buildings are a combination of different components which includes framing members, sheathing members and the connection elements between the sheathing and framing members. The connection elements exhibit generally non-linear behavior along with strength and stiffness degradation under cyclic loading. The built-up components create high degree of redundancy boosting the complexity in modelling. Creating finite element model reduces computational time of such structures or elements. Therefore, Folz and Filitrault (2004) implemented a model reduction technique. This technique involved degeneration of the actual three-dimensional building to a two-dimensional planar model composed of zero-height shear wall spring element connecting floor and roof diaphragms or the foundation. This approach reduces the number of response to be modeled for the building to 3DOF per floor. The shear wall spring elements were calibrated to reflect the strength and stiffness degrading hysteresis characteristics.

Figure 2-16: Comparison for SAPWood simulations vs experiments (Folz and Filitrault, 2004)
The numerical model for wooden frame buildings created using this method is incorporated into a computer program named Seismic Analysis Of Wooden Frame Structure (SAWS).

The validation of the modeling method was done by comparing its simulated results to experimental results of shake table tests on a full-scale two-story wood frame house. The model developed was also implemented for the analysis of the CUREE-Caltech Wood frame Project test structures. The test program included 10 construction phases among which phases 9 and 10 were used to validate the SAWS model as briefly described below.

Phase 9 testing structures consists of only the bare wood framing, the structure consisted of eight zero-height shear wall spring elements and two rigid diaphragms, one for the second floor and one at the roof level as shown in Figure 2-18 (a). The force-deformation response of the shear wall spring element was determined using the 10 hysteretic parameters from the CASHEW model. Test structure in phase 10 is like the ones in phase 9 except for the exterior and interior finishes added to the wood framing.

Figure 2-17: Structural configuration and elevation of phases 9 and 10 (Folz and Filiatrault, 2004)
Phase 10 structures consisted of both exterior stucco and interior gypsum wall board and its SAWS model is shown in Figure 2-18 (b), which is made up of 27 zero height shear spring element that represented 8 exterior walls covered with stucco, 8 walls sheathed with OSB and 11 gypsum wall board covered walls. As in the SAWS model of phase 9 this SAWS model also included two rigid diaphragms one each for the second floor and one for the roof level. Again, the hysteretic parameters for the shear springs were obtained from the CASHEW model for Oriented Strand Board (OSB) sheathing whereas for the walls with Gypsum Wall Board (GWB) and with stucco the parameters were obtained from test data.

Figure 2-18: SAWS model of (a) Phase 9; and (b) Phase 10 (Folz and Filiatrault, 2004)
The overall numerical prediction on the dynamic characteristics, push-over capacities and seismic responses of the structure made by SAWS were in good agreement with the results obtained experimentally. Figure 2-19 displays the relative displacement time histories (Folz and Filiatrault, 2004) for phases 9 and 10 respectively. For the phase 9 testing, the SAWS model under predicted the experimental values, which may be due to two major reasons. One is due to the inability of the model to capture the torsional response and the other is not considering in-plane deformation of the roof diaphragm. However, the SAWS model of phase 10 over-predicted the experimental values due to the simplification used in the SAWS model where only initial estimate of shear wall elements’ parameters were used without updating.

Figure 2-19: Phase 9 Relative displacement time histories (a)SAWS; (b)Experimental

Overall, the SAWS model was able to predict the dynamic responses of the wood frame building structures with good agreement with the experimental results. The prediction either under or over were within 10%. One major limitation of the SAWS model is the assumption of rigid diaphragms which is main cause for the difference in the predictions and the experimental results. But, the simplicity of the model allows to represent each floor with only three degrees of freedom. All in all, the SAWS model meets the requirement as a simple computational tool for the
professionals and researchers to evaluate dynamic characteristics, quasi-static pushover and seismic response of wood frame buildings.

2.3 Commercial Tools

From the study of different tools that were developed in the course of time and observing their results compared with different test results, it can be concluded that analytical modeling approaches of wood frame structures provide an alternative way to evaluate structural seismic performance. Compared to the conventional experimental investigation, analytical modeling and numerical simulation is often more cost and time effective. The numerical tools described in the previous sections were generally developed for research purpose. The level of complexity of such model can be adjusted for different analysis purposes. This section will further introduce modelling methods based on commercial finite element software package, such as “ABAQUS” and “SAP2000” and their applicability to analyze the wood frame structures. The development of modelling methods in this two software and their validation against experimental results are introduced.

2.3.1 Wood Frame Modelling using ABAQUS/CAE

ABAQUS is a commercial finite element software that is used to model and analyze different structural components under different type of loadings. From the study of the academic modeling tools, it is clear that hysteretic behavior of nailed wood joints is the main contributor to the response of the wood frame system subjected to lateral loading. However, hysteretic elements that suits the behavior of nailed wood joints are not readily available in ABAQUS. The following sections describe the modelling methods employed in ABAQUS for both shear walls and wood frame buildings.

2.3.1.1 Shear wall Modelling

In the shear wall modelling by Xu (2006), a beam (B31) element is used to represent studs and top and bottom plates. Beam (B31) is a 2-node 3-D linear beam element, and each having 6 degrees of freedom A S4 shell element is used to represent sheathing panels. S4 is a 4-node general purpose shell element each node having 6 degrees of freedom. The orthotropic characteristics of the sheathing material was considered which defined the different elastic modulus in the two orthogonal directions. The assumption made by Xu (2006) is that the framing members and sheathing panels acts linear elastically while nonlinear responses is solely contributed by the nail
connection joints. As mentioned previously, the ABAQUS library does not have an appropriate hysteretic element that can be used to model the nail wood joints but it allows the user define the element which can be executed as a user subroutine. Such subroutine is defined as a UEL which is programmed and compiled using the computer programming language FORTRAN.

The subroutine created is called on in each iteration for the element it is modeling. At each call, the nodal coordinates are provided by the ABAQUS/Standard including all the variables associated with the nodes (i.e., displacement, incremental displacements, velocities, accelerations) to all the degrees of freedom that is connected by the element. The ABAQUS/Standard also provides other parameters of the element that are connected to the element defined in the UEL and controls flag array indicating what function the user subroutine must perform (Xu, 2006). The contribution of the element to the stiffness matrix, updating the solution-dependent state variables that are associated with the element and much more are done by the control flags. In addition, ABAQUS/Post produces the output plots and extracts the data.

![Displacement vs Time](image.png)

**Figure 2-20: International Standards Organization (ISO) loading Protocol (Xu, 2006)**

Two models with the presence and absence of the opening were created. The walls were attached to the base through tie-down, anchor and shear bolts. To validate the shear, wall model developed using the ABAQUS modeling method, a cyclic loading experiment was carried out based on the International Standards Organization (ISO) loading protocol to generate test data for comparison purpose. Three tests considering full, intermediate and no anchorage conditions were performed. The numerical simulation showed a good agreement with the experimental data the under the full anchorage and intermediate anchorage conditions, thus proving the accuracy of the hysteretic nail joint element developed as the subroutine in ABAQUS model.
Figure 2-21: Shear wall model with (a) full anchorage; (b) intermediate anchorage (Xu, 2006)

Similarly, Rinaldin et al. (2013), created a nailed connection model in ABAQUS to simulate the nail behavior. The shear behavior of the nail in the two perpendicular directions were characterized by the cyclic hysteresis relationship as shown in Figure 2-22. Then this nail model was used to create an analytical model of the light frame timber wall that were tested under the cyclic loading by Dolan (1989) as shown in Figure 2-23.

Figure 2-22: Hysteresis piece wise linear relationship used in ABAQUS for nails and the diagonal springs modelled in ABAQUS (Rinaldin et al., 2013)
In ABAQUS, the wall components are modelled as shell elements (S4R) for the sheathing elements, beam elements (B21) for the framing elements and the non-linear hysteretic spring was defined as a User defined element as discussed above. The 2DOFs non-linear spring was calibrated to the strength and stiffness values based on the experimental behavior of the nails using a special software developed by the authors. The top and bottom plates of the frames are pin connected to the studs. The frames are simply supported at the base and the top left corner of the wall was subjected to an imposed cyclic horizontal displacement history defined by the CUREE standard protocol (see Figure 2-24).

Figure 2-23: Layout of the timber frame walls tested by Dolan 1989 (Rinaldin et al., 2013)
The base shear vs the top displacement simulated by the model was compared to the experimental data published. A good agreement between the numerical and the experimental data were observed data as shown in Figure 2-25.
2.3.1.2 Building Modelling

The proper and accurate shear wall analysis brings to the proper numerical modelling of the whole wood frame building model. For this purpose, several researches are carried out that were able to predict the building response when compared to the corresponding experimental results.

One of the efficient and appropriate method proposed by Rinaldin et al. (2013) is illustrated in Figure 2-26 (a) for a building structure. The walls are modelled using truss element with very high axial stiffness, and the nail to frame connection was modeled using diagonal spring elements as shown in Figure 2-26 (b). The diagonal non-linear spring elements are used to characterize the nonlinear strength and stiffness of the entire shear wall. These diagonal springs needs to be calibrated for the walls of different dimensions. The hysteretic model described in the previous section of the shear wall is implemented in this model. The calibration of the spring element is achieved by minimizing the difference in the total energy between the sets of data set obtained from the analysis of the shear wall as described in section 2.3.1.1 Error! Reference source not found. and the spring element through a software developed by Rinaldin (2011).

![Figure 2-26: (a) Wood frame building model; (b) wall model using diagonal spring element (Rinaldin et al., 2013)](image)
The building model was verified based on the mass and stiffness using modal analysis. Base shear vs. displacement of the numerical analysis result was compared with the experimental results as shown in Figure 2-27, where the numerical model is shown to be able capture the response of the structure. Differences are also visible which may be due to the approximations made during modelling for the presence of the wall opening.

![Figure 2-27: Base shear vs Top Displacement (Rinaldin et al., 2013)](image)

**2.3.2 Wood Frame Modelling using SAP2000**

Another popular computer software for structural design and analysis is Structural Analysis Program 2000 (SAP2000). The structural systems that can be analyzed in SAP2000 range from basic 2D to complex 3D structures. This section discusses the modelling of wooden frame structures using SAP2000.

**2.3.2.1 Shear Wall modelling**

According to Rinaldin et al. (2013), there is no hysteresis model specifically developed for the wooden structures in SAP2000. On the other hand, SAP2000 does provide several general hysteresis models used in structural analysis including the Pivot hysteresis model that accounts for the pinching effect and stiffness degradation. The Pivot hysteresis (see Figure 2-28) can be used to model the nonlinear behavior the nail connections in the shear wall, while the frame and the
sheathing are modeled using the standard beam element and thin shell element in SAP2000, respectively.

The pivot points seen in Figure 2-28 define the slope of loading, unloading and reloading branches. The parameters showed in the figure provides the following information. \( \alpha_1 \) and \( \alpha_2 \) defines the pivot points for the unloading from the positive part of the backbone curve and the negative part. Similarly, \( \beta_1 \) and \( \beta_2 \) defines the reloading towards the positive force and negative force, respectively. \( \eta \) defines the elastic stiffness degradation in plastic field. These parameters are calibrated so that difference in energy between experimental cycle and the numerical approximation is minimized. The same shear wall model (see Figure 2-26) described above in the ABAQUS section was modelled in SAP2000. A cyclic horizontal displacement history as shown in Figure 2-24 was applied to the top left corner of the model. Base shear vs. top displacement obtained from numerical simulation were compared to the experimental results and good agreement is observed (see Figure 2-25). Therefore, SAP2000 was demonstrated to be able to model wood shear walls and produces similarly accurate wall responses as ABAQUS.

Similarly, Simsir and Jain (2008) presented a modelling technique for the seismic evaluation of wood frame lateral load resisting walls. Their proposed SAP2000 model includes frame elements for wood studs, shell elements for wall sheathing and nonlinear link elements for sheathing to stud
fasteners. The analytical model developed was verified with the results from the shear wall tests performed as part of wood frame programs of the City of Los Angeles-University of California and CUREE.

**Figure 2-29: Analytical model in SAP2000 (Simsir and Jain, 2008)**

For the analytical model as shown in Figure 2-29, the shell elements were used to model the sheathing material, frame elements were used to model the wood studs. The connection between the vertical studs and the bottom plates were pinned so that shear deformation of sheathing with framing member can be achieved. The load versus deformation of the fastener was modelled using the non-linear link elements. The CUREE standard loading protocol was applied to the structure.
Figure 2-30: Deformed shape of structural model (Simsir and Jain, 2008)

The load vs. deformation response of the wall was compared between the analytical model and the experimental model (see Figure 2-31). The comparison plot shows that the responses of the analysis model and the experimental model have good agreement with each other.

Figure 2-31: Load vs Displacement comparison between wall model and test model (Simsir and Jain, 2008)

Doudak et al. (2006) proposed a finite element modelling method to represent wood shear walls with opening using SAP2000. The modelling method used linear frame element for the framing
component, shell element for the sheathing element and the non-linear spring elements for fasteners as shown in Figure 2-32.

![Shear wall model](image)

**Figure 2-32: Shear wall model (Doudak et al., 2006)**

The loading arrangement for the series experimental test was done according to the American Society for Testing and Materials (ASTM)-D1037-99 (Standard Test method for evaluating properties of wood-base fiber and particle panel materials). Comparison of the model and test results were done on the basis of load and deformation. One of the comparison plots is shown in Figure 2-33, from which one may observe that there is good agreement between the test result and the model prediction.

![Comparison plot](image)

**Figure 2-33: Experimental and analytical responses of shear wall (Doudak et al. 2006)**
2.3.2.2 Building Modelling

Rinaldin et al. (2013) proposed a modeling method for wooden frame buildings by representing the entire walls using two equivalent diagonal springs. Two numerical models were developed. For the model (a) shown in Figure -2-34 (a), the actual wall sizes are 1220mm, 915mm and 762.5mm. Model (b) contains eight identical shear wall of 2.44x2.44mm which were tested previously and modeled using the same components as in the (a) model (Du, 2003) (see Figure -2-34 (b)).

![Model (a) and Model (b)](image)

**Figure -2-34: Building model with the diagonal spring**

The rigid frame of the building was modelled as the beam element with the moments released at the end. The rigid floor of the building is modelled as shell element with diaphragm type edge constraints. The foundation of the building was modelled as the pinned connection. The walls of the building were modelled as diagonal spring with the single degree Multi-linear plastic pivot hysteretic behavior. An initial validation of the model in terms of mass and stiffness accuracy was carried out following the modal analysis procedure. The fundamental frequency of the structure was determined to be 3.65Hz, which differs from the experimental value obtained through the dynamic identification test by 7.8% on the real structure. Then time history analysis using the Northrigde accelerogram with a peak ground acceleration of 0.89g was performed. The relationship between base shear and the top displacement of the model was compared to the
experimental results which show the good agreement as shown in Figure 2-35.

![Graph of base shear vs top displacement](image)

**Figure 2-35: Comparison of base shear vs top displacement of SAP2000 model and experimental model (Rinanldin, 2013)**

Modal analysis was also performed on model (b) and the first two vibration periods are to be 0.19 sec and 0.16 sec, which matches well with the experimental first period of 0.20 sec. Then the model was subjected to the Northridge accelerogram. The base shear vs. story drift simulated from the SAP2000 model and from the experiment are plotted in the same as shown in Figure 2-36, where good agreement is observed.

![Graph of base shear vs drift](image)

**Figure 2-36: Comparison of Base shear vs displacement in SAP2000 (Rinanldin, 2013)**

The analysis results of these two models show that the building modeling method using diagonal springs representing shear walls is able to predict the building dynamic behavior. Such modeling method, therefore, is considered to be a simplified method using SAP2000.
approximation, (i.e., assume pivot hysteresis of the spring elements) and the difference in the building configurations of the experimental and the numerical models are the reasons for the difference of the responses as shown in the comparison plots.

2.4 Modelling methods of wooden frame structures under wind loading

Disasters due to extreme wind load cause destruction of the built environment and severely impact on the local communities and their economy. According to the Saffir- Simpson scale, wind speed greater than 75mph with a duration up to one minute is classified as hurricane. Between 2000 and 2009, south eastern and Gulf Coastlines of the United states have been struck by 19 hurricanes (National Oceanic and Atmospheric Administration, 2012). Unfortunately, larger population of these regions have wood-frame construction consisting of several conventional materials and framing techniques. To ensure the least damage to the physical buildings as well as human lives, improved and properly engineered wooden-frame structures are needed. Although different building design codes have been developed, there is a strong need to continuously improve the performance of the wooden frame structures under severe wind conditions and to update design codes and procedures considering several wind loading conditions.

Wood light frame structures are assembly of several components which includes walls, floors, roof systems and connectors among these components. Design guides for light frame wooden construction generally specify the design procedures for lateral load resisting systems as the force induced by wind loads on the structure needs to be distributed to and resisted by the lateral force resisting system. Inappropriate assumption on the load distribution during analysis may lead to inappropriate design and unsafe wooden frame structures. Thus, proper load distribution within a building structure should be predicted to create confidence in the performance and safety of the structure. Modelling methods of wooden frame structures under wind loading conditions are, therefore, developed in the last decade with the primary objectives of: (1) determining the effects of the environment loads (specifically wind loads) on the structural performance; (2) determining the load path of the wooden frame structures. Commercial finite element (FE) analysis tools such as ANSYS, SAP2000, ADINA, NASTRAN/PASTRAN are used in these research projects to achieve the objectives.
2.4.1 Modelling method using ANSYS

Kasal et al. (2004) utilized ANSYS to develop an FE model (see Figure 2-37) to improve the design procedure of the light frame wood structure through better prediction on wind load distribution within the building model. The wall framing members were modelled as elastic beam elements; the sheathings were represented using a plate element. The connections between the components (i.e., frame and sheathing) were modelled using nonlinear springs with hysteretic or non-degrading behavior. The shear wall of the frame structure shown in Error! Reference source not found. was also modelled using the nonlinear springs, which is energetically equivalent to the shear wall modeled using a detailed FE model. The components of the FE model of the shear walls included non-linear hysteretic elements for nails and shell elements for studs and sheathing. The stiffness of the spring is determined from the experimental data of the wall (Kasal et al., 2004). The lateral load distribution was determined using eight analytical and FE methods, including tributary area, continuous and simple beam, total shear, relative stiffness, rigid beam or elastic foundation, plate method and the 3D FE method. In addition, the experimental result of an L-shaped test model was compared to the load distribution estimation using the four analytical methods (i.e., rigid beam, plate method with rigid diaphragm, plate method with flexible diaphragm, and FE model methods). The FE model was found to be able to accurately predict the experimentally measured load distribution.

![Figure 2-37: Finite Element of the wooden frame structure (Kasal et al., 2004)](image)
He et al. (2018) developed a wooden frame model using ANSYS to capture the non-linear behavior of the building structure under time history wind pressures measured from the wind tunnel tests conducted at the Florida International University. Figure 2-38 shows the FE model of the wooden frame structure developed. The framing and the truss members are modeled using the Beam element. The sheathing panels were modeled using the shell elements. A standard zero mass nonlinear spring elements were used to modelled the connections of roof to wall and sheathing to frame. A rigid connection was used as the support to the frame model at the base.

![Finite element model developed by He et al. (2018)](image)

**Figure 2-38: Finite element model developed by He et al. (2018)**

(a) ![Deflection of roof to wall connection](image)

(b) ![Numerical vs experimental time history domain prediction](image)

**Figure 2-39: a)Deflection of roof to wall connection (He et al.,2018) b) Numerical vs experimental time history domain prediction (He et al., 2018)**
For the validation purpose, comparisons of the average deflection and deflection time histories of the roof sheathing panels and roof to wall connection were made showing a strong agreement between the FE model estimation and the experimental measurements. Figure 2-39 (a) shows an example comparison of case 3 with a wind speed of 65mph and the structure model without opening. Similarly, for the deflection time history predicted by the numerical model matches very well with the experiment measurement as shown in Figure 2-39 (b) for case 2, of which the wind speed was 65mph and the model had openings.

More recently, Quayyum (2019) developed a wood frame house model using ANSYS to study the influence of roof, side and partition walls’ base connection on the in-plane load resistance of the wall under the in-plane lateral loads. This model included all the connection details in the building structure as shown in Figure 2-40 (a and b).

![Finite element model of the structure](image)

**Figure 2-40:** (a) Finite element model of the structure; (b) details of components of the house

Similarly, to the previous models using ANSYS software, the beam element was used to model the framing members (vertical studs, horizontal plates and roof trusses). The sheathing component are modelled using the shell elements. The concrete foundation of the building is modelled using the beam element whereas the bottom plate of the wooden wall was connected to foundation using the non-linear spring element as shown in Figure 2-40 (b).
The validation of the model was achieved by 1) the comparison of stand-alone wall of the house with the experimental response under in-plane lateral loads and, 2) full-house model’s wind load damage against the observations from the field and literatures. The damage of a full-house was observed after the Tornado loads (EF5 tornado at Parkersburg, Iowa) occurred on May 25, 2008. Also, system level wall responses were compared to the experimental responses under lateral loads. Figure 2-41 Error! Reference source not found. shows the experimental and simulation model comparison of standalone wall with the field observations and responses from previous experiments. The plots show good agreement between the experimental results and the finite element model developed.

Figure 2-41: Displacement vs. lateral loads plot for comparison of experimental response and simulation (a) from Quayyum (2019); and (b) from Doudak et al. (2006), Dolan and Heine (1997)

Wind load damage of the full house model subjected to the tornado loads was compared to the results from previous literatures and the system-level wall responses were compared with experimental responses. The model results of tornado under different location was in good agreement with the failure and damage mechanism. The stress distribution in the house with
respect to location of the house were compared with the field observation. The location of the tornado was defined by the function of distance between the tornado core and the center of the house \((x)\) and radius of tornado core \((r_c)\) as shown in Figure 2-42.

As a representative, the tornado location is taken as \(x=-r_c\) (see Figure 2-43). The failure of the roof shows good agreement with the field observation (see Figure 2-43).

Figure 2-42: Location of the tornado core with respect to center of the house (Quayyum, 2019)

Figure 2-43: (a) Result of finite element model (b) Field observation (Quayyum, 2019)
To validate the model’s ability to yield system level response, the resistance of the shear walls when subject to point lateral loads was obtained and was compared to the experimental results. Error! Reference source not found. (Figure 2-44 b) shows the comparison plot of the force-displacement response of the walls to the in-plane lateral loads with experiment data and a good agreement is observed.

2.4.2 Modelling method using SAP2000

Zisis (2006) developed a typical wood frame building model to determine the effects of the environmental loads on buildings using SAP2000. Basically, the study was divided into three parts. In the first part, the pressure and force coefficients were computed based on the data obtained from the pressure and load monitoring instrument installed on the full-scale test building. A wind tunnel experiment of the test building was conducted in the second part and the pressure results were transformed into the mean and peak local and the area-averaged pressure coefficients. An FE analysis of the 3-D linear model (see Figure 2-45 Error! Reference source not found.) was created in the third part and this model was subjected to the selected wind direction.
The building model consists of the framing element modelled as the built-in SAP linear frame element. Similar to the molding methods developed by Martin (2010), a membrane type shell element was used to model the sheathing elements. The foundation was restrained along the three directions (i.e., x, y and z). The validation of this FE model was done by comparing the force coefficients between the measured ones using the load cell installed on the test model (see load cells arrangement in Error! Reference source not found. (Figure 2-46 a) and the FE predicted coefficients. Area loads were applied to the building model that were converted from the point reading using the area averaged method for each of the 15 examined directions measured during the wind tunnel experiment. The variation of the mean and peak pressure coefficient of the wind tunnel test as a function of the pressure coefficient is plotted and compared to the prediction from the FE model analysis. NW1 pressure tap is taken as an example shown in Error! Reference source not found. (Figure 2-46 b), which shows a good agreement.
Martin (2010) developed an analytical model of the 3D wood building to evaluate the structural system response due to different wind load scenarios and to define the load path of the structure. The author used simplified modelling techniques and material definition for the analysis using SAP2000. A 3D building model with a gable roof using the Fink trusses was developed. The model was linear with all the joints either being pinned or rigid (see Figure 2-47 Error! Reference source not found.). The main characteristic of this model is that instead of modelling individual nail in
the wall system, a single sheathing element was used to represent the entire wall on each side of the building. Also, the property modifier was carried out based on the nailing schedule using the correlation procedure derived using the NDS 3-term shear wall equation. The computer model developed was validated against the experimental studies.

![Figure 2-47: SAP model of the building (Martin, 2010)](image)

The model developed was exposed to several loading scenarios which included uniform uplift pressure applied to the roof, simulated hurricane events and the wind load calculated based on ASCE 7-05. The model developed was able to predict the behavior of the complex, 3D wood-framed structure as discussed in detail below.

a. Uniform Uplift Pressure

Uniform uplift pressure of 50psf was applied to the roof the building shown in Figure 2-47. The reaction profile was developed for the hold down and anchor bolts, where symmetric response is observed. The anchor bolts directly below the ridge line on the end walls showed a spike representing that the load accumulated at roof is transferred to the anchor bolts directly below the ridge line.
Similarly, the reaction profile for the side walls showed parabolic profile with the anchor bolts near the mid building with the highest reaction among all the hold down and anchor bolts as shown in Figure 2-49. The results show that the anchor bolts located in the side walls carries more load than the ones located in the end-wall on the structural configuration defined in this section.
b. Simulated Hurricane uplift pressure

The pressures by simulated hurricanes were determined from the wind tunnel study on the 1:50 scaled model. The reaction profiles of the wind walls showed that the windward wall experiences more uplift than the leeward sidewall due to racking force acting towards the wall (see Figure 2-50Error! Reference source not found.). These racking forces are developed due to unbalanced horizontal component of the uplift pressures oriented normal to the sloped surface of the roof.
c. ASCE 7-05 Pressures

Three different scenarios were considered following the ASCE 7-05 pressure condition. The vertical component acting alone on the plane of roof, horizontal component acting alone and resultant of both components acting. The reaction profile under the vertical component acting alone (i.e., the uplift force) is shown in Figure 2-51 Error! Reference source not found., which
is similar to the profile under the uniform uplift pressure with few differences due to different magnitude of the load applied.

(a) Figure 2-51: Reaction profiles due to ASCE 7-05 Pressures, uplifts loads acting alone (a) End walls; (b) Side walls (Martin, 2010)

(b) Figure 2-52: Reaction profile for the gable walls with ASCE-07 lateral loads only (Martin 2010)

The second scenario considers horizontal components (i.e., lateral loads). The reactions of the gable wall is plotted as shown in Figure 2-52. For the windward, the reaction profile is mirrored about its vertical and horizontal axis due to the symmetry of the building.
The third load condition is the lateral plus the uplift loading. The reaction profiles show that the uplift reaction is the algebraic sum of the forces induced by individual anchorage acting alone as shown in Figure 2-53 (a and b) Error! Reference source not found..

![Reaction profile for gable walls (ends)](image1)

![Reaction profile for eave walls (sides)](image2)

**Figure 2-53: Reaction profile due to ASCE 7-05 lateral plus uplift loads for (a) end walls and; (b) side walls (Martin, 2010)**

Besides determining the load path, the principle of the correlation procedure and the modelling technique was developed. The shear wall deflection for the given load value predicted by the National Design Specification (NDS) for wood construction was compared with the deflection from the SAP model. The NDS has shear modulus tabulated for different edge nailing schedule. Then the analytical model is matched with the computed deflection based on NDS by changing the value of the shear modulus. The correlation procedure is completed when the shear modules in SAP is found to give the same deflection as predicted by NDS. The developed correlation procedure reduced the effort for modelling individual nails for connecting elements.

Pfretzchner et al. (2012) developed a practical modelling method based on the model created by Martin (2010) for the wood building structure having complex geometry (i.e., L-shaped). A linear model was developed using SAP2000 (see Figure 2-54 Error! Reference source not found.) to determine the load path and investigate the behavior of the complex wooden light frame structure system.
The framing and the truss members were modelled using the linear frame element. The oriented strand board (OSB) sheathing walls were modelled using the shell elements. The connection between the framing members (horizontal plates and vertical studs) is modelled as pinned connection. The procedure of adjusting the shear modulus of the shell elements was adopted, which considers the effects of edge nail spacing developed by Martin et al. (2010). The hold downs and anchor bolts of the building model were represented using one spring oriented in the Z-direction and three springs oriented in the X, Y and Z directions, respectively. The validation of this linear model was done against the full-scale test results. Also, several sub-assembly’s models such as the two-dimensional trusses, three-dimensional roof assemblies and shear walls were validated based on the results from previous research. A uniform uplift pressure of 50psf and the ASCE 7-05 design wind loads were the loading cases considered. The vertical reactions and changes in reaction at the anchor bolts and hold-downs of the two index buildings (see Figure 2-55) under the uniform uplift pressure were plotted.

Figure 2-55: Uplift reactions for (a) rectangular and; (b) L-shaped index building (Pfretzschner et al., 2012)
Figure 2-55 Error! Reference source not found. shows the bubble plots of the reaction load obtained of the building models under the ASCE wind loading from the SAP2000. The maximum reaction occurs at the center of the side walls for the rectangular building whereas, the maximum reaction occurs at the hold down located at the re-entrant corner of the L-shaped index building. The anchor bolts directly opposite to the re-entrant corner experience significant reactions. In addition, it can be noted that the truss orientation and the re-entrant corner does not effect on the reaction on the wall-5 which proves that the truss orientation with respect to walls have little to none effects on the load distribution.

![Diagram of building models with reaction loads](image)

**Figure 2-56: Uplift reactions in L-shaped index house under ASCE 7-05 design wind loads** *(Pfretzschner et al., 2012)*

The uplift reactions of the anchor bolts and hold-downs in the L-shaped index house for the wind load cases can be seen on Figure 2-57. Four load cases West-End, North-South, Southeast-Northwest, East-west wind loads were examined. North-south wind loads are taken as example to exhibit lateral load distribution on the structure. It is observed that the percentage of the load carried by the central walls at the center (i.e. 6, 7 and 8) increased by 14% whereas the end wall’s (i.e., 6, 7 and 8) decreased by 7 and 3%. The increase and decrease in the load carrying capacity has been due to the extension of walls 7 and 9 (see Figure 2-56 Error! Reference source not found.) which caused redistribution of the loads to inner N-S walls (i.e., 6 and 8).
Figure 2-57: Lateral load distribution to walls parallel to wind loads for the reentrant corner variation (Pfretzschner et al., 2012)

A comparison of the load path and structural behavior between the timber frame (TF) and the light frame (LF) was made by Malone et al. (2014). Practical analysis model for both structure types were created using SAP2000. The timber frame structure was composed of large-dimension timbers and structural insulated panels, while the light frame structure was composed following the guidelines of the International Residential code.

Figure 2-58: (a) Timber frame and (b) Light frame structures (Malone et al., 2014)

The modelling method using SAP2000 adopted by Martin (2010) and Pfretzschner et al. (2012) was followed to model the two structural frames (see Figure 2-59), where the framing members were modeled using frame elements and the sheathing members were modeled using the layered shell elements available in the SAP2000. The connection between the framing members was modelled as a hinged connection with the moment released in all directions. The stiffness of the sheathing elements was adjusted using the correlation procedure of edge nail spacing developed
by Martin (2010). Anchor bolts and hold downs were modeled as the foundation using the directional linear spring elements.

![Figure 2-59: SAP model of the light frame and the timber frame (Malone et al., 2014)](image)

These two SAP2000 frame models were loaded with the standard dead loads (using material properties), live and wind loads according to ASCE 7-10 and the snow loads were calculated based on its location of Jay, Vermont. The allowable stress design (ASD) load combination 4, 5, 6a (ASCE 2010) were applied to the structure (see Table 2-4), where, D=dead load, L=live load, S=snow load, W=Wind load.

<table>
<thead>
<tr>
<th>Load Combination</th>
<th>Equation</th>
</tr>
</thead>
<tbody>
<tr>
<td>4</td>
<td>D+0.75L+0.75S</td>
</tr>
<tr>
<td>5</td>
<td>D+0.6W</td>
</tr>
<tr>
<td>6a</td>
<td>D+0.75L+0.75(0.6W)+0.75S</td>
</tr>
</tbody>
</table>

The performance levels of the two models developed were compared on the basis of the resistance, story drift, large openings, break in the load path and the range of axial loading in vertical members due to the uplift pressure due to wind. For the uplift pressure due to wind only case (i.e., load combination 5), the foundation reactions of the timber frame and light frame are represented using the bubble diagram as shown in Figure 2-60. The dark bubbles indicate downward force at the foundation connections which induces upward reactions; whereas the light bubbles indicate the upward force inducing downward reaction thus introducing uplift.
The results show that the light frame structure has more uplift in the windward side than that of the timber frame which has uniform positive foundation reaction throughout, showing that the timber frame has less amount of uplift comparing with light frame. The story drift the deflection under the north-south wind loading and load combination 5 of both the structures are shown in Error! Reference source not found.. The story drift of the timber frame is about 3.5 times less than that of the light frame at the center of the north wall at the top of the first floor. The deflections at the floor level of the second story was also measured as an index of the gable end wall stiffness. The deflection of the gable end with garage door is 1.5 times more than that of the deflection without openings, proving that the timber frame is more resistant to opening introduced in shear walls with respect to lateral stiffness.
Figure 2-62 shows the deformed shape of the gable end and side of the building models to understand the effects of the openings on the responses under load combination 4. The vertical displacement for the gable end and side of the timber frame are less than that of light frame.

![Figure 2-62: (a) Deflection of light frame and; (b) timber frame subjected to gravity loading and load combination 4 (Malone, 2014)](image)

Another comparison factor is the range of axial loading on the vertical members. Maximum axial loads and the average axial loads were determined with the application of load combination 4. The timber frame’s vertical member possessed the maximum axial loads than that light frame vertical member. The deflection after the removal of the central post in the first floor in both timber and light frame structures were determined using the SAP2000 models and the deflection shapes are shown in Error! Reference source not found.. It was found out that the deflection of the timber frame is 25% of the light frame floor system demonstrating that the timber frame is comparatively less affected by the break in the load path than the light frame.
Comparing all the responses between the timber frame and the light frame, it is determined that the light frame was unable to resist uplift force and inter story drift. Also, the timber frame is more resistive to the break in load path due to the introduction of openings or the removal of central post when compared to light frame structure.

2.4.3 Modelling using other FE tools

Datin (2010) developed a modelling method to advance fundamental understanding of structural load paths in a simple gable roofed residential light frame wooden structures subjected to wind loads. A general FE analysis computer program named ADINA (Automatic Dynamic Incremental Nonlinear Analysis) was used to analyze the roof truss model as shown in Figure 2-64.
This roof truss model uses beam elements for the truss members. The roof sheathing is modeled by the shell elements. The inter-component connection is assumed to be a rigid link which is a built-in function of the computer program. The dynamic load path was expressed using the impulse and frequency response function. The validation of the model was done in a step by step manner.

First step included validation of the single truss model and compared with the previous experimental work. The vertical deflection of the top and bottom five chords were compared with the experimental results shows good agreement for both loading conditions (i.e., 55lb/ft and 66lb/ft) for the truss configuration shown in Figure 2-65.

![Figure 2-65: Single truss validation (Datin, 2010)](image)

The second step validation is the single truss influence lines for the vertical reactions at the heel joints in the 4 in 12 sloped roof with an 18-inch overhang on both the sides. The influence line generated where compared to the theoretical influence line developed which both were same as shown in Figure 2-66.
The final validation step is based on the influence line coefficient of the truss reaction of the 21-truss roof model under the point load. The influence line coefficient for the truss reaction was compared with the experimental result where good agreement is obtained.

Frequency analysis validation of the FE model developed by ADINA was conducted and validated by comparing to fundamental frequency calculated based on the relation proposed by Tedesco et al. (1999):

\[
W_n = \left( \frac{n \pi}{L} \right)^2 \frac{EI}{\rho A}
\]

(1)

where, \(W_n\) = fundamental frequency of the n-th mode

L = Span

E = Modulus of elasticity

I = moment of inertia

\(\rho\) = density of the material

A = Cross sectional area of the beam

Frequency analysis of the roof system was first conducted by modeling the 8ft long 2x4 member using the beam elements. The fundamental frequency was obtained from the analysis using ADINA and calculated theoretically based on equation (1). Table 2-5 lists the theoretical
frequencies and the ADINA frequencies whose values are very close to each other. Thus the capability of the ADINA program of performing fundamental frequency analysis is validated.

<table>
<thead>
<tr>
<th>Mode</th>
<th>Theoretical Frequency (Hz)</th>
<th>ADINA Frequency (Hz)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>33.95</td>
<td>33.93</td>
</tr>
<tr>
<td>2</td>
<td>135.8</td>
<td>135.5</td>
</tr>
<tr>
<td>3</td>
<td>305.5</td>
<td>304.0</td>
</tr>
</tbody>
</table>

Wang (2013) developed a structural model using an FE program named NASTRAN (see Figure 2-67) together with PASTRAN for pre- and post-processing data. The objective of the research was to quantify the uncertainties in the load paths and its effects on the reliability of wood structural members under uplift pressure created by the hurricane winds. The wall framing members were modelled using the beam element, the sheathing panel for the truss were modelled using the shell element while the wall sheathing panels were not included in the model to reduce the computational effort. Similarly, nail for the connection between the sheathing and the top chord in the roof was modelled using the beam element.

![Figure 2-67: Wooden frame structure developed by Wang et al. (2019)](image)

The experimental model is a single storied slab on grade structure with the hip roofed configuration. The test model was installed with the load cells denoted by “L” between the truss
reactions and the supporting wall (see Figure 2-68). In addition, a total of 76 pressure taps are attached on the roof surface as denoted by “P”.

![Figure 2-68: Load cell configuration for test structure (Wang, 2013)](image)

The FE model and the test model were subjected to dead and wind loads, and the trusses reactions were determined. The comparison between the truss reaction measured from the test model and simulated using the numerical model was done to validate the model, as shown in Figure 2-69, where discrepancies is observed. These discrepancies maybe due to the member curvatures, material stiffness variations, roof weight variations, structural geometry variations, construction error, settlement of foundations and instrument errors. In order to create good agreement between the FE analysis and experimental results, variations of the material stiffness, material density was analyzed in the FE model, but little or no effect on the reaction responses was observed. Thus, the stud length variation was introduced in the FE model to match up with the construction error. The length variation was created by imposing small downward displacement of 0.02 inches on the stud under the load cells S8, S11 and S14. Figure 2-69 (b) shows the adjusted FE model achieved by imposing the small downward and better agreement is obtained.
Similarly to the dead load analysis, adjustments were made for the wind load cases and the results are shown in Figure 2-70 (a and b), where again better agreement between the FE model and the test model is obtained after adjusting the FE model to consider construction error in the studs.
Chapter 2 discusses about the different tools that have been developed to model wooden frame structures subject to environmental loadings such as earthquake and wind. The purpose of the numerical modelling development has been varied from the study focused on the component level behavior to the whole structural system-level performance. Computer analysis tools initiated during the CUREE-Caltech project with the primary objective of investigating seismic performance and implementing engineering features of the wooden frame construction. Different tools have been developed, thereafter, for research purposes for wood frame analysis under seismic as well as wind loadings. Parametric hysteretic models for sheathing to framing connectors of shear walls were developed in the Cyclic Analysis of the wood shear walls (CASHEW), Evolutionary Parametric Hysteretic Model (EPHMS). Likewise, seismic Analysis package for wood frame structures (SAPWood), Matlab-Cyclic Analysis of wooden shear walls-2(M-CASHEW2), LightFrame3D were developed with the objective of the predicting dynamic response of the whole wooden frame building structure. The modelling complexity demanded the need in the development of finite element modelling method for wooden frame structures. For example, the finite element model was incorporated into a computer program called Seismic Analysis of Wooden frame structures (SAWS). Some of the commercial tools that are used for the development of wooden frame model were also identified, including ABAQUS/CAE, SAP2000, ANSYS, ADINA (Automatic Dynamic Incremental Nonlinear Analysis), NASTRAN/PASTRAN. These software tools helped to develop wooden frame model using their built-in elements to represent framing, sheathing and inter-components connections. The objectives of wooden frame modelling methods using these tools were generally to predict the responses of the structures and load distribution under seismic and wind loadings.

Among these modeling tools researched and presented in this chapter, the commercial tools are deemed to be efficient for the general purpose in wood structure analysis. For example, SAP2000 is an efficient and effective tool for developing models of wooden frame structure and analyzing its behavior under different loading conditions. The versatile nature of the SAP2000 analysis capability and the elements available in its library makes SAP2000 a preferred tool and was selected in modeling shear wall and building structures for seismic and wind loading conditions, as presented in detail in the next chapter.
CHAPTER 3: MODELLING PROCEDURES

3.1 Introduction

SAP2000 is one of the two commercial tools discussed in section 2.3 that is widely used in wood building structure and assembly modeling due to its versatility and the various linear and nonlinear components. This chapter will describe the modelling procedure of a wooden frame structure using SAP2000 including the different element that are adopted to model the different structural component. Two models are setup in SAP2000 and are described in this chapter. The first model is a linear 3D wooden frame building subject to a uniform uplift pressure applied to the roof presenting typical wind load condition. The second model is nonlinear wooden frame shear wall assembly subject to the cyclic horizontal displacement loadings representing the seismic load condition. The detailed modelling process of these two models is presented in Appendix of the report.

3.2 Modelling of the 3-D wooden frame building

The wooden frame model as shown in Figure 3-1 is setup in SAP2000 according to the procedure presented by Martin (2013). As mentioned in Chapter 2, this model was established to evaluate different wind loading scenarios on the system-level load distribution and to identify the load paths within the building structure. Table 3-1 provides an overview of the material and the elements used in this SAP2000 wood building model.

Figure 3-1: SAP model of the test structure (Martin, 2010)
### Table 3-1: Elements and Material Overview

<table>
<thead>
<tr>
<th>Members</th>
<th>SAP Element</th>
<th>Materials Property</th>
<th>Dimensions</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Framing</strong></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Corner Studs (x4)</td>
<td>Frame Element</td>
<td>Isotropic, Elastic</td>
<td>Two 2×4”</td>
</tr>
<tr>
<td>Exterior Studs</td>
<td></td>
<td></td>
<td>2×4”</td>
</tr>
<tr>
<td>Truss</td>
<td></td>
<td></td>
<td>2×4”</td>
</tr>
<tr>
<td><strong>Sheathing</strong></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Wall Sheathing</td>
<td>Shell Elements</td>
<td>Orthographic, Elastic</td>
<td>7/16”</td>
</tr>
<tr>
<td>Roof Sheathing</td>
<td></td>
<td></td>
<td>½”</td>
</tr>
<tr>
<td><strong>Hold Downs</strong></td>
<td>One grounded Spring Elements</td>
<td>-</td>
<td>Stiffness in Z-direction only</td>
</tr>
<tr>
<td><strong>Anchor Bolts</strong></td>
<td>Three grounded Spring Elements</td>
<td>-</td>
<td>Stiffness in X,Y and Z direction</td>
</tr>
</tbody>
</table>

The plan dimension of the building is 30ft x 40ft with overhangs on the side of the roof having a slope of 4:12. The spacing between the studs is 16-inches on center and the roof trusses are 24 inches’ center to center. The building does not have interior partition wall. Figure 3-2 shows the SAP model developed with various elements listed in Table 3-1. The modelling process using different elements is described briefly next.

![Figure 3-2: A complete 3D model in SAP](image-url)
3.2.1 Framing Elements

The materials used for framing elements are elastic isotropic. The properties of these materials are determined based on the NDS code (AF & PA 2005a) and the wood handbook (USDA 1999) and listed in Table 3-2.

<table>
<thead>
<tr>
<th>Item</th>
<th>Wall Member</th>
<th>Truss Member</th>
</tr>
</thead>
<tbody>
<tr>
<td>Description</td>
<td>SPF, stud grade</td>
<td>SYP, No.3 and stud</td>
</tr>
<tr>
<td>MOE (10^6 psi)</td>
<td>1.2</td>
<td>1.4</td>
</tr>
<tr>
<td>Poisson's Ratio</td>
<td>0.4</td>
<td>0.36</td>
</tr>
<tr>
<td>Specific Gravity</td>
<td>0.42</td>
<td>0.55</td>
</tr>
<tr>
<td>Density (lb/in^3)</td>
<td>0.01512</td>
<td>0.0198</td>
</tr>
<tr>
<td>Specific Weight (lbf/in^3)</td>
<td>5.84</td>
<td>7.64</td>
</tr>
</tbody>
</table>

The stud to the plate connection and the plate to plate connection of the frame elements are pinned. The connection within the wall framing members can be seen in the figure below.

Figure 3-3: Connection Between the Wall Framing Members (Martin, 2010)

Figure 3-4: Pinned Connection Between The wall framing Members in SAP2000
3.2.2 Sheathing Elements

There are three options to model area objects in SAP2000, as listed below:

1. Membrane element: consider in-plane forces.

2. Plate element: opposite to the membrane behavior and considers out-of-plane bending and transverse shear.

3. Shell element: considers both in-plane and out of plane behavior. Further, two types of shear elements are available based on the transverse shearing deformation.

As listed in Table 3-1, the thick shell element is selected for the sheathing members of the wood building structure to capture the full shell behavior. Nine material properties are needed for the sheathing elements but only four constants are utilized in this model (Martin 2010) and they are listed in the table below.

<table>
<thead>
<tr>
<th>Properties</th>
<th>Oriented strand board (OSB) 7/16&quot;</th>
<th>Plywood</th>
</tr>
</thead>
<tbody>
<tr>
<td>Specific Gravity</td>
<td>0.62</td>
<td>0.57</td>
</tr>
<tr>
<td>Density (lb/in³)</td>
<td>0.02232</td>
<td>0.02052</td>
</tr>
<tr>
<td>Specific Weight(lbf/in³)</td>
<td>8.62</td>
<td>7.92</td>
</tr>
<tr>
<td>Modulus of Elasticity(psi),E₁</td>
<td>7.4</td>
<td>19</td>
</tr>
<tr>
<td>Modulus of Elasticity(psi),E₂</td>
<td>2.3</td>
<td>2.9</td>
</tr>
<tr>
<td>Shear Modulus (psi), G₁₂</td>
<td>1.2</td>
<td>1.5</td>
</tr>
<tr>
<td>Poisson's Ratio, μ₁₂</td>
<td>0.08</td>
<td>0.08</td>
</tr>
</tbody>
</table>

SAP2000 has a feature to perform automatic meshing for the sheathing elements. The shell element is meshed using the automatic meshing function based on either the Maximum size and the General divide tool. Points along each edge at equal interval of the shell elements is added during the meshing process. For the sheathing member of the wall to properly interact with the framing members, the mesh size is assigned as a multiple of the frame spacing. In the presence of the odd length such as the end wall or the wall below the ridge line, the “General divide” option is applied.

3.2.3 Hold-Downs and Anchor Bolts

The hold-downs are modeled as one grounded spring with the stiffness in the axial direction (i.e., Z-direction). The anchor bolts are modeled as three grounded springs with the stiffness along
shear forces parallel and perpendicular to the wall (i.e., X and Y-direction) and axial direction (Z-direction). The orientation of the anchor bolts and hold down devices is shown in Figure 3-5.

Table 3-4 lists the stiffness values for the hold downs and the anchor bolts.

Figure 3-5: Hold downs and Anchor bolts configuration (Martin, 2010)

Table 3-4: Stiffness of Hold-down and Anchor bolts (Martin,2010)

<table>
<thead>
<tr>
<th>Item</th>
<th>Hold Down</th>
<th>Anchor Bolts</th>
</tr>
</thead>
<tbody>
<tr>
<td>X-Direction (lb/in) shear</td>
<td>-</td>
<td>65000</td>
</tr>
<tr>
<td>Y-Direction (lb/in) shear</td>
<td>-</td>
<td>65000</td>
</tr>
<tr>
<td>Z-Direction (lb/in) axial</td>
<td>35000</td>
<td>35000</td>
</tr>
</tbody>
</table>

3.2.1 Application of Loads

A uniform uplift pressure of 50psf (0.35psi) is applied to the roof sheathing panels. The reactions of the hold downs and anchor bolts to the uniform load are obtained through static analysis and the results are analyzed.

Figure 3-6: Application of uniform uplift pressure of (0.35psi)
3.2.2 Results and Discussion

The reaction profile of the hold downs and the anchor bolts are plotted (see Figure 3-7) after the application of the 50psf (0.35 psi) loads normal to the surface of the roof. As can be seen, symmetric response of the building to the uplift loading is achieved. The end walls show spike directly below the ridge line, which is expected. The holds downs show the maximum reaction at both ends of the wall. Similarly, the side wall reaction profile is shown Figure 3-8 where the reactions of side wall are quite uniform with the slightly higher value at the mid wall.

![Reaction Profile for the end walls](image1)

**Figure 3-7: Reaction profile for end walls**

![Reaction Profiles for the side walls](image2)

**Figure 3-8: Reaction profile for side walls**
Similarly, the equivalent stress or also known as “von Mises stress” of the end walls, side walls are plotted using the SAP functions and are shown in Figure 3-9 and Figure 3-10, respectively. The stress distribution on the end wall shows a concentration directly below the ridge line, which is expected and validate the building model created.
3.2.3 Conclusion

The modelling procedure of a 3D linear wood building model is briefly discussed and the building model is subject to a uniform uplift wind pressure load to obtain structural responses. Comparing the results obtained from this study and the ones from the reference, similar responses and structural behavior are obtained. Therefore, the modeling method using the commercial software is validated. The model developed herein can be used for numerical simulation of the wood building structures subject to various wind loading analysis scenarios in future research. Although the current model is linear, nonlinear behavior can be easily incorporated into this whole building model which will greatly expand this model’s capability for capturing large responses under more severe wind loading conditions.

3.3 Modelling of the Wooden Frame Shear wall

The modelling of a wooden frame shear wall assembly is carried out according to a research work presented by Rinaldin et al. (2013). This model captures the nonlinear cyclic behavior of the light-frame timber shear wall, including both stiffness and strength degradation, post-peak softening branch, and pinching effect. The nonlinear behavior of the nailed connections within the shear wall model is modeled using two multi-linear link elements in SAP2000. Although SAP2000 does not provide a specific hysteretic model for nail connections of wood structures, among different models that are available, the Pivot hysteresis model fits the best and is selected that accounts for both the pinching and the stiffness degradation. The cyclic displacement time history is imposed at the top left corner of the shear wall model that directly mimic the traditional shear wall cyclic loading experiment according to the CUREE Standard Protocol (see Figure 3-12).

3.3.1 Modeling of the Wooden Frame Shear Wall Assembly

The light-frame timber wall of 2.4m × 2.4m tested by [Dolan, 1989] is modelled. The connection between wood frame to the sheathing is represented using a non-linear spring with hysteretic shear-displacement cycle along the two perpendicular directions of the wall plane. To capture the first branch of the back-bone curve, the following data were used for the shear wall model [Rinaldin et al. 2013]. In addition, element types adopted for the different structural components of the shear wall are illustrated in Figure 3-11 and their average elastic modulus values are listed Table 3-6. For the nonlinear spring element used to model a single nail connection, the hysteresis model (i.e., the backbone curve [Fischer et al., 2001]) is input point by point to form the multi-linear force-deformation.
Table 3-5: Material Properties of the Shear Wall Assembly

<table>
<thead>
<tr>
<th>Properties</th>
<th>Values</th>
</tr>
</thead>
<tbody>
<tr>
<td>Average density of timber frame, $\rho_{\text{m,stud}}$</td>
<td>420 kg/m$^3$</td>
</tr>
<tr>
<td>Characteristics density of timber frame, $\rho_{\text{k,stud}}$</td>
<td>350 kg/m$^3$</td>
</tr>
<tr>
<td>Average density of sheathing panels, $\rho_{\text{m,pan}}$</td>
<td>630 kg/m$^3$</td>
</tr>
<tr>
<td>Thickness of the sheathing panel, $t_{\text{pan}}$</td>
<td>9.5 mm</td>
</tr>
<tr>
<td>Nail diameter, $\varnothing_{\text{nail}}$</td>
<td>3.25 mm</td>
</tr>
<tr>
<td>Nail length, $L_{\text{nail}}$</td>
<td>76.2 mm</td>
</tr>
</tbody>
</table>

Table 3-6: Model Elements and Materials

<table>
<thead>
<tr>
<th>Elements</th>
<th>SAP elements</th>
<th>Material</th>
<th>Dimensions</th>
</tr>
</thead>
<tbody>
<tr>
<td>Framing Members</td>
<td>Standard beam</td>
<td>Spruce-Pine-Fir (SPF)</td>
<td>38x89mm</td>
</tr>
<tr>
<td>Sheathing Members</td>
<td>Shell element</td>
<td>Plywood</td>
<td>9.5 mm thick</td>
</tr>
<tr>
<td>Nail</td>
<td>2 DOF Non-linear Spring</td>
<td>Hot dipped galvanized common nails</td>
<td>63.5mm</td>
</tr>
</tbody>
</table>

Figure 3-11: Layout of the light-frame timber walls (Rinaldin et al., 2013)
The framing member of the shear wall is modeled as the standard beam element with the average modulus of elasticity of 8400 N/mm$^2$ and these frame elements are pin connected. The frame is simply supported at the base. The sheathing member of the wall is the standard thin shell element with the average modulus of elasticity of 3000 N/mm$^2$. The connections between the panel and the frame are modeled using two non-linear springs along the two in-plane direction of the wall whose values are calibrated to the strength and stiffness based on the nails used. The shear wall model is illustrated in .

---

**Index:**

- Simply supported at the bottom
- Restrained at the left top node
- Link Elements

---

**Figure 3-12:** CUREE cyclic Loading Protocol (Rinaldin et al., 2013)

**Figure 3-13:** Detailed shear wall configuration in SAP2000
Results and discussion

After the application of the cyclic horizontal displacement history, the model was analyzed using the time history analysis. The deformed shape of the model developed in the model and the deformed shape from the reference (Rinaldin et al., 2013) are comparatively shown in Figure 3-14, which show similar shape. The force vs. displacement obtained from the analysis and from the reference is shown in Figure 3-15.

Figure 3-14: (a) Deformed Shape of the shear wall model obtained from analysis; and (b) original paper
The results obtained from the analysis shows close agreement with the results obtained from the reference. As mentioned by Rinaldin et al. (2013), the accuracy of the model is due to the pivot hysteresis’s ability to approximately model the pinching behavior and its limitation is due to the inability to capture the strength degradation. The differences in the responses of the model created herein and the one in the reference are due the following factors:

- Different version of the software used.

- Variations in the values assigned to the model elements and loadings.

- The load (i.e., displacement history) is estimated based on the visual observation of the loading protocol from the reference.

### 3.3.2 Conclusion

The comparison between the shear wall model developed and the one from the reference is conducted to validate the nonlinear modeling procedure of the shear wall assembly when subject to cyclic loading conditions. From response figures, it can be concluded that the shear wall modelling using the 2 nodes non-liner spring is able to capture the nonlinear seismic response of the wall. As mentioned by Rinaldin et al. (2013), SAP2000 is considered to be an applicable software tool to simulate wood structural seismic response based on comparison of the analyses result obtained using SAP and the results obtained from other commercial software.
CHAPTER 4: CONCLUSIONS AND RECOMMENDATION

The state-of-the-art modelling methods for the wooden frame structure was studied and the main finds are summarized. The modeling methods are categorized as academic tools and commercial tools. The academic tools were mainly developed for research purposes with specific objectives such as defining the behavior of the wooden frame shear wall, hysteretic behavior of the connection between the sheathing and framing members. Commercial tools, on the other hands, are used for validating the results obtained from the experimental test and provide relatively simple modeling methods for structural parametric analysis and design verifications. Two of the commercial tools widely used for creating the numerical model of wooden frame structures were identified as ABAQUS/CAE and SAP2000.

The modelling method using SAP2000 was selected to be further investigated and it modeling method was studied using two wooden structure model, namely the 3D wood frame building model and the 2D wood shear wall assembly. The 3D building model was setup in SAP2000 and it static responses subject to uniform uplift wind pressure were obtained. Specifically, the reaction profiles the hold downs and anchor bolts for both end walls and side walls of the building were plotted and validated against the results from the model developed in the reference (Martin, 2010). As expected the wall showed symmetric responses. The hold down at the middle of the side wall experienced the maximum reaction among all the hold downs and the anchor bolts.

The second numerical model was setup for a shear wall assembly considering the nonlinear connections between the sheathing to the framing using nails. The sheathing to framing connection was modeled using two non-linear hysteretic springs along the two in-plane directions of the shear wall. The hysteretic behavior of the spring was modeled as the Pivot hysteresis available in the SAP2000 library. A cyclic horizontal displacement history based on the CUREE Standard Protocol was applied on the shear wall model and time history analysis was carried out. The results obtained were compared with the results of the model in the reference (Rinaldin et al., 2013) and good agreement was observed. The two wood structural models and their validation using SAP2000 demonstrated that SAP2000 can be a useful tool for modelling of wood frame structures considering environmental loadings, including extreme loading conditions.
Following the work presented in this report, it is recommended to develop a 3D nonlinear wood structural building model and validate the numerical model created with experimental results, such as shaking table tests conducted in the past 20 years. In addition, the numerical modeling procedure developed herein can be applied to study the effects of advanced construction materials, for example construction adhesives, on the performance of hazardous loading resistance of wooded frame building structures.
APPENDIX A.

MODELLING PROCEDURE FOR 3D-WOODEN FRAME BUILDING SUBJECTED TO UNIFORM UPLIFT PRESSURE IN SAP2000

4.1 General Overview

The detailed procedure for modeling of 3D-Wooden Frame structure on SAP2000 are provided in this section. This section describes modelling procedure of the 3D-wooden frame building under uniform uplift pressure. For the modelling purpose the procedure is adapted from Martin et al. (2010).

The actual foot print dimension of the building is 30ft by 40ft. In SAP, line segments are used for drawing the framing members thus the actual foot print of the building is taken as 29.3 ft. by 40 ft.

Figure A- 1: Model in SAP2000
4.2 Framing Members

As described on the section 3.2.1 the wall framing members are Spruce-Pine-Fir (SPF) and the roof framing members (Trusses) are Southern-Yellow-Pine (SYP). The dimensions of the framing components for wall and the framing members are shown on Table A-1 and Table A-2.

Table A-1: Dimensions of the wall framing members

<table>
<thead>
<tr>
<th>Components</th>
<th>Dimensions</th>
<th>Placement</th>
<th>Remarks</th>
</tr>
</thead>
<tbody>
<tr>
<td>1. Interior Studs</td>
<td>2in.X4in.</td>
<td>16in. c-c</td>
<td>Nominal Dimension</td>
</tr>
<tr>
<td>2. End Corner Studs</td>
<td>4in.X4in</td>
<td>-</td>
<td>Double 2X4's</td>
</tr>
<tr>
<td>3. Top Plates</td>
<td>4in.X4in</td>
<td>-</td>
<td>Two 2X4's</td>
</tr>
<tr>
<td>4. Bottom Plates</td>
<td>2inX4in</td>
<td>-</td>
<td>Single 2X4's</td>
</tr>
</tbody>
</table>

Table A-2: Dimension of roof framing members

<table>
<thead>
<tr>
<th>Components</th>
<th>Dimensions</th>
<th>Placement</th>
<th>Remarks</th>
</tr>
</thead>
<tbody>
<tr>
<td>1. Truss members</td>
<td>2inX4in</td>
<td>24in. c-c</td>
<td>Entire Truss Member</td>
</tr>
<tr>
<td>2. Pitch of Roof</td>
<td>4:12</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>3. Overhangs</td>
<td>24 in</td>
<td>-</td>
<td>Both sides</td>
</tr>
</tbody>
</table>

All the members in the walls are pinned, including studs-to-plate connections at the both ends.

This section provides step by step procedure for assigning and creating framing members.

1.1. Defining Material Properties

Step 1: Click on Define/Material definition
Step 2: Select Add New Materials a new window pops out. Select Region as United States, Material type as other.

Step 3: A window of material property options pops out. Create a name as desired in this case “Wall Members”. Select directional symmetry type as Isotropic and click on Modify/Show materials property to input Modulus of Elasticity, Poisson’s ratio, Weight per unit volume and mass per unit volume.
Step 4: A new window for assigning material property data pops out. Here we have to assign the values as mentioned in Table 3-2.
Step 5: Following the similar process for truss members. Only difference will be the material property.

1.2. Defining Material Section

Step 1: Select Define drop down box. Then place the cursor on section properties. Then click on Frame Sections. A window will pop out. Select on Add new property. Then a new window naming Add frame section property will appear. In frame section property type select other and then click on section designer.

![Add Frame Section Property](image)

**Figure A-6: Addition of the frame section**

Step 2: Then a window naming SD section data appears. Define section name as required. Define base material, and click on section designer.
Step 3: After clicking on Section designer on Step 2 a window will appear. Setup the suitable units. And click on “Draw” on the menu bar. Select draw solid shape and select rectangle. We can use shortcut on the left side toolbar.

Step 4: After drawing the section. Right click on the axis to set up the dimensions. The figures below show the section design of Edge studs, Interior studs, top plates, bottom plates, truss members as Table A- 1 and Table A- 2.
4.3 Sheathing members

Sheathing members as described on section 3.2.2 is elastic orthotropic Oriented Strand Board (OSB) having 7/16” thickness for the wall sheathing and ½” thickness plywood for roof sheathing. The properties of the sheathing elements are described on the Table 3-3. The shear modulus of the sheathing element is assigned following the correlation procedure with the shear wall deflection for the given load value predicted by National Design Specification (NDS). Similarly, this section provides detailed procedure for defining sheathing material:

*Step 1: Click on Define/Material Definition and follow same procedure for assigning framing material except it’s properties.*

![Material Property Options](image)

*Figure A- 9: Defining sheathing material directional symmetry*
Step 2: A window pops out naming Material Property Data. Assign the values for modulus of Elasticity, Poisson ratio Shear modulus, Weight and mass.

![Material Property Data](image)

**Figure A- 10: Defining material property**

Similar process is followed for the roof sheathing is defined according to Table 3-3.

Step 3: Select Section Properties/Select Area Section/ Select Section type as shell/ Click add new section
Step 4: Shell Selection Data window will appear. Define Section name/Select thick type shell/Define thickness, material name and material angle.

![Figure A- 12: Defining oriented strand board section](image)
4.4 Framing connectivity

All the framing members of the walls including stud to plate connections are pinned. Similarly, the vertical web members in the gable end trusses and overhang framing are considered to be pinned at each ends. The green dots in the figure shows the pinned connection.
4.5 Hold-downs and anchor bolts

The hold downs and anchor bolts are defined as the one grounded and three grounded springs respectively. The stiffness of the hold-downs is defined in the axial direction (i.e. Z-direction). The stiffness of the anchor bolts is defined as shear forces parallel and perpendicular to wall (i.e. X and Y-direction) and axial direction (Z-direction). The values of the stiffness are mentioned on Table 3-4. The hold-downs are modeled as one grounded springs with the stiffness in the axial direction (i.e. Z-direction). The anchor bolts are modeled as a three grounded springs with the stiffness along shear forces parallel and perpendicular to the wall (i.e. X and Y-direction) and axial direction (Z-direction).

Step 1: Select the joint at the edge
Step 2: Then click on “Assign” Menu. Then on the drop down menu select joint. Then select spring. Then “Assign Joint Spring” Window appears. Assign Stiffness values for the hold downs.

![Assign Joint Springs](image)

**Figure A- 17: Defining stiffness for the hold downs**

Step 3: For assigning the anchor bolts each at **32in spacing**. Select the point on the bottom plates each at 32” spacing. Then click on “Assign” menu and follow the same procedure as followed in the hold downs.

![Selection of the interior studs](image)

**Figure A- 18: Selection of the interior studs**
Figure A-19: Assigning stiffness for anchor bolts

The complete SAP model with the hold down and anchor bolts can be seen as

Figure A-20: SAP model with anchor bolts
4.6 Meshing of the shell elements

SAP2000 provides the features of automatic meshing. The shell element is meshed using the automatic meshing and meshed using either the maximum size and general divide tool. Automatic meshing is applied to the panel in the wall to properly interact with the framing member according to the framing spacing. Due to presence of the odd length in the gable walls general divide meshing is applied.

1.3. Creating Groups of Shell Elements

*Step 1:* Create group for bottom walls, side walls, gable walls and roof sheathing.

*Step 2:* Select bottom wall first. Click on Assign → Select Assign to Group from drop down menu

*Step 3:* Assign to group window will appear. Click on Define groups.

*Step 4:* Repeat same steps for side walls, gable walls and roof sheathing

---

**Figure A-21:** Assigning group for different components

---

1.4. Defining Mesh

*Step 1:* For Assigning Automatic Mesh. Select the element to mesh. For bottom wall. Click “Select” on the menu. Then select groups.
Step 2: Click on Assign on the menu bar → Area → Automatic Area Mesh. A window will appear named Assign Automatic Area Mesh. Select Auto Mesh area into objects of this maximum size (Quads and Triangles Only). Then Assign 16 in “Along Edge from Point 1 to 2” and “Along Edge from Point 1 to 3”
Step 3: To mesh Gable wall. Select the Gable wall similar to Bottom wall as in Step 1.

STEP 4: Click on Assign on the menu bar \( \rightarrow \) Area \( \rightarrow \) Automatic Area Mesh. A window will appear named Assign Automatic Area Mesh. Select “Auto Mesh Area Using General Divide Tool Based on Points and Lines in Meshing Group”. Assign 24in. Select Meshing group on the right hand top to Gable wall created previously.

![Figure A-24](image)

**Figure A- 24:** Assigning mesh for the gable walls using general divide

Step 5: Follow similar process for the side walls. The mesh size will be studs spacing.
Step 6: Model will display how the elements are meshed.

Figure A-25: SAP model with the assigned mesh

Step 7: Select Roof Sheathing group as assigned. Then, click on Assign on the menu bar/Area/Automatic Area Mesh. A window will appear named Assign Automatic Area Mesh. Select Auto Mesh area into objects of this maximum size (Quads and Triangles Only). Then Assign 24in “Along Edge from Point 1 to 2” and “Along Edge from Point 1 to 3”

Figure A-26: Assigning automatic area mesh for roof sheathing
Step 8: Model will display how the elements are meshed.

Figure A-27: Roof of SAP model with automatic area mesh
4.7 Load Application on the SAP model

1.5 Defining Load Pattern

**Step 1:** Setup load Pattern Name “Wind” → Type Wind → Self-weight Multiplier “0” → Click on Add New Load Pattern → Click “OK”

![Figure A-28: Defining the load pattern](image)

**Step 2:** Select Roof Sheathing → Assign → Area loads → Uniform to Frames → Select Load pattern “Z” → Load Direction Z → Load Distribution “Two way” → Assign Load “0.35 lb/in²”

![Figure A-29: Assigning Uniform uplift (0.35 lb/in²)](image)
### APPENDIX B

#### Table B-1: Summary of finite element models of wooden frame structures under wind loads

<table>
<thead>
<tr>
<th>Author</th>
<th>FE tool</th>
<th>Structure</th>
<th>Framing</th>
<th>Sheathing</th>
<th>Foundation</th>
<th>Connections</th>
<th>Loading Pattern/Validation of model/Application of model on research</th>
</tr>
</thead>
<tbody>
<tr>
<td>Kasal et al (2004)</td>
<td>ANSYS</td>
<td>Building</td>
<td>Beam elements</td>
<td>Plate elements</td>
<td>-</td>
<td>Non-linear springs</td>
<td>Design wind load of 30kN. Comparison of 5 different methods and four methods compared against the results of the physical experiment conducted on a full-scale test house. Provide basic understanding required for the development of improved design procedures for light-frame wood buildings subjected to lateral loads.</td>
</tr>
<tr>
<td>Zbis (2006)</td>
<td>SAP2000 V 7.11</td>
<td>Building</td>
<td>Linear &quot;Frame&quot; Element</td>
<td>Shell Elements (&quot;Membrane Type&quot;)</td>
<td>Support points restrained on three translational components (x,y, and z)</td>
<td>-</td>
<td>Area loads converted form point readings of the 124 pressure taps using area averaged method for each 15 examined directions. Full-scale load cell measurements with the finite element analysis results. Determination of the environmental loads effects on a typical wood building.</td>
</tr>
<tr>
<td>Pfretzscheuer et al (2012)</td>
<td>SAP2000</td>
<td>Building</td>
<td>Linear &quot;Frame&quot; Element</td>
<td>Shell Elements</td>
<td>Hold downs represented with only one spring oriented in Z-direction, anchor bolts represented with three springs oriented on X, Y and Z directions</td>
<td>-</td>
<td>Uniform uplift pressure of 50psf and ASCE 7-05 design wind loads. Modelling methods validated against the full-scale tests. Sub-assemblies models which included deflection of two-dimensional trusses, three dimensional roof assemblies, shear walls validated form the previous works. Martin et al. (2011) model can predict the lateral load paths of the complex geometric structure. The reentrant corners and the wall openings have the effects on the load path of the structure.</td>
</tr>
<tr>
<td>Wang (2013)</td>
<td>NASTRAN/PASTRAN</td>
<td>Building</td>
<td>Beam Element</td>
<td>Plate elements</td>
<td>Pin connection</td>
<td>Beam element</td>
<td>Measured wind pressure data. Comparison of the measured reactions on the test model with the theoretical model. To quantify the effect on the reliability of wood structural members.</td>
</tr>
<tr>
<td>Malone et al (2014)</td>
<td>SAP2000</td>
<td>Building</td>
<td>Frame Elements</td>
<td>Shell Elements</td>
<td>Hold downs represented with only one spring oriented in Z-direction, anchor bolts represented with three springs oriented on X, Y and Z directions</td>
<td>-</td>
<td>Dead</td>
</tr>
<tr>
<td>He et al. (2018)</td>
<td>ANSYS</td>
<td>Building</td>
<td>Beam Element</td>
<td>Shell elements</td>
<td>Rigid connection</td>
<td>Standard nonlinear spring elements</td>
<td>Time-averaged mean value of each wind loading cases and the time-history loading under wind direction normal to the roof ridge i.e., 90° comparison of deflection of roof sheathing panels and roof to wall frame connection on the basis of time-averaged response and time-history domain. Development of Finite element model to capture the behavior of a building under wind loading.</td>
</tr>
<tr>
<td>Quayyum (2019)</td>
<td>ANSYS</td>
<td>Building</td>
<td>Beam element (BEAM188)</td>
<td>3D shell element (SHELL181)</td>
<td>Beam element (BEAM188)</td>
<td>zero-mass nonlinear spring element (COMBIN39)</td>
<td>Tornado loads (EF5 tornado at Parkersburg, Iowa) May 25, 2008. Comparison of two models developed (standalone wall and full-house) wind load damage against field observation from the literature and system-level wall responses against experimental responses under in-plane lateral loads. Study the influence of the roof, side and partition walls, and base connection on the in-plane load resistance of the walls.</td>
</tr>
</tbody>
</table>
Table B-2: Summary of academic tools for wood frame modelling and analysis tools

<table>
<thead>
<tr>
<th>Author</th>
<th>Tool</th>
<th>Structure</th>
<th>Load</th>
<th>Objective/Result/Conclusion/Recommendation</th>
</tr>
</thead>
<tbody>
<tr>
<td>B. Folz &amp; A. Filiatrault (2001)</td>
<td>CASHEW</td>
<td>Shear Wall</td>
<td>Seismic Loading</td>
<td>Prediction of the load-displacement response and energy dissipation characteristics of wood shear walls under general cyclic loading. The load displacement response and energy dissipation characteristics of wood shear walls, with or without opening under arbitrary quasi-static cyclic loading was predicted. Generalization and use of the calibration process for the cyclic and seismic analyses of complete 3D wood-framed buildings is recommended as subject for future research.</td>
</tr>
<tr>
<td>Pang et al. (2007)</td>
<td>EPHMS</td>
<td>Shear wall</td>
<td>Time history ground motion</td>
<td>To improve the model predictions by incorporating more features of the actual hysteretic behavior into the EPHMS while still retaining the key advantage of CURUE model. There are total of 17 parameters required in this model to capture the non-linear hysteretic response of the shear wall. EPHM captured the hysteretic response of the wood shear walls than the static parameter CURUE model. EPHM is good choice for the peak displacement analyses when the accuracy in the displacement prediction is required over the entire range of design hazard levels. Development of the fragility curves using EPHM to estimate the drift-based failure probabilities of shear walls at various seismic hazard levels or for the use in post disaster condition assessment is recommended as future work.</td>
</tr>
<tr>
<td>W. Pang &amp; M. H. Shirazi (2010)</td>
<td>M-CASHEW2</td>
<td>Shear Wall</td>
<td>Gravity Loads</td>
<td>Model proposed to analyze shear walls with and without hold-down devices and with various levels of gravity loads. Hold-down devices have higher shear capacities than that of the same shear wall model without hold-down devices confirming the experimental findings. Investigation of the undesirable “over pinching” effects developed on the hysteretic behaviour of the oriented panel to frame system model.</td>
</tr>
<tr>
<td>He et al. (2001)</td>
<td>Light Frame 3D</td>
<td>Woodframe building</td>
<td>Static loading</td>
<td>Study the performance of 3D timber light-frame building under static loading condition by implementing mechanics based representation of the load-deformation characteristics of individual panel-to-frame nail connections in the diaphragm system. Preliminary application of the program showed good accuracy on the prediction of the behaviour of wood light-frame structures. It was found applicable with wide range of structural, material and loading variations. Study of the effectiveness of the program for the analysis of an individual structure or in the construction of response surfaces for the reliability assessment of general wood structures which eventually leads to development of design rules for wood frame systems.</td>
</tr>
<tr>
<td>B. Folz &amp; A. Filiatrault (2002)</td>
<td>SAWS (Seismic analysis of Wood frame Structures)</td>
<td>Woodframe building</td>
<td>Seismic Loading</td>
<td>1. Formulation and development of a simple numerical model to predict the dynamic characteristics, quasi-static pushover and seismic response of wood frame buildings. The numerical model was incorporated into the computer program SAWS and prediction are compared with recent shake table tests. The numerical predictions on the dynamic characteristics, pushover capacities and seismic responses of these structures were generally in good agreement with the experimentally obtained results.</td>
</tr>
<tr>
<td>Lindt et al. (2010)</td>
<td>SAPWood</td>
<td>Full Scale two-storied Building</td>
<td>Seismic Loading</td>
<td>1. To verify the accuracy of the soft ware package developed under NEESWood Project which included shear deformation of shear walls as as cumulative floor displacements caused by the out-of-plane rotations of the floor and ceiling diaphragms. 2. The comparison was done on interstory drifts and shear wall deformation for the various construction phases of the test buildings and excitation levels. Very good agreement between the numerical predictions and experimental results for the peak interstory drifts with the closest agreement for the building with the phase were OSB sheathing plus gypsum wall boards were applied to all walls. Requirement of the robust computational tools for the performance based seismic design (PBSD) tools.</td>
</tr>
<tr>
<td>Pang et al. (2014)</td>
<td>Timber 3D</td>
<td>Woodframe building</td>
<td>Seismic Loading</td>
<td>Development of the seismic analysis package which is the combination of the numerical model and experimental testing. It allowed the improved predictions of the seismic performance over a seismic loading condition ranging from the small deformation all the way to collapse. The package included slow and real-time hybrid tests which is used to study effectiveness of different retrofits used for strengthening the soft first story. Hybrid test allows the evaluation of the different retrofits without having to physically re-construct the first story multiple times.</td>
</tr>
<tr>
<td>Author</td>
<td>Tool</td>
<td>Structure</td>
<td>Load</td>
<td>Linearity</td>
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<tr>
<td>Xu J. (2006)</td>
<td>ABAQUS</td>
<td>Shear Wall/Building</td>
<td>Static monotonic, static cyclic, dynamic loading</td>
<td>Non-Linear</td>
</tr>
</tbody>
</table>
b. Simulated Hurricanes  
c. ASCE 7-05 | Linear    | Development of analytical model of light-framed wood structure using structure analysis software (SAP2000) to determine load paths within the structure under different loading condition (Uniform uplift, simulated hurricanes and ASCE 7-05) and incorporate the novel correlation procedure which eliminates the needs to represent each nail individually. Successful prediction of behavior of complex, three dimensional, wood framed structures with novel correlation procedure. Use of built in SAP function of layered shell to model gypsum wall board attached with the interior side of the studs. Investigation under other additional load cases, roof geometries, framing styles. Full-scale shear wall test to validate correlation procedure are future works recommended. |
REFERENCE


