NUMERICAL MODELING OF SEISMIC PERFORMANCE OF LIGHT-FRAME WOOD BUILDINGS

by

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A THESIS SUBMITTED IN PARTIAL FULFILLMENT OF

THE REQUIREMENTS FOR THE DEGREE OF

MASTER OF APPLIED SCIENCE

in

THE FACULTY OF GRADUATE AND POSTDOCTORAL STUDIES

(Civil Engineering)

THE UNIVERSITY OF BRITISH COLUMBIA

(Vancouver)

April 2017

Abstract

Light-frame wood structures are the most prevalent construction type in North America, representing over 90% of the residential building stock. Many of these buildings were built prior to the adoption of seismic engineering design practices and thus may be vulnerable in a seismic event. The primary objective of the research is to examine the use of numerical models to predict the seismic behaviour of light-frame wood structures. Models for (i) a full-scale two-storey house, (ii) a full-scale classroom, and (iii) a two-storey school block were created in light-frame wood non-linear analysis packages. The first two models were validated with full-scale shake table tests. The effect of sheathing type, nailing schedule, openings and ground motion characteristics on the seismic behavior of light-frame wood buildings were investigated. A three-dimensional model of a two-storey light-frame timber house with different sheathing configurations was calibrated using non-linear dynamic analysis to the full-scale experimental shake table results. The model of the test structures was able too predict the time-history response of the drift with reasonable accuracy. The contributions of the strength and stiffness from the openings and non-structural sheathing were included in the model. A detailed numerical model (each nail, framing member, hold-down and panel are modeled), as well as a global numerical model was used to predict the seismic behaviour of an additional dynamic shake table testing was also conducted on a full-scale classroom. The effect of openings, sheathing and ground motion duration was further investigated. Finally, the seismic performance of existing structures and the performance of several retrofit options was investigated with the validate modeling techniques using non-linear dynamic analysis of a typical school block built between 1950 – 1960 in Vancouver. The retrofit options met the target performance objectives.

Preface

Chapter 3 is based on shake table testing of a two-storey house with different sheathing configurations as part of the Earthquake 99 Woodframe House Project initiated in 1999. The testing was conducted in the Earthquake Engineering Research Facility (EERF) at UBC. C. E. Ventura, G. W. Taylor, H. G. L. Prion, M. H. K. Kharrazi and S. Pryor made significant contributions to the testing design and implementation and data analysis. The financial support was provided by Simpson Strong-Tie Co., Inc., Forest Renewal BC, Natural Sciences and Engineering Research Council (NSERC) of Canada, Canada Customs and Revenue Agency (SR&ED), Canada Mortgage and Housing Corporation, National Research Council of Canada, British Columbia Ferry Corporation and UBC Department of Civil Engineering.

Chapter 4 is based on the shake table testing of a full-scale classroom at the EERF-UBC. The testing program is part of the Post Earthquake Evaluation study and long-duration study for the BC Schools Seismic Retrofit Program, a collaborative partnership between the British Columbia Ministry of Education (BC MOE); the Association of Professional Engineers and Geoscientists of British Columbia (APEGBC); the University of British Columbia. Testing was coordinated by M. Turek, M. Motamedi, and Graham Taylor. The structure was built and repaired by Rain City Renovations.

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List of Abbreviations

APEGBC	Association of Professional Engineers and Geoscientists of British Columbia
ATC	Applied Technology Council,
CASHEW	Cyclic Analysis of Wood Shear Walls
CLT	Cross Laminated Timber
CUREE	Consortium of Universities for Research in Earthquake Engineering
CWC	Canadian Wood Council
DDL	Design Drift Limits
DKB	Distributed Knee-Braced
DOF	Degree-of-Freedom
EERF	Earthquake Engineering Research Facility
EERI	Earthquake Engineering Research Institute
EPHM	Evolutionary Parameter Hysteretic Model
FE	Finite Element
FEM	Finite Element Model
FEMA	Federal Emergency Management Agency
FVD	Fluid Viscous Damper
GWB	Gypsum Wall Board
HSS	Hollow Structural Sections
HWS	Horizontal Wood Siding
ICBO	International Conference of Building Officials
ICC	International Code Council
IDA	Incremental Dynamic Analysis
IMF	Inverted Steel Moment Frame
LDRS	Lateral Displacement Resisting System
M-CASHEW	MATLAB - Cyclic Analysis of Wood Shear Wall version 2
MCE	Maximum Considered Earthquake

MSE	Mean Squared Error
MSTEW	Modified Steward Hysteretic Model
NBCC	National Building Code of Canada
NEES	Network of Earthquake Engineering Simulation
NLTHA	Nonlinear Time History Analysis
NP	Nail Pattern
NSERC	Natural Science and Engineering Research Council
OSB	Oriented Strand Board
PBSD	Performance based seismic design
PBSR	Performance based seismic retrofit
PDE	Probability of Drift Exceedance
PEER	Pacific Earthquake Engineering Research Centre
PGA	Peak Ground Acceleration
RESST	Residual Strength Hysteric Model
SAPWood	Seismic Analysis Package for Wood-frame structures
SAWS	Seismic Analysis of Wood-frame Structures
SDOF	Single-Degree-of-Freedom
SDPWS	Seismic Design Provisions for Wind and Seismic
SMA	Shape Memory Alloy
SMF	Special Moment Frame
SRG	Seismic Retrofit Guidelines
SSMF	Steel Special Moment Frames
WSP	Wood Shear Panel

Acknowledgements

I would like to express my deepest gratitude to my supervisor, Dr. Carlos Ventura, whose support has allowed me to openly explore my field. His vast knowledge and encouragement gave me the direction to question the applications and implications of my work and gave me motivation to work hard and achieve more. His patience and kindness to me will always be remembered.

I thank Mike Fairhurst and Armin Bebamzadeh for patiently answering my endless questions and providing companionship each day.

I thank Martin Turek, Graham Taylor, and Mehrtash Motamedi for organizing and conducting the shake table laboratory experiments. Special thanks to Scott Jackson for technical support.

I thank Dr. W. Pang for allowing me to use his numerical modeling program and answering my questions through the process.

Special thanks are owed to my parents who have supported me through my years of education. I want to thank Adam Silvester for this continuing encouragement and steadfast belief in me.

This thesis is dedicated to my Mom and Dad.

Your unconditional love and support has given me strength and confidence in everything I do.

Thank you for chatting everyday through my commutes and lunch breaks.

Chapter 1: Introduction

Light-frame wood structures are the most prevalent construction type in North America, representing over 90% of the residential building stock (CUREe, 1998). Many of these buildings (over 75% in San Francisco, United States (Scawthorn C. , Kornfield, Seligson, & Rojahn, 2006) and over 40% in Vancouver, Canada (Ventura, Finn, Onur, Blanquera, & Rezai, 2005)) were built prior to the adoption of modern building codes and seismic engineering design practices. Thus, a number of buildings may be vulnerable in a seismic event due to insufficient strength and stiffness of their seismic force resisting system, poor load path definition, and vertical/torsional irregularities. Many of these structures were built in a construction era where the use of archaic materials (i.e. lath and plaster or horizontal boards) and archaic construction practices with little, to no detailing for establishing a loading path were applied. The quality of the materials and level of detailing can significantly affect the performance and likelihood of collapse in a seismic event (Bahmani P. , 2015). A study initiated by the San Francisco Department of Building Inspection and the Applied Technology Council (ATC) in California predicted that 40% – 80% of the structures will be flagged as unsafe and 25% of existing multistory wood buildings would be expected to collapse in a magnitude 7.2 earthquake in the Bay Area of San Francisco (Applied Technology Council, 2008). Therefore, there is a critical need to access and retrofit the existing light-frame wood structures.

Light-frame wood structures use wood shear walls as the primary gravity and lateral force resisting system. The floor and roof diaphragms distribute the gravity and lateral loads to bearing and shear walls. The walls systems then transfer the loads to the next lower level or to the foundation, as shown in the depiction of the loading path in Figure 1 (Toothman, 2003). Wood shear walls, as shown in Figure 2, consist of: vertical studs; framing members with frame-to-frame connections; sheathing panels, and sheathing-to-framing connections. The in-plane lateral resistance is primarily developed through the sheathing-to-framing connections (i.e. nails) in racking deformation. The connections provide hysteretic damping and energy dissipation under cyclic or seismic loading conditions.

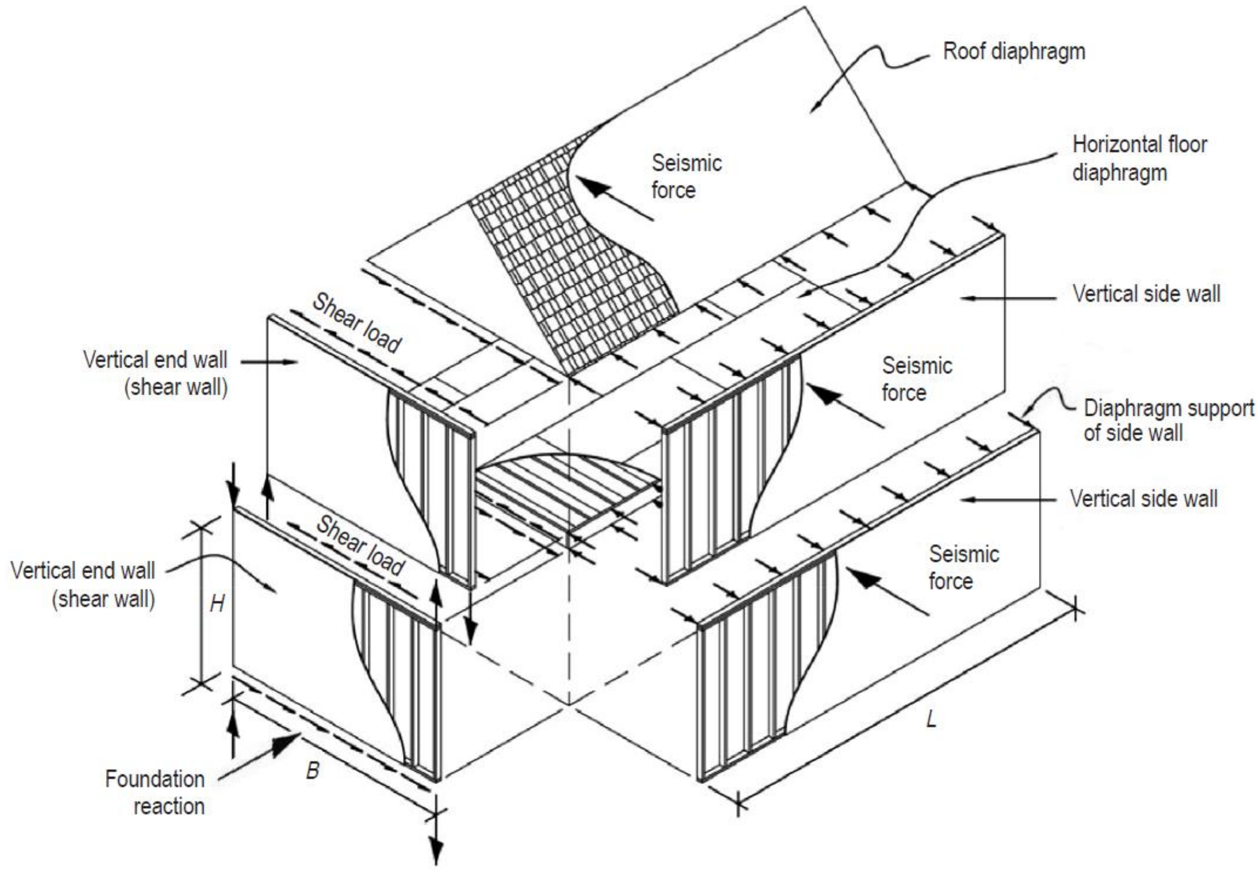


Figure 1: Light-frame wood building with load path illustrated (Toothman, 2003)

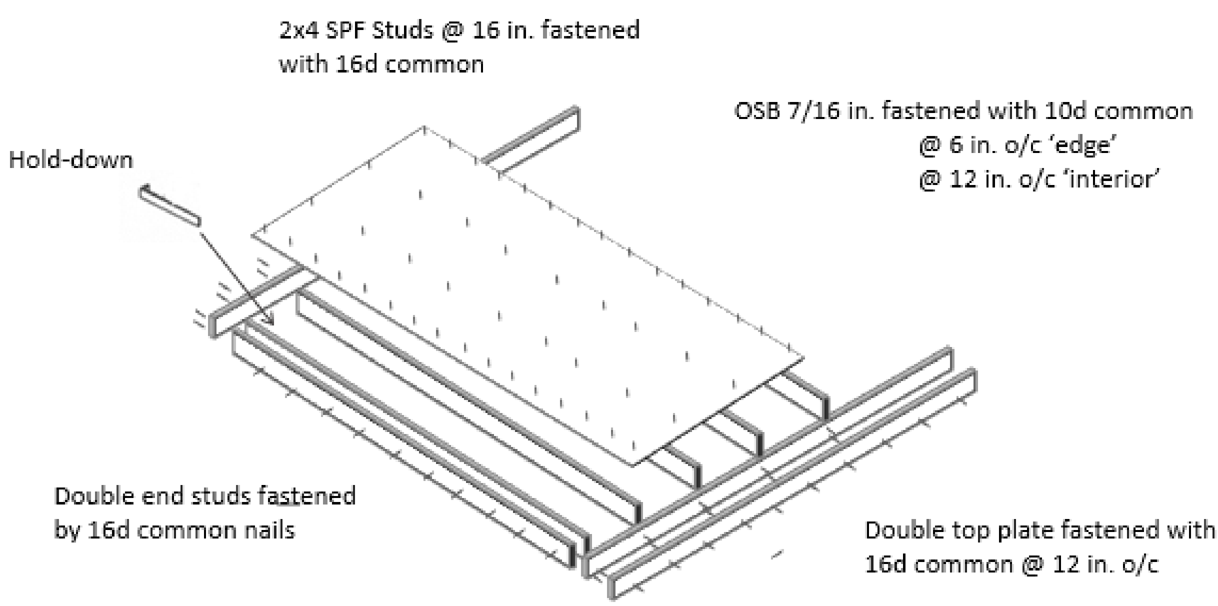


Figure 2: Typical Shear wall construction (Heine, 1997)

1.1 Problem Overview

The prescribed capacity/demand methodology in the current code practice does not provide an indication of the damage level of a structure after an earthquake and is most often not a financially viable option for retrofit. Performance based seismic design (PBSD) can be used to provide a rational basis for verifying life-safety of buildings and to develop cost-effective tools for seismic assessment and retrofit. Inelastic deformation predictions can be used rather than force or base shear demand to quantify the building performance and the probability of collapse given a certain intensity of earthquake shaking.

Performance-based engineering and design requires numerical models that can accurately predict the deformation and collapse of a structure. The level of nonlinearity, structural redundancy and load history dependence of light-frame wood structures make it difficult to create accurate global models. State-of-the-art finite element (FE) numerical models can accurately predict the lateral behaviour of wood frame buildings, however these models tend to be computationally intensive and therefore are not feasible for common practice. Furthermore, the behaviour of short-period light wood-frame structures is detail dependent; the size of openings, the number of hold-downs, the nailing schedule, as well as the structural and non-structural sheathing type can change how a structure will behave in an earthquake. Simplified single-degree-of-freedom (SDOF) analytical models of short-period structures commonly ignore diaphragms, foundations, and other sources of system flexibility. Hence, there seems to be little agreement in academia and industry on how to model light-frame wood buildings. Reliable numerical modeling could provide a rational method to assess and retrofit existing structures by evaluating the predicted performance.

1.2 Goals, Objectives, Tasks and Scope

This research aims to investigate the use of light-frame wood numerical modeling to help develop a more rigorous and standardized methodology to model these types of structures; to contribute to ensure adequate life-safety of structures; to help prioritize retrofits and define what level of retrofit is needed; and to use a performance-based approach to quantify different seismic upgrading options.

The primary objective of the research is to examine the ability to use three-dimensional numerical nonlinear modeling to predict the dynamic behaviour of a light-frame wood structure. Achieved through the following sub-objectives:

1. Validate a numerical model with full-scale testing.
2. Investigate the effect of sheathing layer type, nailing schedule, openings on the seismic response of light-frame wood buildings and validate the modeling methods with full-scale experimental results.
3. Investigate the ability for detailed and global numerical models to predict the seismic behaviour for long duration ground motions.
4. Predict the seismic performance of a typical existing light-frame wood building and evaluate the performance of several retrofit options with specific performance objectives.
5. Evaluate the seismic behavior the collapse mechanisms of light-frame wood buildings with the validated numerical models.

The work was broken down into a series of tasks to accomplish the objectives of the research. First, the available commercial and state-of-the-art numerical modelling methods for light-frame wood structures were researched to determine what numerical programs would most appropriate for the study. The programs: SAWS (Folz & Filiatrault, 2002); SAPWood (Pei & van de Lindt, 2010) and Timber3D (Pang, Ziaei, & Filiatrault, 2012) are generally accepted and validated by the academic community for global seismic modeling of light-frame wood structures. CASHEW and M-CASHEW2 were developed for detailed modeling of wood shear walls. Each program has several constrains and limitations that were considered.

Second, the experimental results from available testing at UBC, as well as published material were catalogued to develop material hysteretic models for various construction materials typical in light-frame wood buildings. The material models were based on multiple experimental results from several sources, as well as recommendations from ATC-116 (Pang, 2015) and FEMA P-807(2012) technical review committees. The materials models were then calibrated to the shake table results for a full-scale two storey house with various sheathing configurations.

Third, the effects of openings, combining materials, and hold-downs were studied. Experimental results, existing analytical studies, and guidelines were researched to develop a framework on how to consider these effects on the lateral resistance and modeling of the global structure. Analytical studies validated with full-scale shake table tests were completed.

Forth, the retrofit method on light-frame wood structures were researched. Conventional, as well as alternative seismic retrofit options were investigated. Material hysteretic models defining the load-displacement behaviour were defined based on experimental results. Existing analysis tools using simplified single-degree-of-freedom models were used to access and evaluate a typical school block built in the 1950s as seismically deficient. The resistance requirements for the retrofits were defined based on the simplified model; the performance of the retrofits were then assessed using a three-dimensional global numerical model of the school building block.

Finally, a method to evaluate the performance of the retrofit options for an existing structure was defined. The performance objectives were based on recommendations from the FEMA-P807 guidelines, as well as the NEES-Wood and Soft-Storey projects. Non-linear time history analysis was used to determine the collapse probability, medium drift and probability of drift exceedance at different hazard levels for crustal, subcrustal and subduction events for Vancouver, British Columbia.

The studies in this thesis are limited to light-frame wood structures typical to North America. The research focuses on the use of numerical models for assessment and retrofit of existing light-frame wood buildings. The work could, however, be applied to assess new light-frame wood construction for design. CLT and heavy timber structures were considered outside the scope of the study. The ground motions for non-linear time history analysis were selected based on the seismicity in the lower mainland of British Columbia. Crustal, subcrustal and subduction earthquakes were considered; near-fault effects were outside the scope.

1.3 Organization of Thesis

This thesis is organized into five chapters to address objectives and goals of this study.

In Chapter 2, entitled *"Literature Review"*, the performance of light-frame wood buildings in previous earthquakes and the available state-of-the art global, shear wall and material hysteretic numerical modeling techniques and programs were summarized.

In Chapter 3, entitled *"Global Numerical Model Validation"*, a numerical model was developed using Timber3D and calibrated to the experimental tests of a light-frame wood house conducted in the Earthquake Engineering Research Facility (EERF) at the University of British Columbia (UBC) as part of the Earthquake 99 (EQ-99) testing program.

In Chapter 4, entitled *"Prediction of Full-Scale Test"*, detailed (M-CASHEW2) and global (Timber3D) numerical models were developed and compared to the experimental shake table dynamic response of a full-scale classroom tested at EERF, UBC as part of the Seismic Retrofit Program for public schools implemented by a coloration between the Ministry of Education, the Association of Professional Engineers and Geoscientists BC (APEGBC) and UBC. The effect of opening, nailing schedules and sheathing layers were investigated.

In Chapter 5, entitled *"Seismic Assessment and Retrofit"*, a Timber3D numerical model for a typical light-frame wood school block constructed prior to the 1960s based on the validated modeling methodology from Chapter 3 and Chapter 4. The seismic performance of the existing structure was evaluated over a range of hazard levels using non-linear time history (NLTH) analysis. Several retrofit options were proposed based on simplified performance based engineering tools and the performance of the retrofits were evaluated with non-linear time-history (NLTH) analysis.

In Chapter 6, entitled *"Summary and Conclusions"*, the research completed in this study and the contributions to the structural engineering research and practice has described. Recommendations for future research in field of study were made.

Chapter 2: Literature Review

This chapter provides a literature review of the performance of light-frame wood structures in resent earthquakes, a summary of the current state-of-the-art numerical models and validation testing for light-frame wood buildings and shear walls. The limitations of each of the numerical models have been discussed. Material Hysteretic models developed and used for light-frame element-wise numerical modeling has also been described in detail.

2.1 Performance of Light-Frame Wood Structures in Recent Earthquakes

Wood-frame structures have traditionally been considered to perform well in terms of life safety during moderate seismic events. This belief is derived from the inherit light weight of timber structures, as well as the high deformation capacity, structural redundancy and the ability to dissipate energy within the connections. Although this has been generally observed, in many recent, worldwide earthquakes there has been several recorded incidences of excessive damage or collapse of light, wood-frame structures subjected to significant ground shaking. These cases are usually caused by easily identifiable structural deficiencies, such as a weak first storey, inadequate load path, or inadequate anchorage. Rainer and Karacabeyli (2000) provide an overview of the performance of light, wood frame buildings in several past earthquakes.

In the 1971 San Fernando, California earthquake (magnitude 6.7), many older wooden houses suffered varying levels of damage: from non-structural damage to collapse of the structure. Some newer, multi-storey apartment buildings with large openings at their ground level were also severely damaged. The prominent deficiencies observed were: sliding off foundations, collapse of cripple walls, collapse of non-structural partitions such as porches and chimneys, and collapse or major damage in weak first storeys. Most modern (at that time) houses with no major deficiencies performed well (Pacific Fire Rating Bureau, 1971).

The 1987 Edgecumbe earthquake in New Zealand comprised a magnitude 6.3 main shock, preceded by a magnitude 5.2 fore-shock, and followed by four significant aftershocks (magnitudes greater than 5.0). The earthquake occurred in a rural area near several small towns, including the town of Edgecumbe, which was 8km from the epicenter of the earthquake. Although nearly 7000 buildings (mostly light, wood-frame structures) were affected by the shaking, Pender and Robertson (1987) reported no deaths or serious injuries. No houses collapsed, and less than 50 structures suffered substantial damage; damage was typically due to sliding of foundations, collapse of brick veneer, collapse of brick chimneys, and failure of foundation posts.

The 1989 Loma Prieta, California (magnitude 7.1) earthquake was one of the most damaging earthquakes in Western North America. Although most wood buildings near the epicenter of the Loma Prieta earthquake performed well, there were several recorded collapses of older four-storey wooden apartment buildings in the Marina Bay district of San Francisco. These collapses were observed in buildings with large garage openings in their first storeys which caused the weak first-storey to collapse (Bruneau, 1990; Harris & Egan, 1992).

The 1994 Northridge earthquake (magnitude 6.7) caused between 30-40 billion U.S. dollars in property damage, making it one of the most expensive natural disasters in the history of the United States (EERI, 1996). More than $20 billion in losses was directly associated with the repair cost of structural and non-structural (e.g. gypsum wall board cracking) components of wood frame residential buildings The light-frame wood buildings have been observed to have structural and non-structural (e.g. gypsum wall board cracking) repairs after a seismic event (Pei S. , 2007). Similarly, to both the 1989 Loma Prieta and 1971 San Fernando earthquakes, several multi-storey apartment buildings collapsed onto weak first storeys during the Northridge earthquake (EERI, 1996).

2.2 Global Numerical Models

The level of nonlinearity, structural redundancy and load history dependence of light-frame wood structures make it difficult to create accurate global models. The behaviour of light-frame wood structures is detail dependent; the sheathing configuration, nailing pattern, anchorage, and size of openings significantly affect the seismic response (Filiatrault, Fischer, Folz, & Uang, 2002). Furthermore, the load paths and structural elements are not easily identifiable due to the numerous interconnected framing members and structural redundancy. State-of-the art numerical models have been developed: a 3D finite element (FE) model was proposed by Collins et al. (2005) that uses nonlinear diagonal springs, shells, and beams in the ANSYS FE package. Tarabia and Itani (1997) developed a 3D model with special wooden shear elements. Mosalam et al. (2002) created a three-storey light-frame wood model consisting of shell and beam using SAP2000. These models could predict the behaviour of light-frame wood structures with considerable accuracy, however were computationally intensive and thus have a limited application for use in practice.

Applying several simplified kinematic assumptions and using inter-story drifts as the main performance indicator is a way to balance computational expense with accuracy. The pancake style biaxial model (Folz & Filiatrault, 2004b) has two translational degree of freedom (DOF) and one rotational DOF at each storey level. The model has been implemented in the nonlinear dynamic analysis programs SAWS (Folz & Filiatrault, 2002) and SAPWood (Pei & van de Lindt, 2010); these programs were specifically developed for light-frame wood structures. Each shear wall is represented with a pure non-linear spring. The diaphragm is assumed to be perfectly rigid; this assumption was presumed to be acceptable for buildings with a diaphragm planar aspect ratio within the order of 2:1. The effect of vertical motion of the system and story height were neglected and the floors were assumed to act independently (Folz & Filiatrault, 2004b).

To evaluate the predictive capacity of the SAWS model the numerical predictions were compared to the experimental results for the shake table tests of a full-scale, two-storey wood frame house as part of the

CUREE-Caltech Woodframe Project (Fischer, Filiatrault, Folz, Uang, & Seible, 2001). The house was designed to represent California residential construction in accordance to the 1994 edition of the Uniform Building Code (ICBO, 1994) for seismic zone 4. The house with and without finishes (i.e. gypsum wall board (GWB) partition walls and sheathing, stucco exterior wall finishing, windows and doors) was tested on the shake table and compared to the model developed in SAWS. The input ground motions were sourced from the 1994 Northridge Earthquake recorded at Canoga Park and scaled between 0.12-1.2 with a peak ground acceleration of 0.05g-0.89g. The model could achieve acceptable predictions for the relative displacement when compared to the experimental results. Folz and Filiatrault (2004b) attributed the discrepancy between the numerical predictions and experimental results to the SAWS model not properly capturing the torsional response and diaphragm flexibly of the test structure. It should also be noted that the maximum drift observed over the structure was less 2.0%. At this drift level the structure behaves near elastically and the response is relatively simple to predict in comparison to higher drift levels where collapse is likely to occur.

The biaxial model can predict the seismic response at the first and often the second level with reasonable accuracy. At higher floor levels the cumulative uplift of hold-down rods and coupled interaction between lateral displacements and horizontal diaphragm rotation becomes more significant. The bearing contacts between the framing (e.g. stud-to-sill plate and sill-plate-to- foundation), uplift of hold-downs and shear slip of anchor bolts can significantly affect the lateral behaviour of the structure (Christovasilis I. , 2010). Thus, the role of hold-down devices and overturning moments should not be ignored for taller buildings.

A coupled shear-bending model was developed by Pei et al. (2010) to account for the out-of-plane floor rotations and rocking/uplift behaviour observed in the shake table benchmark test building (Christovasilis, Filiatrault, & Wanitkorkul, 2007) as part of the NEESWood Project. A pure shear formulation does not adequately capture the behavior mechanism of the storeys at higher levels. Six DOFs were assigned to each storey and the overall response was controlled by shear deformations of the shear walls and out-of-plane rotations of the floor and ceiling diaphragm controlled by hold-down restraints. The shear walls were modelled with non-linear pure shear elements, and the uplift restraints and compression struts/studs were modeled with non-symmetric linear springs. The diaphragm was modeled as perfectly rigid in plane and allowed for diaphragm rotation out-of-pane (analogous to Euler-Bernoulli beam theory). This model was developed as part of the software package: SAPWood.

In a study by Pei and van de Lindt (2012) a SAPWood coupled shear-bending model was compared to the shake table data for an isolated three-storey wood shear wall. Each storey consisted of 2.44m×2.44m wood shear walls with 1421 kg of seismic mass. Continuous vertical hold-down devices were installed. These types of hold-down systems are commonly used for stacked wood shear wall assemblies with an aspect ratio of 4:1. The structure was tested on an uniaxial shake table subjected to the near-field Rinaldi recording of the Northridge earthquake. The lateral responses and uplift at each story and tension force in the steel rods were recorded. The numerical model could accurately predict the storey deformation and simulate the influence of the hold-down system. The decomposition of the overall inter-story drift into pure shear and rigid body rotation showed that the behavior of the upper storeys of the stacked shear wall system was dominated by the cumulative uplift and out-of-plane rotation of the diaphragm. The author noted that the test and model of the isolated walls does not fully characterize the mechanisms in a full-scale structure. In a building system, it is likely that entire walls will go into tension and/or compression. This behavior is not captured in conventional earthquake engineering practice that are designed at the sub-assembly level.

The SAPWood model was also validated with the experimental results from a full-scale six-story wood frame building tested at Japan’s E-Defense shake table (Pei & van de Lindt, 2011). The test was part of the NEESWood Capstone test program and is described in detail by Pei et al. (2010) and van de Lindt (2010). The structure was designed as an apartment building with a footprint of 18mx12m (60ft x 40ft) and an overall height of 17m (56ft). Continuous anchor tie-down systems at the ends of all shear walls, compression stud packs in the lower floor shear walls, and shear transfer details within the walls and floor system were installed. Interior GWB walls were installed; the exterior finishing material was not included in the testing. The building was tested with the vertical and horizontal (x, y, z) ground motion components from the Canoga Park Station during the 1994 Northridge earthquake scaled to represent the seismic hazard levels with 50%, 10%, and 2% probability of exceedance in 50 years as per the ground motion research by Krawinkler et al. (2003).

The SAPWood numerical model predicted the inter-story drifts and global displacements of the building with reasonable accuracy. The model slightly overestimated the base shear of the structure and slightly underestimated the maximum inter-story drifts. The author proposed that a factor should be used to ensure conservative design. The numerical model could not accurately predict the torsional response of the structure and therefore is not suitable to capture the effect of the accidental torsion on the expected performance of the structure (Pei & van de Lindt, 2011).

The SAPWood model could predict the peak interstorey drifts for a full-scale house shake table test with considerable accuracy. The shake table testing program (Christovasilis, Filiatrault, & Wanitkorkul, 2007) involved testing a three-unit, two-story townhouse designed to the Uniform Building Code (ICBO, 1988) for seismic zone 4. Common design and construction practices in California were followed. The apartment units consisted of 170m^2 of living space with an attached two car garage. Christovasilis (2007) observed a potential soft story mechanism along the line of the garage wall. The two-story building tested by Filiatrault et al. (2010) was modeled by van de Lindt et al. (2010) for the structure at four building phases: (1) structural wood walls installed; (2) GWB installed on structural walls; (3) GWB interior partition walls installed; and (4) the stucco exterior finish installed. The building was tested with several crustal ground motions sources from the 1994 Northridge earthquake scaled to a PGA between 0.05-0.84. The maximum interstorey drift observed in the tests was just over 2.0% drift. It should be noted that this drift level is well within the life-safety limits of light-frame wood structures.

A study by Pang & Rosowsky (2010) compared the accuracy of the response predictions of a numerical model with a perfectly rigid diaphragm and a numerical model with a semi-rigid FE beam-spring diaphragm. The predictions were compared to same shake table test of the a three-unit two-story townhouse, as mentioned above. The semi-rigid FE beam–spring model accurately predicted the magnitude of the displacements and deformed shapes when compared to the experimental results. The rigid diaphragm model underestimate the magnitude of the displacements observed in the shake table experiments.

The SAWS biaxial model and SAPWood coupled shear-bending model uses rigid plates for the floor diaphragms, therefore the models have limited accuracy when in-plane deformations of the floor diaphragms are large. For structures with small building plans and isolated stacked shear wall systems (Pei S. , van de Lindt, Pryor, Shimizu, & Isoda, 2010) the rigid body assumption is appropriate. Full-scale experimental tests, conducted as part of the NEESWood project, indicate that there may be significant out-of-plane deformations of the floor diaphragm with larger floor palms. Therefore, the roof and floor diaphragms should be modeled as semi-rigid (Christovasilis, Filiatrault, & Wanitkorkul, 2007). A three-dimensional modelling program, Timber3D, was proposed by Pang et al. (2012) as an extension of the 2D shear wall models. The model was formulated based on co-rotational and large displacement theory and is defined using two types of elements: frame elements and link elements. The in-plane and out-of-plane roof and floor diaphragm flexibility is characterized with 2-node, 12-DOF (three translational and three rotational DOF at each node) frame elements. The frame elements can capture tension, compression, torsion and bending effects. The variation of axial loading is tracked in the analysis and the geometric stiffness matrix of the frame elements are updated at each time-step to account for geometric nonlinearity caused from large deformations. The lateral stiffness of the wood shear walls is modeled with 2-node, 6-DOF, zero-length, link elements. The axial stiffness of the studs can be modeled with either the frame or link elements. Hold-downs can be modeled explicitly with link elements or can be accounted for by altering the shear wall link elements. Shape functions of the frame elements are applied to eliminate the DOFs of the link elements to reduce the computational time. The condensed global stiffness matrix is then dependent on only the number of frame elements in the model. The co-rotational formulation involves decomposing the total deformation of the framing elements into the rigid body motion and relative deformations. The global stiffness matrix is then updated based on the rotated coordinate system of the elements (Pang, Ziaei, & Filiatrault, 2012).

The Timber3D model could predict the seismic performance of a wood-frame structure with considerable accuracy for the full range of response: small deformation to collapse of the structure (Pang, Ziaei, & Filiatrault, 2012). As part of the NEES-soft project two full-scale buildings were tested in 2013: (i) a hybrid test of a three-story building at the University at Buffalo, and (ii) a shake table test of a four-story building at the University of California – San Diego. The buildings were retrofitted and tested in multiple phases using two retrofit methodologies: soft-story retrofit only (as described in the FEMA P-807 Guidelines) and performance based seismic design (PBSD).

A pseudo-dynamic real-time hybrid test of the three-story wood-frame building was completed to study soft-story retrofit options. The structure was designed to represent 1920 – 1970 typical San Francisco Bay Area wood construction. The first story of the structure was modeled numerically in Timber3D with the Cross-laminated timber (CLT), distributed knee-braces (DKB), inverted steel moment frame (IMF), fluid viscous damper (FVD), shape memory alloy (SMA) and steel moment frame (SMF) retrofit options. The remaining upper storeys were constructed on the Buffalo lab strong floor and was physically tested with the hydraulic loading equipment. The exterior sheathing of the building was 1x10 horizontal wood siding fastened with two 8d common nails at each stud. The interior was covered with 12.5 mm (0.5in.) thick

GWB. The hybrid testing set-up allowed for more retrofits to be tested, while still physically examining the damages that would occur in the upper storeys. The tests revealed that the retrofit solutions performed well and met the objectives of the FEMA P-807 retrofit.

Pang et al. (2012) predicted the collapse of the three-storey NEES-Soft apartment building using the Timber3D numerical model. Incremental dynamic analysis (IDA) was performed with 22 bi-axial ground motions and global collapse was defined when the tangent-to-initial slope ratio of the IDA curve was less than 20%. The medium collapse capacity was predicted to be 13% interstorey drift. The model showed that the building is susceptible to side-sway collapse in the first-story.

The full-scale four-storey wood-frame building tested at University of California – San Diego was subjected to a series of seismic tests on the NEES outdoor shake table (van de Lindt J. , et al., 2014). The architecture of the building was selected to be like a typical San Francisco Bay Area soft-storey wood frame structure. The top three storeys were designed with two two-bedroom apartment units; the bottom storey was designed as a parking garage with several large openings. The high wall density in the upper storeys combined with the large openings in the first-storey created a very soft and weak first-storey. The building represented a corner building with two neighboring buildings on its North and West sides. Because of this, the North and West first-storey walls had no openings and were much stiffer than the South and East Walls. This configuration created a large geometric stiffness irregularity in the already vulnerable first-storey. The test structure was instrumented with over 400 instruments and subjected to two earthquake records: one from the 1989 Loma Prieta earthquake, and another from the 1992 Cape Mendocino earthquake, scaled from 0.2g to 1.8g (MCE level) (van de Lindt J. , et al., 2014).

The building was retrofitted using the FEMA P-807 and PBSD retrofit methodologies with multiple retrofit options including: steel special moment frames (SSMF) and inverted moments frames (IMF); rocking cross laminated timber (CLT) walls; energy dissipation systems (dampers); distributed knee-brace (DKB) systems and shape memory alloy device (Bahmani P. , van de Lindt, Gershfeld, Mochizuki, & Pryor, 2014). The structure was then tested in multiple phases on a full-scale shake table. In the FEMA P-807 retrofits, most the damage and deformation was concentrated in the first-storey – very little damage was transferred to the upper storeys. In the PBSR retrofits damage was distributed over the height of the structure, which helped it resist higher intensities of ground shaking. These tests demonstrated that retrofit solutions could adequately meet the performance objectives defined by the two retrofit methodologies (van de Lindt J. W., Bahmani, Mochizuki, & Pryor, 2014).

The four-storey apartment building without retrofits was also tested to collapse. This building had significant soft-storey deficiencies in both directions. The building was tested with a series of smaller less intense shaking levels followed with the Superstition Hills record scaled to the maximum credible earthquake (MCE). The first Superstition Hills run caused the structure to have a residual drift of 16.4% in the first story; above 14% interstorey drift the building was deemed to be unrepairable and uninhabitable. The building collapsed in the second run with the Superstition Hills record at a maximum first-storey drift of 19.3%. The building collapsed toward one of the soft-side corners in a side-sway torsional mechanism. It was concluded that torsional moments induced by eccentricity in the building plan can lead to significant damage in the building that can result in the global collapse of the entire structure. The upper storeys of the structure behaved close to a rigid body throughout the testing. A numerical collapse study of the structure conducted by Pang and Ziaei (2012) predicted that the collapse would occur between 11% - 16% interstorey drift. Further research is to be conducted to improve the numerical model.

2.3 Detailed Shear Wall Models

Numerical models have been developed to predict the behaviour of specific wood shear wall assemblies. The global behaviour of a light-frame wood buildings is very detailed dependant. By modeling each component of a wall assembly (i.e. openings, hold-downs, nailing schedule, panel orientation) the lateral behaviour and collapse mechanisms for specific engineered and non-engineered (conventional) shear wall assemblies can be estimated without needing to set up a laboratory testing program.

Lumped-parameter shear wall models use single-degree-of-freedom (SDOF) nonlinear shear springs to capture the global behaviour of the wall. The rule-based material models used to describe the behaviour for wood shear wall assemblies are defined in Section 1.3.3: Material Hysteretic Spring Models. The SDOF lumped-parameter models are computationally efficient and therefore can be easily implemented into global models. The models, however, do not capture the failure mechanisms of the wall and can not consider combined effects of vertical (gravity and uplift) and horizontal loading.

Detailed FEM models have also been developed. These models tend to be computationally intensive and therefore have limited application in practice and in global models. Several FEM models applying different principals and simplifications have been developed and proposed. There, however, has been little consensus between the independent studies on the methods used to model light-frame wood connections, shear walls or diaphragms. For instance, a diaphragm model by Itani and Cheung (1984) used beam and plane-stress elements to model the framing and sheathing panels. “Smeared” nonlinear springs were used to model the panel-to-frame connections. The smeared connection approach involves simplifying a nail line by evaluating the response along a panel at the Guassian integration points. Discrete nails were not modeled, therefore the failure mechanism and failure sequence of the nails, missing nails/nail spacing changes were not considered. Dolan (1989) developed an FEM model using beam elements to represent the framing members, plate elements for the panels, bilinear springs for the connections between the framing members and the gap-contact between sheathing panels, as well as discrete zero-length joint and sheared-connector elements for the panel-to-framing connections. Pang et al. (2012) developed an FEM model (as part of the M-CASHEW2 analysis program) using a correlational formulation and large displacement theory. Nodal condensation using shape functions for the framing and panels elements was used to decrease the computational expense of analysis. The framing and sheathing panels were assumed to be linear and elastic; the connectors were modeled using non-linear hysteretic springs. The model is very flexible and can accurately predict the collapse characteristics and lateral behaviour of various shear wall configurations (engineered and non-engineered), opening configurations and nailing schedules. The M-CASHEW2 model is currently considered state-of-the-art.

Numerical FEM models have also been developed using commercially available analysis programs. ANSYS, ABAQUS and SAP2000 have been used to model light frame wood walls by a number of researchers ((Asiz, Chui, Smith, & Zhou, 2009; Kasal & Leichti, Nonlinear finite-element model for light-frame stud walls, 1992; Xu, 2009; Li & Ellingwood, 2007; Blasetti, Hoffman, & Dinehart, 2008). In general 3D beam elements are used to model the framing members, shell elements are used to model the sheathing panels and two-node zero-length joint elements are used for the nail connections.

The CASHEW (Cyclic Analysis of Wood Shear Walls) program was developed as part of the CUREE-Caltech wood-frame project (Folz & Filiatrault, 2001). The program implements several simplifications to reduce the computational cost of the analysis. The framing is assumed to be pin-jointed rigid elements that can only deform into a parallelogram, framing members are modeled as pin-ended rigid elements without lateral stiffness and the sill plate is assumed to be rigidly attached to the foundation. The separation between the framing members is ignored. This program can give reasonable predictions for standard, engineered shear walls with proper anchorage detailing (Pang W. , Rosowsky, Ellingwood, & Wang, 2009). The program is not appropriate for collapse analysis

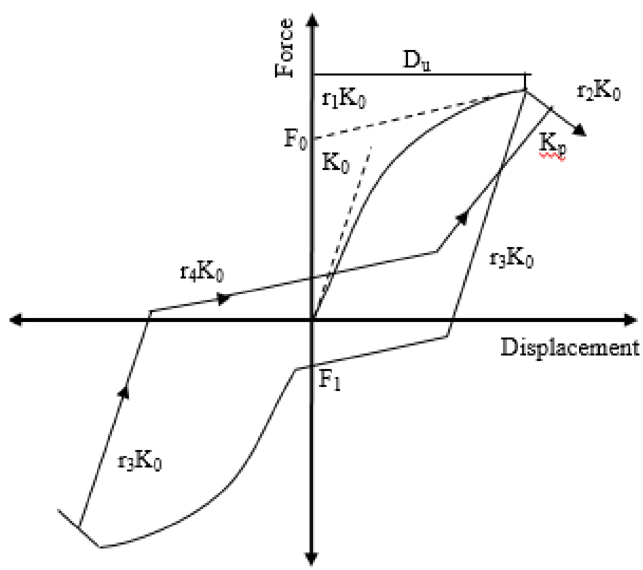
2.4 Material Hysteretic Spring Models

Material hysteretic models have been developed to represent the shear behavior of wall assemblies used in light-frame wood structures. These models can represent the full wall assemblies down to a single nail. The global and wall numerical modeling programs such as SAWS, SAPWood, Timber3D, CASHEW and

M-CASHEW2 have the material models integrated into the software. The details of the Modified Steward Hysteretic Model (MSTEW), the Evolutionary Parameter Hysteretic Model (EPHM), and the Residual Strength Hysteric Model (RESST) has been described.

2.4.1 Modified Steward Hysteretic Model (MSTEW/CUREE Model)

The MSTEW model, as shown in Figure 3, is a well-established hysteresis model developed by Folz and Filiatrault (2002) for the CUREE project. The hysteresis model was based on the Foschi (1974) single degree of freedom system model of a wood shear wall. The model was defined by with 10 parameters that describes the exponential backbone curve, and the linear loading/unloading paths. The MSTEW model can be adapted for variety of materials, such as OSB/plywood, gypsum wall board, stucco and horizontal shiplap.



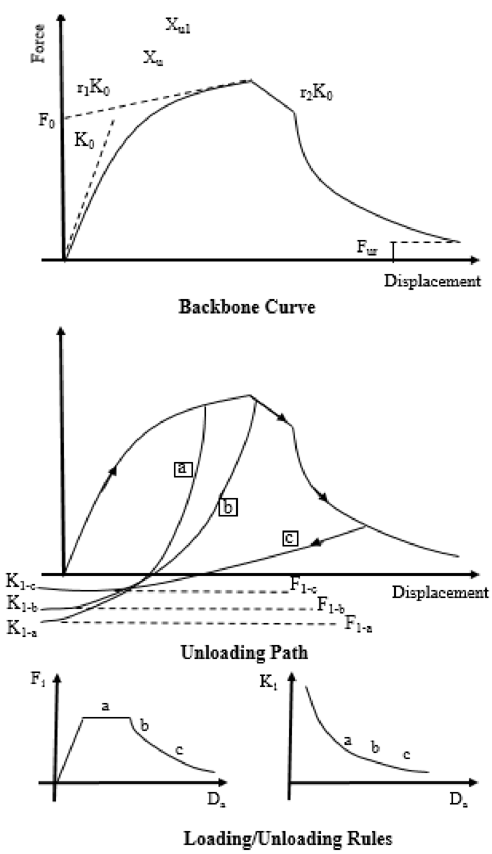
K_0	Initial stiffness
F_0	Resistance force parameter of the backbone
F_1	Pinching residual resistance force
r_1	Ratio of stiffness parameter of the ascending backbone to K_0
r_2	Ratio of stiffness parameter of degrading backbone to K_0
r_3	Ratio of the unloading path stiffness to K_0
r_4	Ratio of the pinching load path stiffness to K_0
D_u	Drift corresponding to the maximum restoring force
α	Stiffness degradation parameter
β	Strength degradation parameter

Figure 3: Loading paths and parameters of MSTEW material model

It should be noted that the MSTEW models uses static parameters, therefore has limited accuracy at large drift levels where strength and stiffness degradation can be significant. The model tends to overestimate energy dissipation which would lead to an under prediction of the deformation and assumes a linearly decaying backbone response after the shear wall reaches its peak capacity, whereas a nonlinear curve would better represent experimental data.

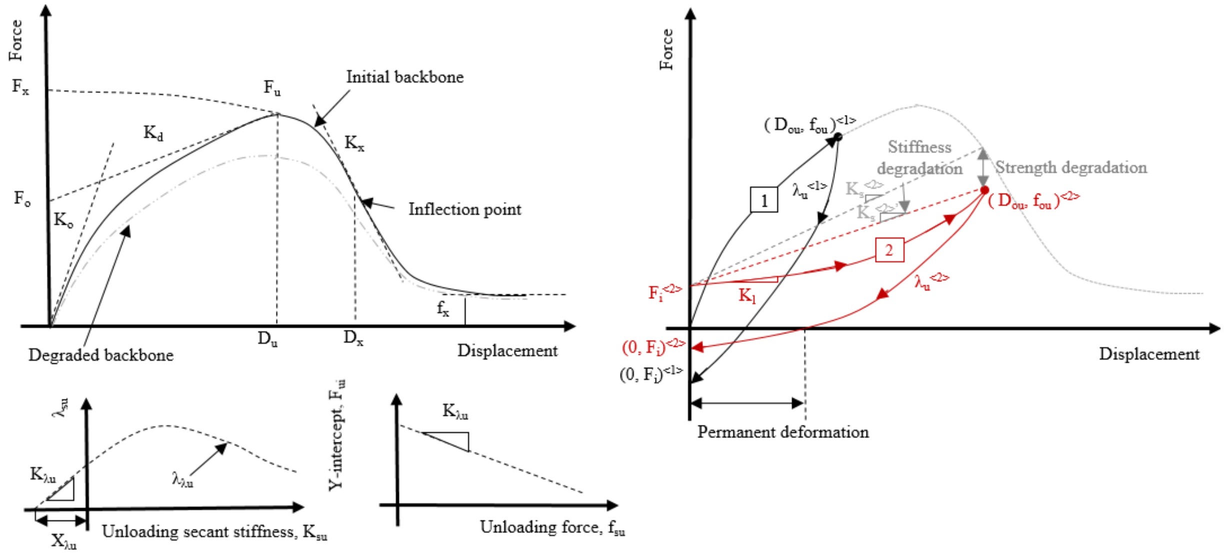
2.4.2 Evolutionary Parameter Hysteretic Model

The evolutionary parameter hysteretic model, EPHM, was developed as an extension of the MSTEW material model to represent a non-linear SDOF system for a wood shear wall. The model defines non-linear loading and unloading paths, as well as evolutionary parameters that can capture energy dissipation, as well as in-cycle and out-of-cycle stiffness and strength degradation. EPHM gives an improved prediction for elastic and inelastic responses over the static MSTEW model and gives a better estimation of the fragility curves used to develop drift-based failure probabilities for performance based design, as well as (Pang W. C., Rosowsky, Pei, & van de Lindt, 2007). Hysteretic model consists of four main components: (i) backbone curve; (ii) tracking indices; (iii) loading rules/paths; (iv) evolutionary parameters (degradation rules). Variations of the EPHM are described in detail by Pei (2012) and Pang et al. (2007). A summary of the EPHM hysteretic model by Pei and van de Lindt (2012) and Pang et al. (2007) is given in Figure 4 and Figure 5, respectively.



K_0	Initial stiffness
F_0	Resistance force parameter of the backbone
r_1	Stiffness ratio parameter of the ascending backbone
X_u	Displacement corresponding to max. restoring force
r_2	Stiffness ratio parameter of degrading backbone
X_{u1}	Displacement corresponding to end of linearly degrading backbone
p_1	Exponential degrading rate parameter of the backbone
F_{1m}	Max. value of residual pinching force
F_{1r}	Min. value of residual pinching force in severe damage
D_{F1a}	Damage index associated with pinching force, F_I
D_{F1b}	Damage index associated with pinching force, F_I
p_{F1}	Exponential degrading rate parameter associated with pinching force, F_I
p_{r4}	Exponential degrading rate parameter associated with K_I degrading function
r_{4r}	Ratio of residual K_1 to initial stiffness
β	Strength degradation parameter
F_{ur}	Residual resistance force of backbone at severe damage state

Figure 4: Loading paths/parameters for EPHM 16 parameter material hysteresis by Pei and van de Lindt (2010)

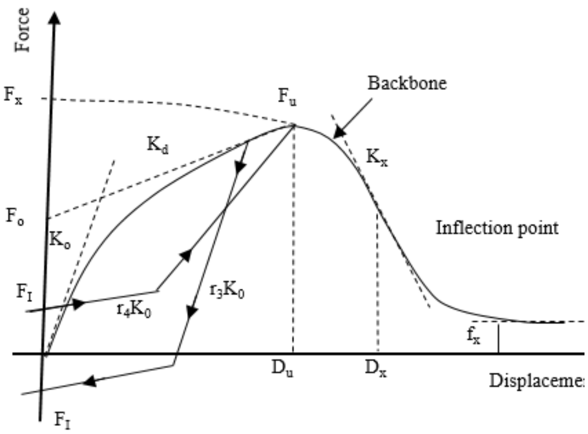


Initial Ascending Backbone	K_o	Initial tangent stiffness of the backbone curve
	K_d	Degraded stiffness
	F_o	Resistance force parameter of the backbone
Initial Descending Backbone	D_x	Point of inflection of descending backbone
	K_x	Tangent stiffness of the descending backbone at D_x
	F_x	Upper force asymptote of descending backbone
	f_x	Lower force asymptote of descending backbone
Maximum Point – Initial Backbone	D_u	Displacement at F_u
	F_u	Maximum load-carrying capacity
Degraded Ascending Backbone	K_d	Ascending backbone stiffness
	F_o	Resistance force parameter of the backbone
Degraded Ascending Backbone	D_{xd}	Point of inflection of descending degraded backbone
	K_{xd}	Tangent stiffness of the descending degraded backbone at D_{xd}
	F_{xd}	Upper force asymptote of descending backbone
	f_{xd}	Lower force asymptote of descending backbone
Maximum Point – Degraded Backbone	D_{ud}	Displacement at F_{ud}
	F_{ud}	Maximum load-carrying capacity of degraded backbone
Unloading Curve	K_{fi}	Local degradation parameter associated with force intercept
	$K_{\lambda u}$	Local degradation parameter
	$\lambda_{\lambda u}$	Local degradation parameter associated with decay rate
	$x_{\lambda u}$	Local degradation parameter
Internal Model Parameters	λ_u	Evolutionary shape parameter
	f_{ou}	Initial unloading force
	D_{ou}	Initial unloading force drift
	F_i	Force intercept parameter
	K_l	Initial tangent stiffness of the loading function
	λ_l	Evolutionary loading parameter

Figure 5: Loading paths and parameters of EPHM 17 parameter material model by Pang et al. (2007)

2.4.3 Residual Strength Hysteretic Model

The residual strength hysteric model (RESST) was developed based on the combination of the MSTEW model and the EPHM model by W. Pang. It is a 12 parameter model with a defined backbone curve based on the EPHM model and linear loading paths based on the MSTEW model.



K_o	Initial tangent stiffness of the backbone curve
r_1	Ratio of the ascending backbone stiffness and K_o ($r_1=K_d/K_o$)
r_2	Ratio of the tangent stiffness of the descending degraded backbone and K_o ($r_2=K_x/K_o$)
r_3	Ratio of the unloading path stiffness to K_o
r_4	Ratio of the pinching load path stiffness to K_o
F_x	Upper force asymptote of descending backbone
f_1	Ratio of the resistance force parameter of backbone and F_x ($f_1=F_o/F_x$)
f_2	Ratio of the force intercept parameter and F_x ($f_1=F_i/F_x$)
f_3	Ratio of the lower force asymptote of descending backbone and F_x ($f_3=f_x/F_x$)
D_x	Point of inflection of descending backbone
α	Stiffness degradation parameter
β	Strength degradation parameter

Figure 6: Loading paths and parameters of RESST material hysteresis model by Pang et al. (2007)

2.5 Summary

The development of numerical models for light-frame wood structures has been described in detail. The Timber3D and M-CASHEW2 analysis programs can accurately model the structure at high drift levels imminent of structural collapse. The models apply large-displacement theory and include P-delta effect. The key objective of the research is to examine the ability for 3D nonlinear modeling to predict the seismic performance and of the structure. To achieve this a Timber3D model was validated in Chapter 3 over a wide range of ground motion intensities: from serviceability to collapse.

Chapter 3: Global Numerical Model Validation

3.1 Introduction

The available state-of-the-art numerical modelling methods for light-frame wood structures were discussed in Chapter 2 to determine what numerical programs would most appropriate for the study. The Timber3D and M-CASHEW2 program can model the structure from near elastic behaviour to imminent collapse. In Chapter 3 the global model for a typical light-frame wood construction was calibrated with experimental results. Previous work conducted at the UBC have included a series of shake table tests of a full-scale two-story light-frame wood house. Construction types with different sheathing configurations, including Blocked OSB, Unblocked OSB, Shiplap and Stucco/Blocked OSB were tested with ground motions scaled from low to high intensities. Material hysteresis models were defined based on monotonic, cyclic and dynamic testing of wood shear walls, as well as recommendations from technical review committees. A sensitivity study to investigate the use of simplifications to account for combined sheathing configurations, wall openings, nailing patterns and holdown/anchorage details was completed.

3.2 Full Scale Testing

The University of British Columbia (UBC) conducted a shake table test with two-storey full-scale light-frame timber houses as part of the Earthquake-99 Test Program. A variety of sheathing configurations and detailing was used to represent common construction practices in decades prior to and after the implementation of seismic guidelines for light-frame wood structures. The ground motions were selected and scaled to represent the seismicity in the Lower Mainland of British Columbia (Vancouver and the surrounding area). The testing program and a description of the test specimens have been summarized in Table 1. The floor plans for the first and second floor are shown in Figure 7. The interior walls are sheathed with gypsum wall boards (GWB). Detailed information on the shake table testing can be found in TBG (2002), Kharrazi (2001), Ventura et al. (2002) and Kharrazi et al. (2002).

Table 1: Summary of Shake Table Testing Program

No.	Earthquake	Test Description
9	Sherman Oaks	Type 2: OSB walls (Engineered).
10	Nahanni	Type 1: OSB walls, hold-downs and stucco (Engineered).
11	Nahanni	Type 1: OSB walls, hold-downs & rain-screen stucco (Engineered).
12	Landers	Type 3: OSB walls (Non-Engineered)
13	Kobe	Type 4: Horizontal boards w/o stucco, hold-downs or roof blocking
14	Landers	Type 2: OSB walls (Engineered).
15	Llayllay (scaled 175%)	Type 2: OSB walls (Engineered).
16	Llayllay (scaled 175%)	Type 3: OSB walls (Non-Engineered)

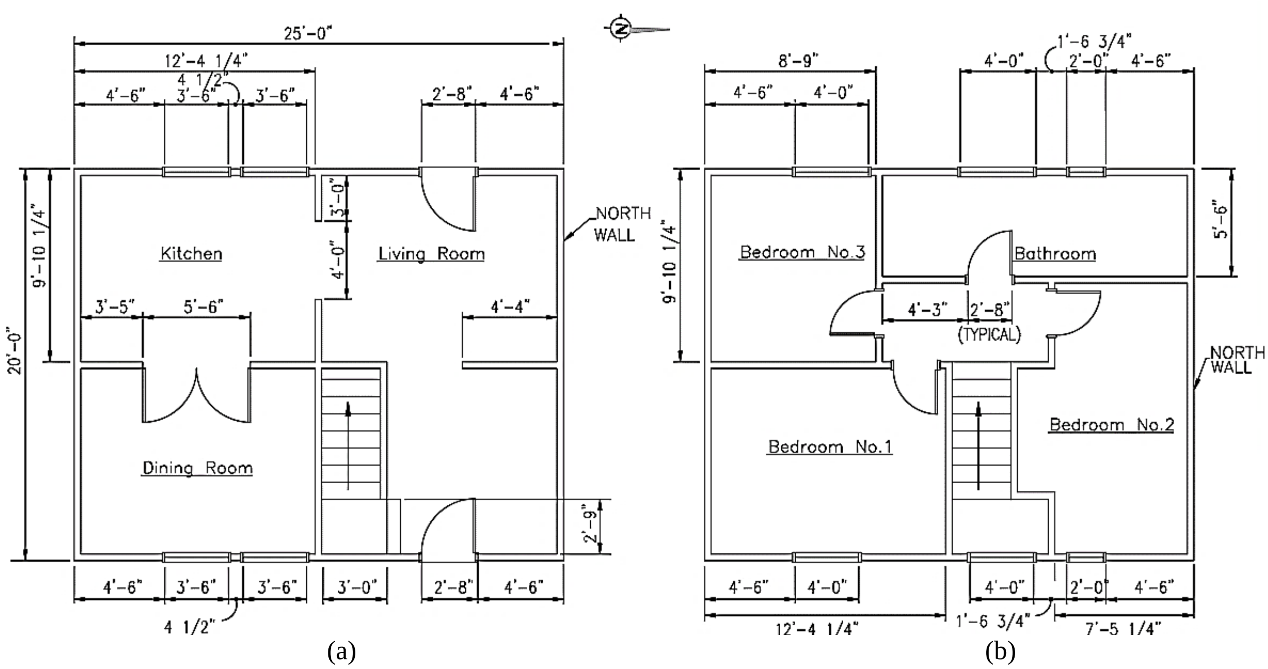


Figure 7: Details of full-scale house (a) first floor (b) second floor

(a)

(b)

Figure 8: Photograph of (a) Linear shake table, (b) Type 2 full-scale house

3.3 Numerical Model

The current state-of-the-art three-dimensional (3D) numerical modelling software developed by Pang et al. (2012) as part of the NEES-Soft project was used to model the two-storey light-frame wood house. The in-plane and out-of-plane roof and floor diaphragm flexibility is characterized with 2-node, 12-DOF (three translational and three rotational DOF at each node) frame elements. These frame elements can capture tension, compression, torsion and bending effects, as well as geometric nonlinearity. The end studs are also modeled with the frame elements; the intermediate studs are not explicitly modelled to reduce the computational time. The first-floor studs have a fixed ground boundary condition. The lateral stiffness of the wood shear walls is modeled with 2-node, 6-DOF, zero-length, link elements. These link elements were defined with the RESST and CUREE wall hysteresis models. The direct superposition of the lateral strength of the various sheathing layers was applied where each layer was modeled separately as a shear spring. Please refer to Appendix C for more information on combined sheathed walls. The parameters stiffness and strength parameters were assumed to be linearly proportional to the height and length of the wall. The opening factor (see Appendix E) recommended in the FEMA P-807 documents were used to account for the windows and doors. This factor was developed based on experimental results and a review process of perforated walls by the American Forest and Paper Association, Special Design Provisions for Wind and Seismic (AF&PA SDPWS, 2008), Sugiyama, 1981, Dolan and Johnson, 1997a, 1997b; and APA, 2005.

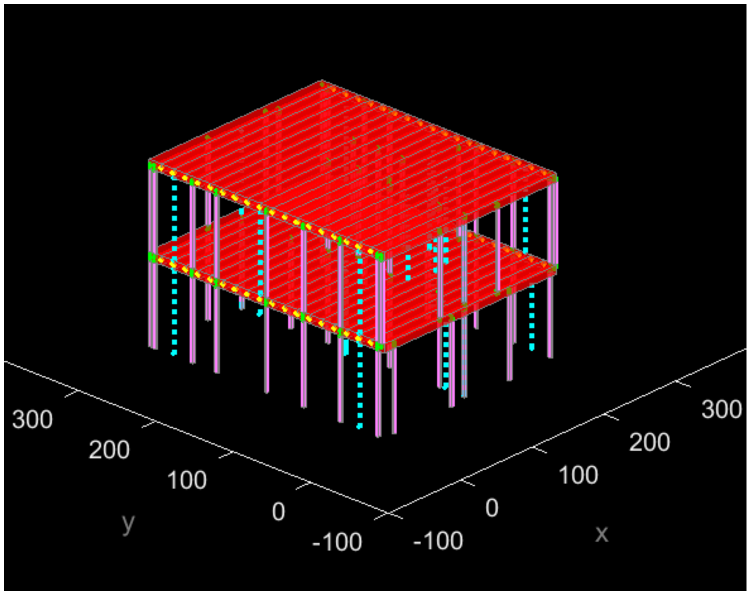


Figure 9: Modelling light-frame house with Timber 3D

3.4 Wall Hysteresis Models

The behavior of the shear walls was modeled with the RESST or MSTEW material hysteresis models, as given in Table 2.

Table 2: Wall Hysteresis Parameters (per 8ft. wall)

RESST Material Model												
	K_{oi} kN/mm kip/in.	**r_1**	**r_2**	**r_3**	**r_4**	**F_x kN kip**	**f_1**	**f_2**	**f_3**	**D_x**	**α**	**β**
Gypsum Wall Board	0.89 5.1	0.07	0.46	1.01	0.010	5.87 1.32	3.02 0.68	0.18	0.3	82 3.23	0.80	1.10
Engineered Blocked Wood Panel	1.57 9.0	0.01	-0.23	1.01	0.030	41.2 9.26	4.31 0.97	0.13	0.3	121 4.77	0.76	1.15
Unblocked Wood Panel	1.05 6.0	0.06	-0.12	1.01	0.015	15.8 3.55	3.38 0.76	0.11	0.8	99 3.90	0.80	1.1
New Stucco Construction	2.63 15.0	0.13	-0.05	1.45	0.005	40.2 9.04	1.97 0.442	0.09	0.1	119 4.70	0.38	1.09
MSTEW Material Model												
	K_0 kN/mm kip/in.	**r_1**	**r_2**	**r_3**	**r_4**	**F_O kN kip**	**F_I kN kip**	**D_u mm in.**	**α**	**B**		
Horizontal Siding	0.21 1.18	0.1	-0.95	1.01	0.035	1.6 0.36	0.6 0.136	241 9.5	0.45	27 1.06		

The gypsum wall parameters were based on data obtained from the tests conducted as part of the CUREE project, the cyclic wall tests from the University of British Columbia as part of the testing program for the School Seismic Retrofit Guidelines (EERF, 2009), tests performed by Bahmani and van de Lindt (2016), as well as the recommendations from the FEMA P-807 and the technical committee review for the on-going ATC-116 project.

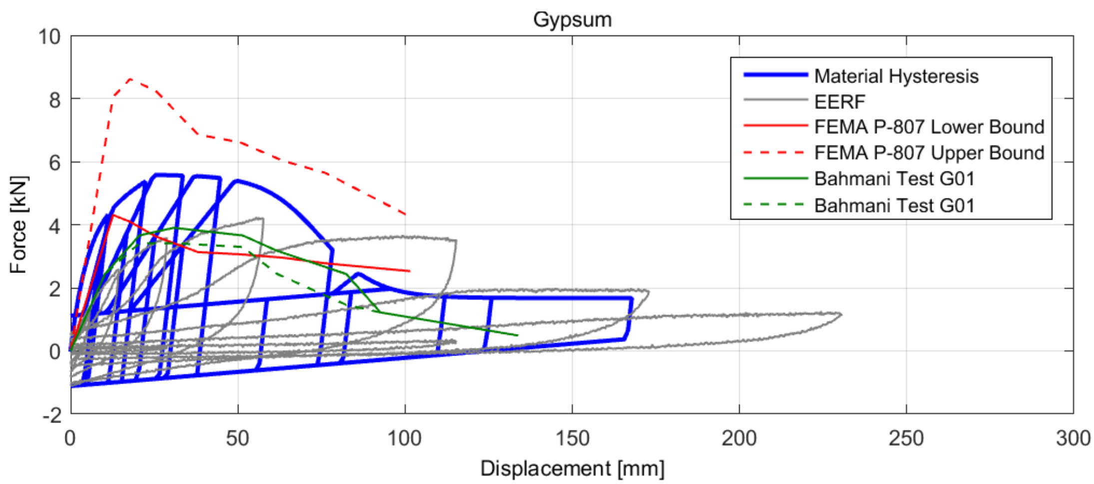


Figure 10: Gypsum Material Model compared to experimental data

The blocked engineered shear wall hysteretic parameters are based on data from the cyclic wall tests from UBC as part of the testing program for the School Seismic Retrofit Guidelines (EERF, 2009), tests performed by Bahmani and van de Lindt (2016), recommendations from the FEMA-P807, the technical committee review for the on-going ATC-116 project and by Bahmani et al. (2014) as part of the NEES-soft project. The blocked shear wall prototype is for walls with proper blocked and anchorage with hold-down devices. The sheathing nails should be spaced at a minimal of 100mm (4") and 300mm (12") for the panel edges and interior, respectively. Figure 11 shows the experimental data compared to the material hysteresis for the blocked engineered OSB prototype.

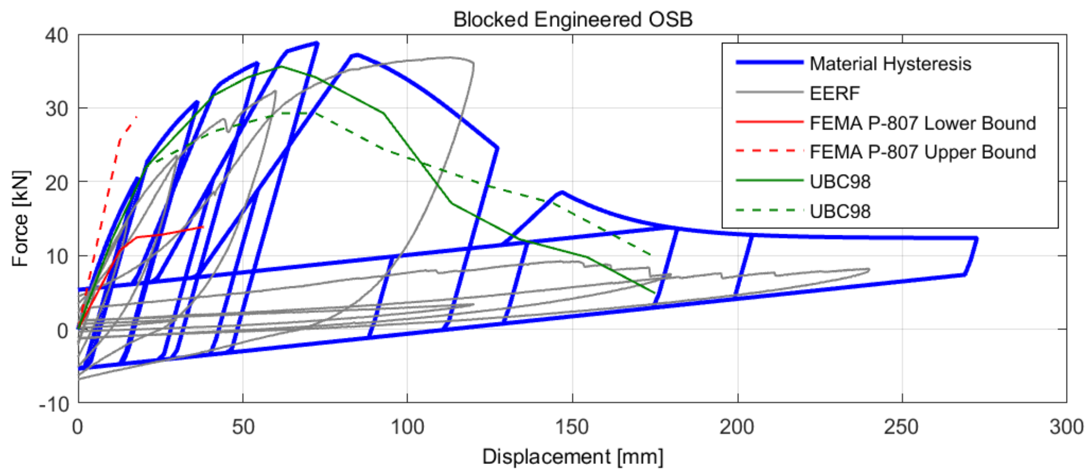


Figure 11: Blocked Engineered Wood Shear Wall Material Model compared to experimental data

The unblocked wood shear walls are based on the wall test data by the University of British Columbia in the EERF (2009) and UBC98 projects. This type of wall system is typically OSB with 8d common sheathing nails spaced at 6" (150mm) o/s at the panel edges and 12" (300mm) o/s in the interior. Figure 12 shows the experimental hysteresis for the EERF tests and the UBC98 backbone curves compared to the RESST material hysteresis for the unblocked shear wall prototype. The results are for a wall segment 2400mm (8ft.) in length.

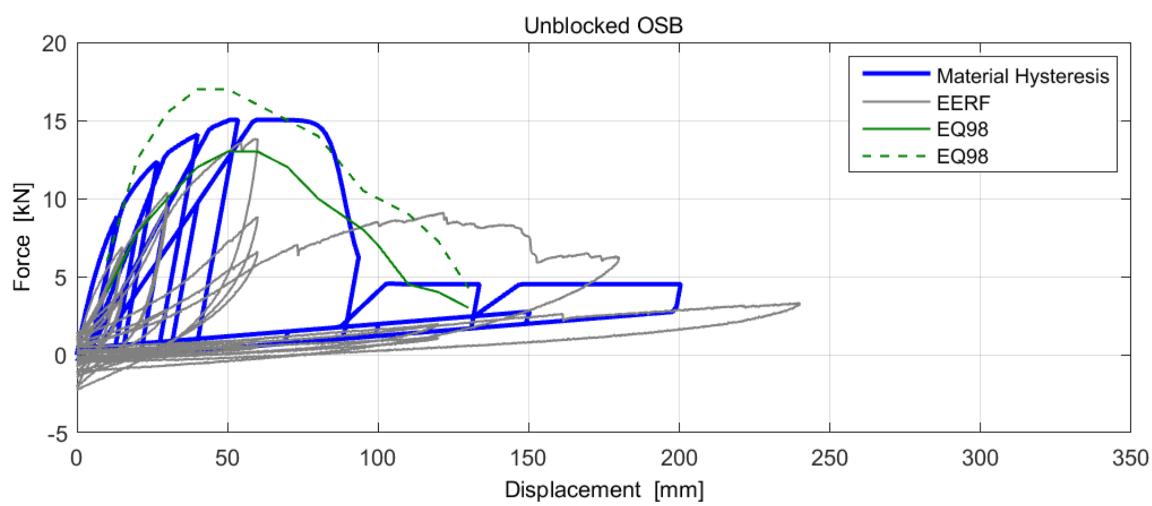


Figure 12: Unblocked shear wall model compared to experimental data

The horizontal wood siding model was based on wall test data conducted in the 1950s in the Forest Products Laboratory, the cyclic wall tests from UBC as part of the testing program for the School Seismic Retrofit Guidelines (EERF, 2009), tests performed by Bahmani and van de Lindt (2016), as well as the recommendations from FEMA P-807 and the ATC-116 project. The wood siding was observed to have very high ductility and were stable at high drift levels (>8% drift).

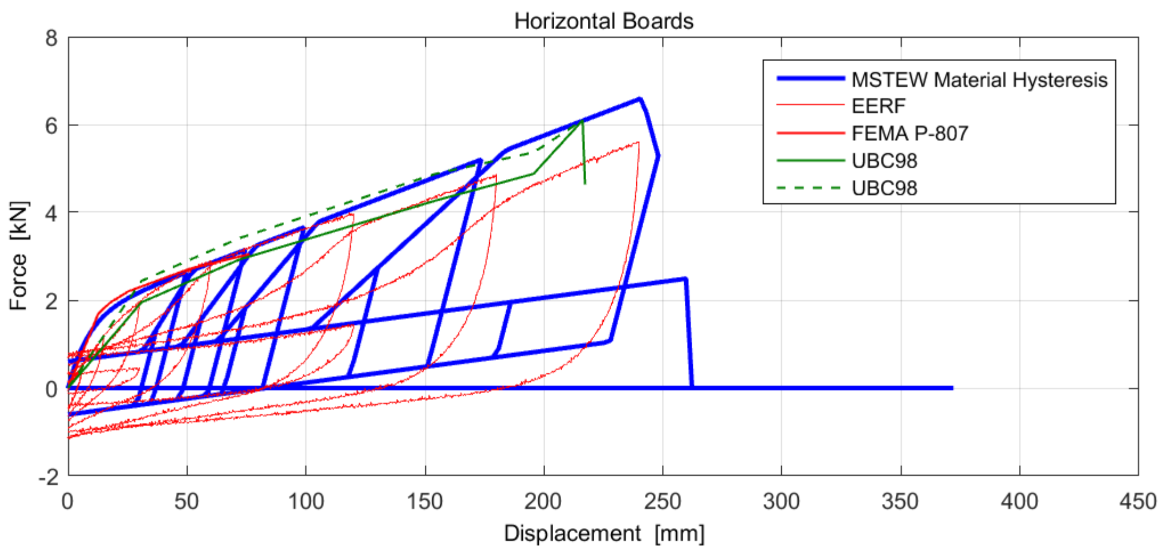


Figure 13: Horizontal Board Material Model compared to experimental data

The stucco external finishing was based on the recommendations of the technical committee review for the on-going ATC-116 project, stucco tests performed at the University of British Columbia as part of the EQ-99 project and test performed by Sofali (2008). This material model was developed to represent new stucco construction. New stucco practices have been documented to be significantly increase the strength, stiffness and ductility of the wall systems when tested.

In the EQ-99 project eighteen (18) stucco walls were tested to determine strength, ductility and earthquake damage estimates of the stucco walls, as well as investigate the influence of the rainscreen cavity, strapping materials, strapping fasteners, and types of lath and lath fasteners. Cyclic quasi-static tests were used and the tests were stopped if the wall had effectively failed or reached the last loading cycle at 8% drift. The tests showed that stucco with and without rainscreen had very good cyclic performance. The peak resistance of the specimens occurred between 2.5% and 4% drift and the specimens show residual capacity over 6% drift.

Sofali (2008) completed tests of stucco shearwalls with a special shear connector. Shearlocks were developed by Adebar et al. (US Patent No. 6668501, 2003) to provide a connection of the stucco to the wood frame that has high strength, stiffness, and significant ductility. The shearlocks are designed to act as a ductile “fuse” and significantly increase the overall ductility of the wood shear wall system. The shear locks were spaced at 6in. along the perimeter of the wall. Tests were also conducted on 8 ft. by 8 ft. stucco wall panels. It should be noted that this material model is not appropriate for older, existing stucco.

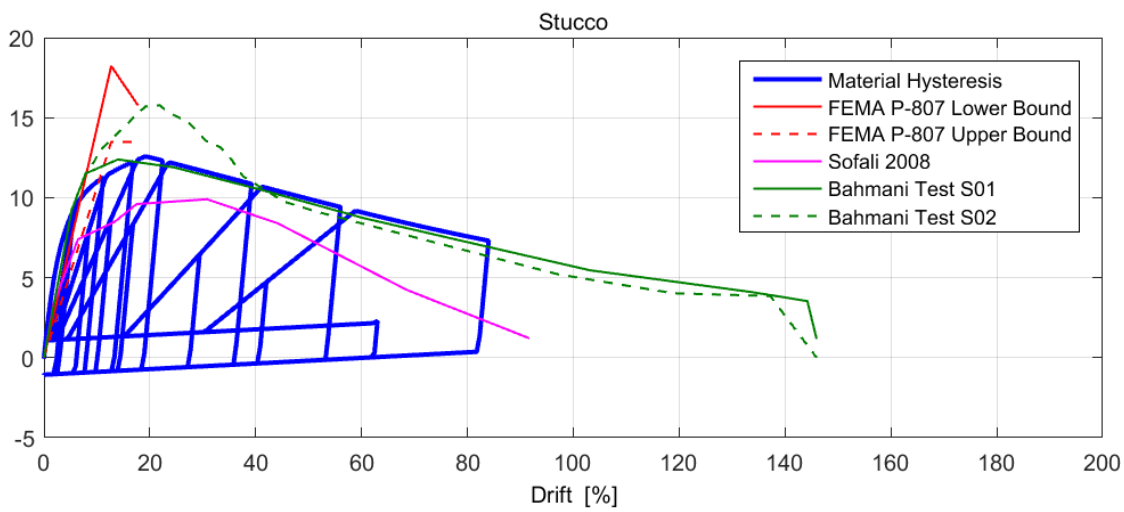


Figure 14: New stucco construction shear wall material model compared to experimental data (8ft. wall)

3.5 Comparison of Numerical Prediction and Experimental Results

The period for the first mode of vibration for the model and the measured structure are given in Table 3, Table 4, Table 5 and Table 6. A comparison between the time history response of the model and experimental results are shown in the following plots for Shake Table Test 9 - 16, as shown in Figure 15, Figure 16, Figure 17, and Figure 18. The measured and numerical absolute base shear is summarized in Table 7. A summary comparing the absolute maximum drift for the numerical model and experiment is given in Figure 19.

Table 3: Measured and Model Natural Period of Prototype 1 (Stucco, Blocked OSB, hold-downs)

Test Number	Measured T_n	Model T_n
Test 10	0.25 sec	0.26 sec.
Test 11	0.26 sec	0.26 sec.

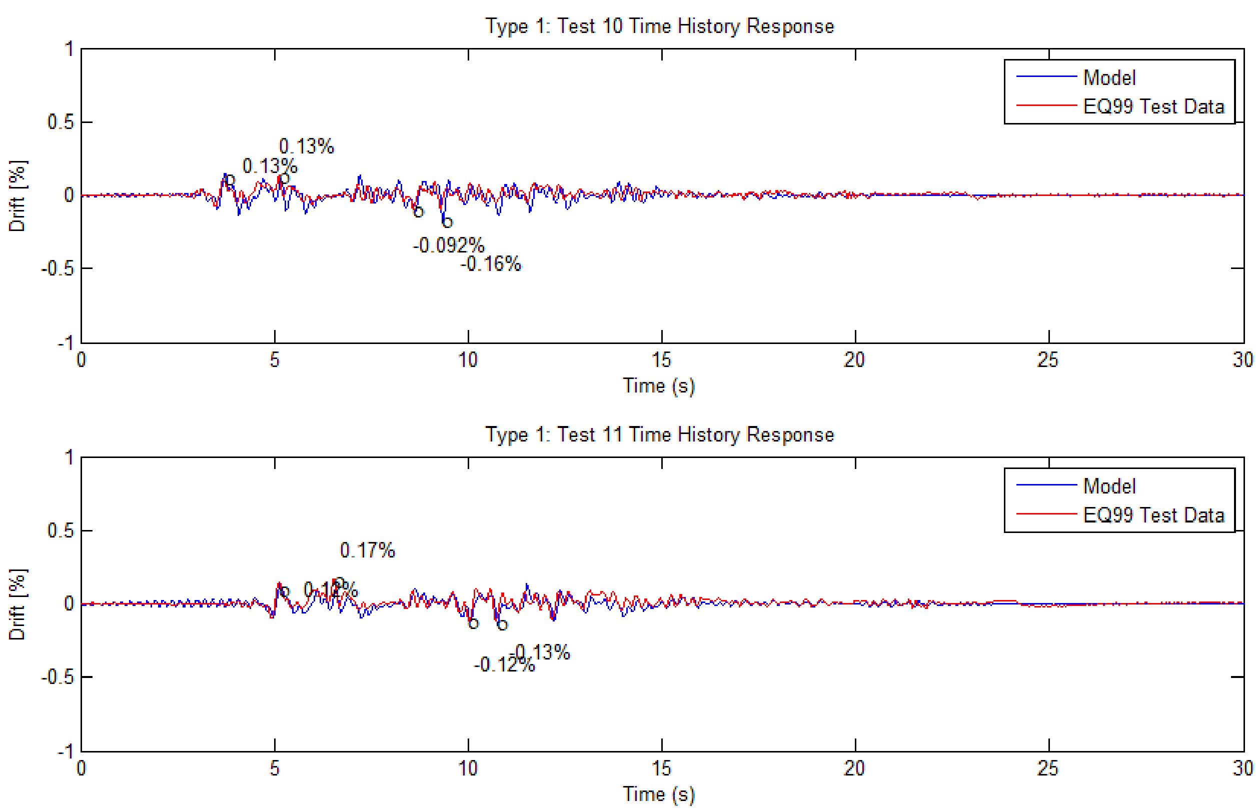


Figure 15: Comparison of experimental and numerical results for Prototype 1 (Stucco, Blocked OSB, hold-downs)

Table 4: Measured and Model Natural Period of Prototype 2 (Blocked OSB, hold-downs)

Test Number	Measured T_n	Model T_n
Test 9	0.29 sec.	0.32 sec.
Test 14	0.32 sec.	0.32 sec.
Test 15	0.36 sec.	0.32 sec.

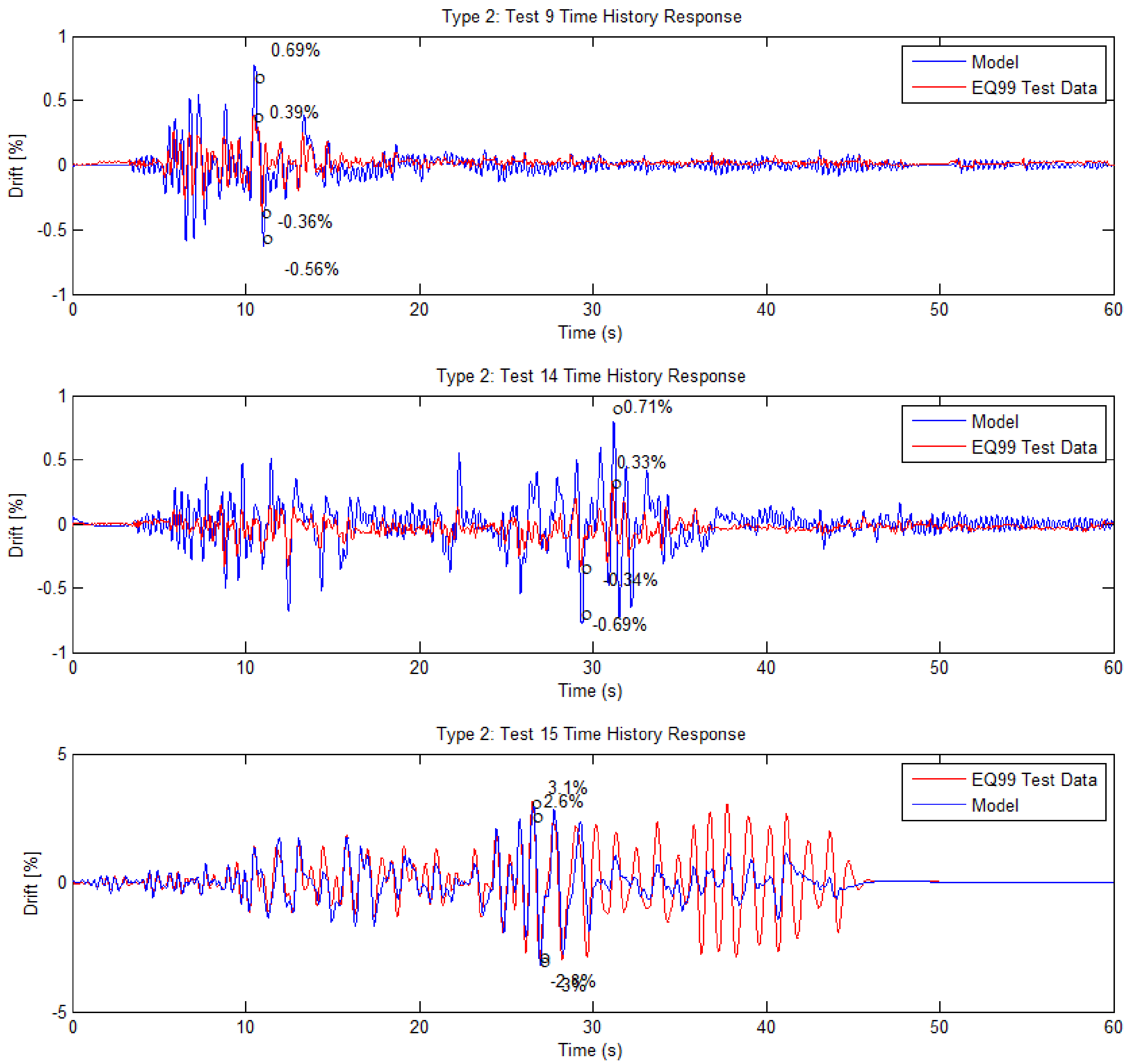


Figure 16: Comparison of experimental and numerical results for Prototype 2 (Blocked OSB, hold-downs)

Table 5: Measured and Model Natural Period of Prototype 3 (Unblocked OSB)

Test Number	Measured T_n	Model T_n
Test 12	0.36 sec	0.33 sec.
Test 16	0.38 sec.	0.33 sec.

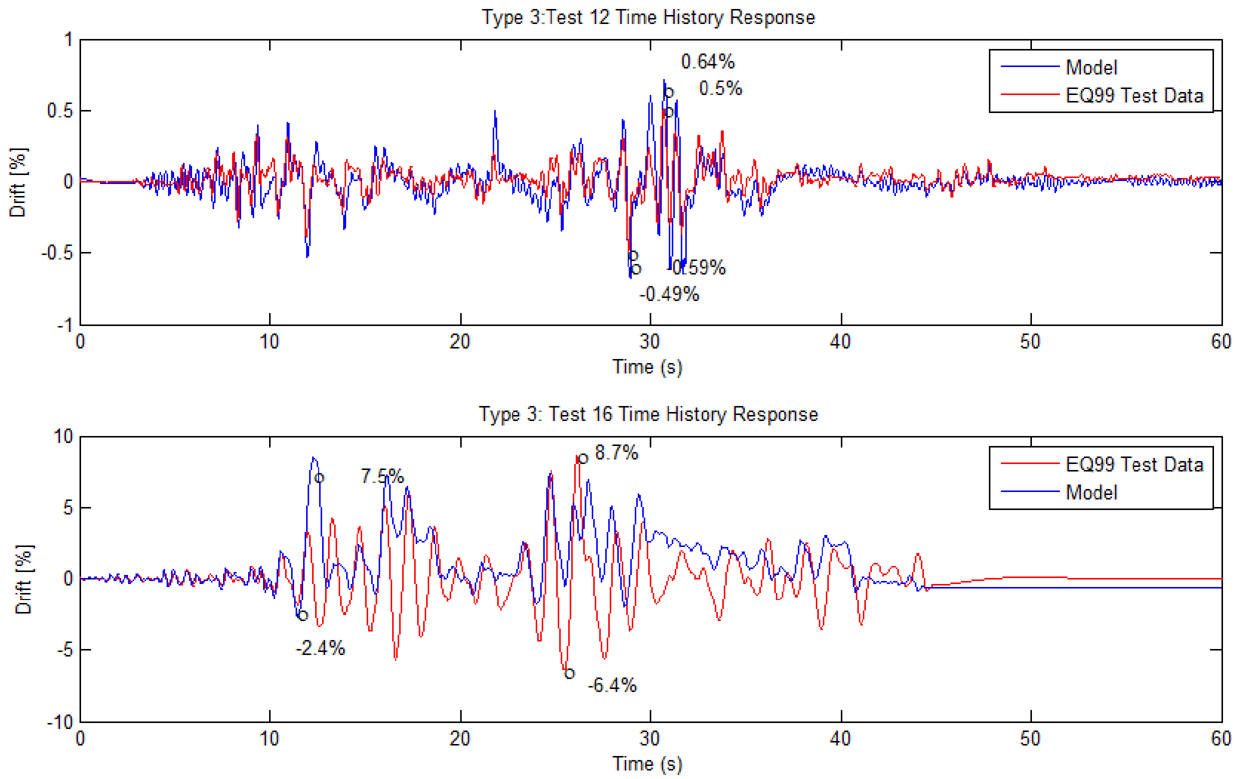


Figure 17: Comparison of experimental and numerical results for Prototype 3 (Unblocked OSB)

Table 6: Measured and Model Natural Period of Prototype 4 (Horizontal Boards)

Test Number	Measured T_n	Model T_n
Test 13	0.37 sec	0.40 sec.

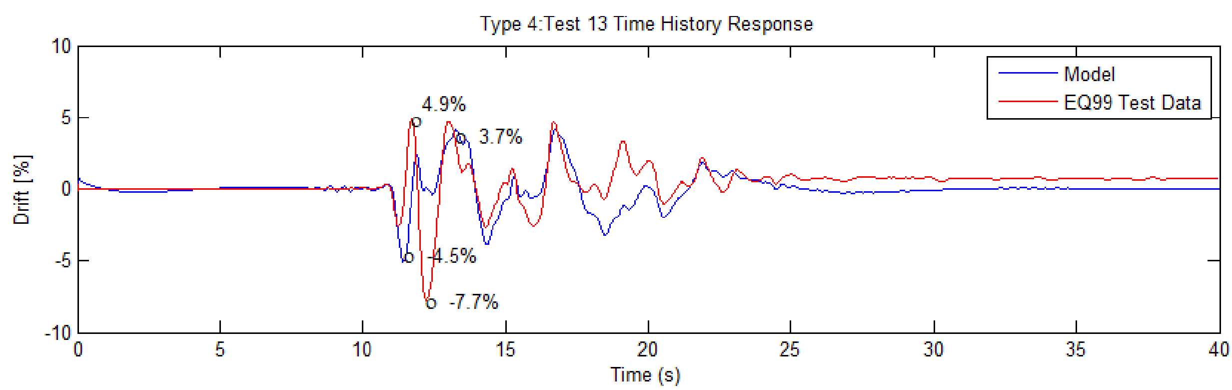


Figure 18: Comparison of experimental and numerical results for Prototype 4 (Horizontal Boards)

Table 7: Measured and numerical absolute base shear

Test Number	Measured Maximum Absolute Base Shear	Numerical Maximum Absolute Base Shear
Test 9	67.4 kN	76.6 kN
Test 10	50.7 kN	74.9 kN
Test 11	68.5 kN	68.6 kN
Test 12	62.3 kN	65.3 kN
Test 13	108.9 kN	42.4 kN
Test 14	73.4 kN	92.6 kN
Test 15	159.0 kN	115 kN
Test 16	110 kN	76.1 kN

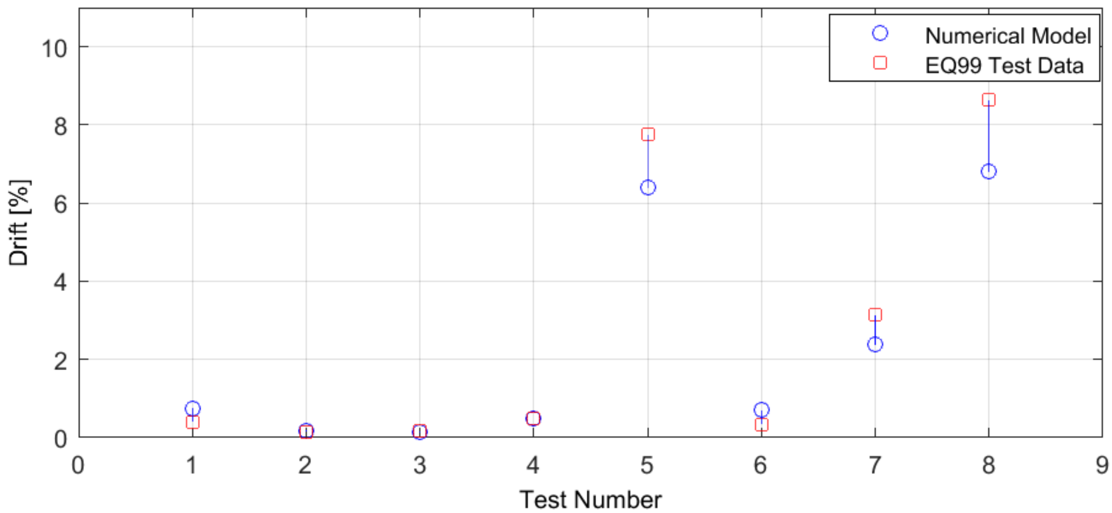


Figure 19: Comparison of absolute maximum drift of numerical and experimental results

3.6 Summary

The Timber3D models could predict the absolute peak drift response of the different housing configurations with considerable accuracy. Although the peak drifts matched well, the time history response was not accurately predicted over the full duration. For instance, in Test 15 the model seemed to have too much damping after significant deterioration. Furthermore, in Test 16 maximum drifts occurred at different times in the response. It was challenging to calibrate the models to the full range of responses observed in the experimental testing program. The same modeling methods and hysteretic models were used for the different construction types and response intensities. Further work could be completed to have a better calibration of the model to the time-history response, however for the purposes of determining the maximum experienced drift the modelling method works well.

Chapter 4: Prediction of Full-Scale Test

4.1 Introduction

The accuracy of the global numerical modelling method, shear wall parameters and detail simplifications applied in Chapter 3 to predict the seismic response of light-frame wood structures was further investigated in an additional full-scale testing and numerical modeling study of a typical light-frame wood classroom. Global models of the light-frame wood classroom were created using Timber3D to make a blind prediction of the shake-table response of a single storey light-frame wood structure. An addition M-CASHEW2 wall model was created of the test structure to investigate the effect of the higher level of detailing in the modeling accuracy. The time-history analysis was directly compared to the experiential shake table results to validate the models. Further analysis to determine a validated method to account for openings and to considered the effect of ground motion duration was completed.

4.2 Test Specimen

As part of the Seismic Retrofit project, a full-scale one-storey wood frame classroom was tested on the linear shake table at UBC EERF facility. This testing was part of the BC School Seismic Retrofit Program for limited long-duration testing, as well as for developing the post-earthquake evaluation methodology and inspection techniques. The testing was coordinated by: Martin Turek, Graham Taylor, and Mehrtash Motamedi. The classroom had a plan dimension of 7.62m x 6.096m (300”x200”). The sheathing nails on the blocked shear wall segment were 8d common nails spaced at 100mm (4”) on the sheathing panel edges and 150mm (6”) on the interior studs. The unblocked wall sheathing nails were 8d common nails spaced at 6in. on the sheathing panel edges and 12in. on the interior studs. The studs were 2x4 Douglas Fir Lumber and the sheathing was 9.5mm plywood panels. Six (6) steel inertia plates (3600 kg each plate) and HSS sections were loaded on the specimen to simulate a second school storey. The total seismic weight was 250kN (56kips). A schematic of the north and south elevation is shown in Figure 20. An image of the structure is shown in Figure 21.

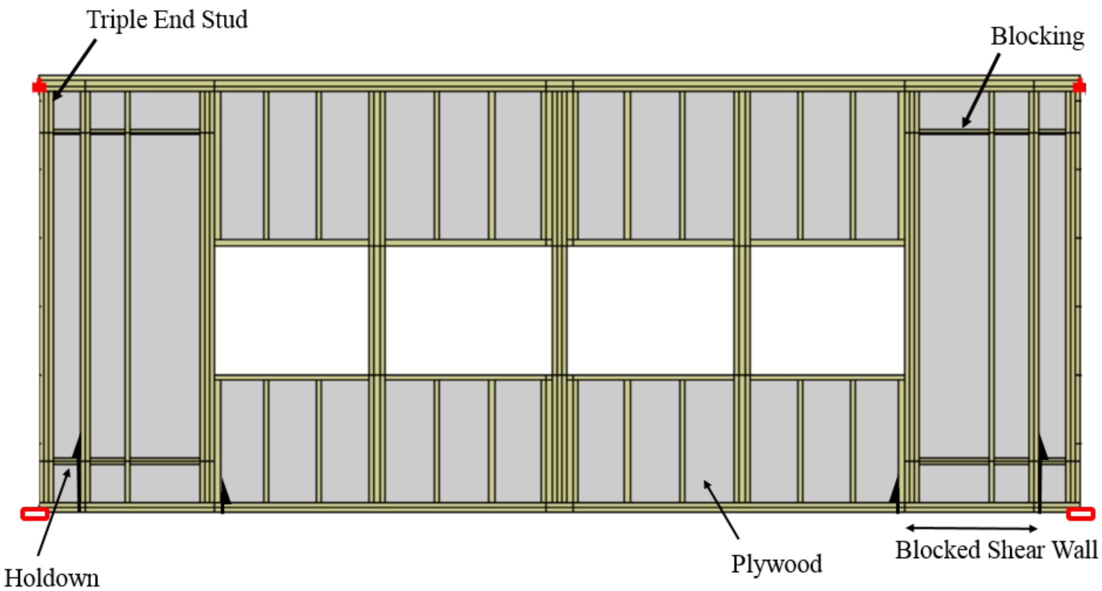


Figure 20: M-CASHEW2 Model of Classroom North and South Elevation

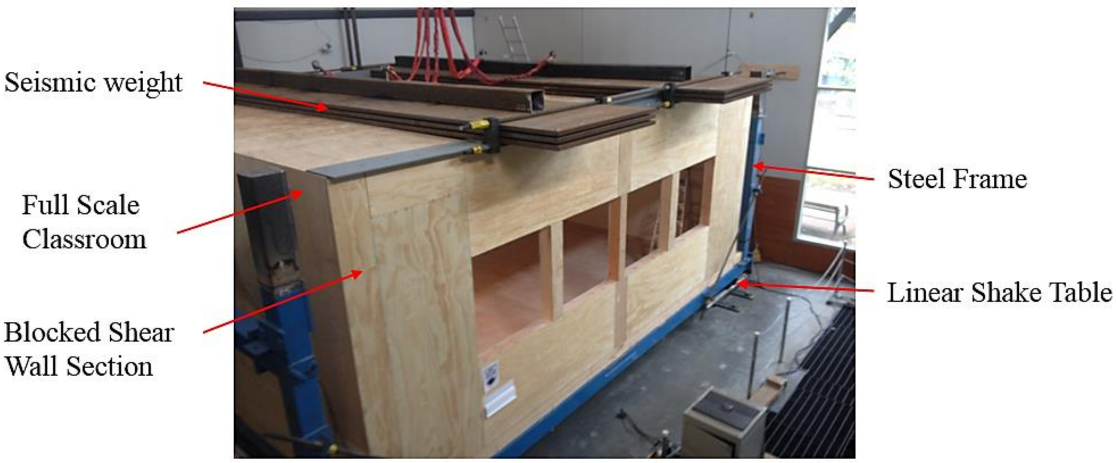


Figure 21: Photograph of test setup prior to testing

4.3 Numerical Model

The prediction for the wall behavior was completed in two parts: (1) a detailed M-CASHEW2 model, (2) a global Timber 3D model.

4.3.1 Detailed Model

The M-CASHEW2 model, developed by Pang and Hassenzadeh (2010), is a 2D shear wall and diaphragm modeling program. The frame elements have four translational and two rotational degrees of freedom (DOF). The sheathing panels are modeled with one rotational DOF, two translational DOFs and two shear DOFs. The bending and axial elongation of the framing members, separation and bearing contacts between framing members, uplift and anchorage of the hold down devices, shear deformation of the sheathing panels, nonlinear shear slip response of the sheathing nails, and second order effect of gravity loads (P-delta) can be captured.

Several connection types are defined in a database available in the M-CASHEW2 program and have been used for the classroom wall model. The sheathing nails between the framing and the plywood were modelled with the EPHM material model fitted to the connection test data by Ekiert and Hong (2006) for nominal 51mm (2 in.) thick Hem-Fir attached to 11.1 (7/16 in.) thick OSB using 8d common nails. This data was available and the difference in the sheathing type was felt to not significantly effect the response. The EPHM model was developed to capture the behaviour of light-frame wood shear walls at high drift levels where stiffness and strength degradation is significant. In-cyclic and cyclic deterioration of strength and stiffness is included in the model, which according to Ibarra et al. (2005) and Chandramohan et al. (Chandramohan, Baker, & Deierlein, in press) makes the model suitable for studying the influence of duration of ground motion on collapse.

The gypsum sheathing and framing connections were modeled with the MSTEW material model based on cyclic tests by Dinehart et al. (2008) of No. 6 gypsum screws and 12mm (1/2 in.) thick gypsum wall board.

The frame-to-frame shear slip for the double stud nails were modeled elastically. The end nail connections between the end posts and sill plates were modelled with a non-linear hold-down spring to describe the uplift response and nail withdrawal, a well as a M-STEW model to described the shear-slip response of two 10d sinker nails. A non-linear contact element was used to describe the bearing deformation between the framing elements. The hold-down elements were modelled with non-linear hold-down springs based on the component testing by United Steel Products (UPS) hold-downs and matched by van de Lindt et al. (2012b). The details of the components of the M-CASHEW2 model and the hysteretic models used are shown in Figure 22 and Figure 23.

It should be noted that the elements were tested using the CUREE protocol (Hassanzadehshiraz, 2012). This protocol has been recognised to be realistic for simulating earthquake loading effects for light-frame wood construction. This protocol better captures the effect of crustal ground motions, further investigation of the effect on behaviour of the elements with longer protocols with multiple pulses should be completed to have a better representation of the element behavior in a long duration seismic event.

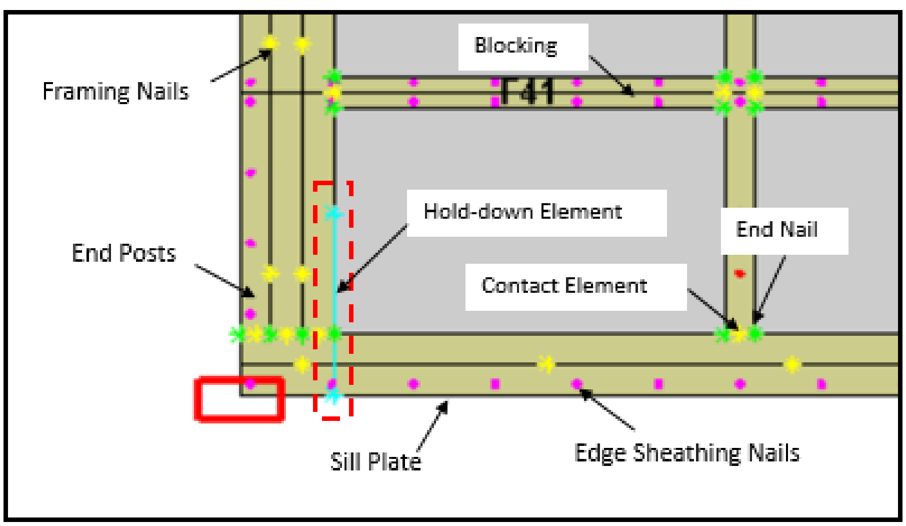


Figure 22: Details of M-CASHEW2 model

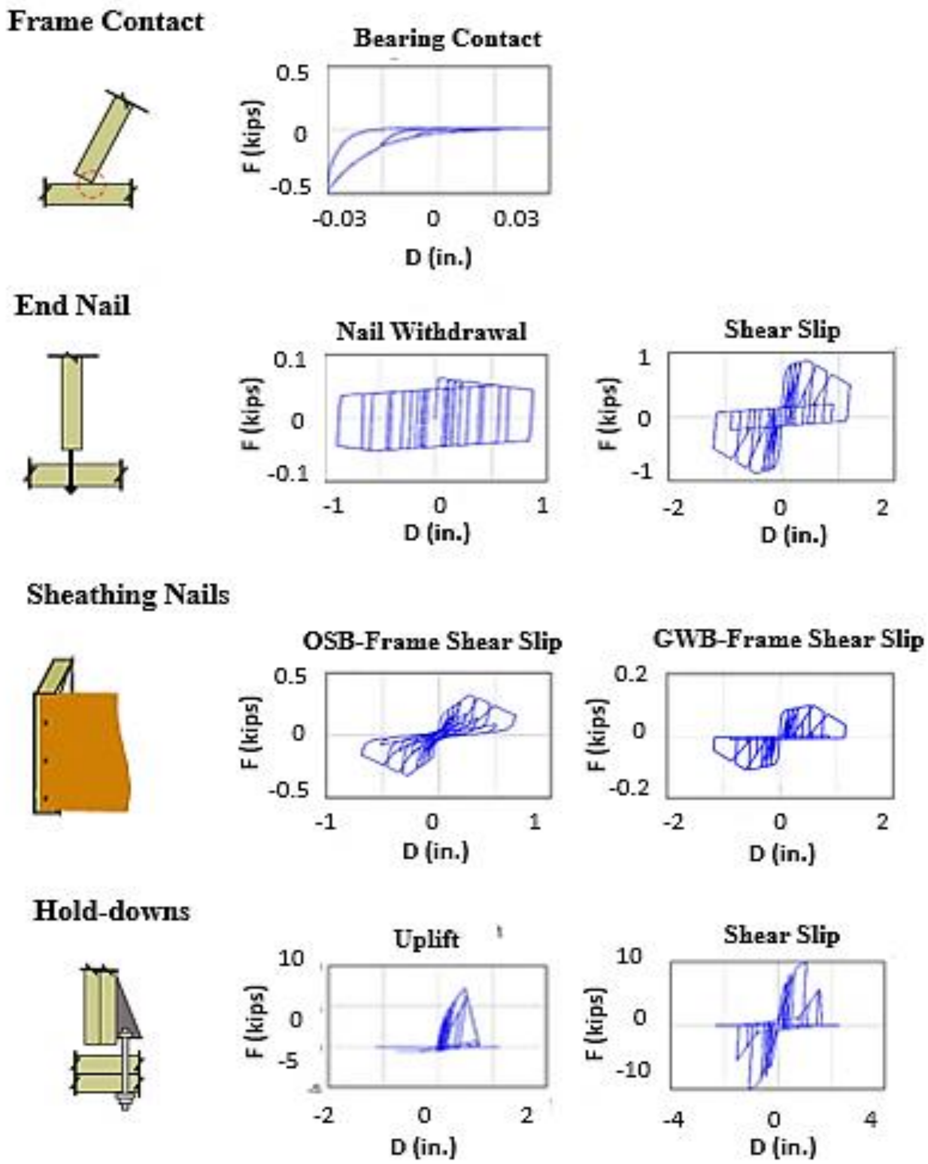


Figure 23: Hysteretic models for (a) frame contact, (b) end nails, (c) sheathing nails, and (d) PHD5 Hold-downs, (van de Lindt J. W., Pei, C., & Hassansadeh, 2012b)

The monotonic and cyclic response of the shear wall model was determined, as shown in Figure 24 and Figure 25, respectively. The standard cyclic protocol in MCASHEW was used. The ultimate force and initial stiffness was estimated as 76.4kN (17.1 kips) and 2.62kN/mm (15.0 kips/in.) The displacement at ultimate is approximately 122mm (4.8 in). The results are for only one side of the classroom test structure, the capacity would be multiplied by a factor of two for the full monotonic and cyclic response of the full structure .

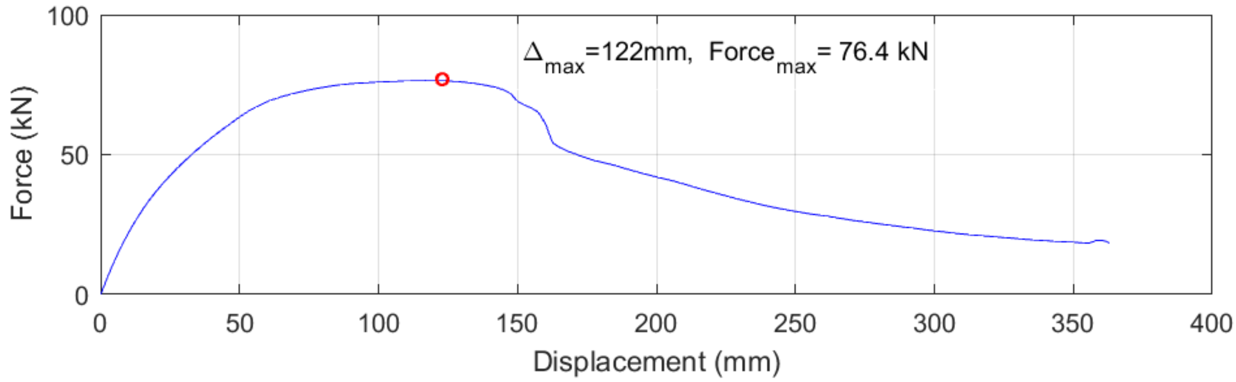


Figure 24: Monotonic response of classroom shear wall numerical model

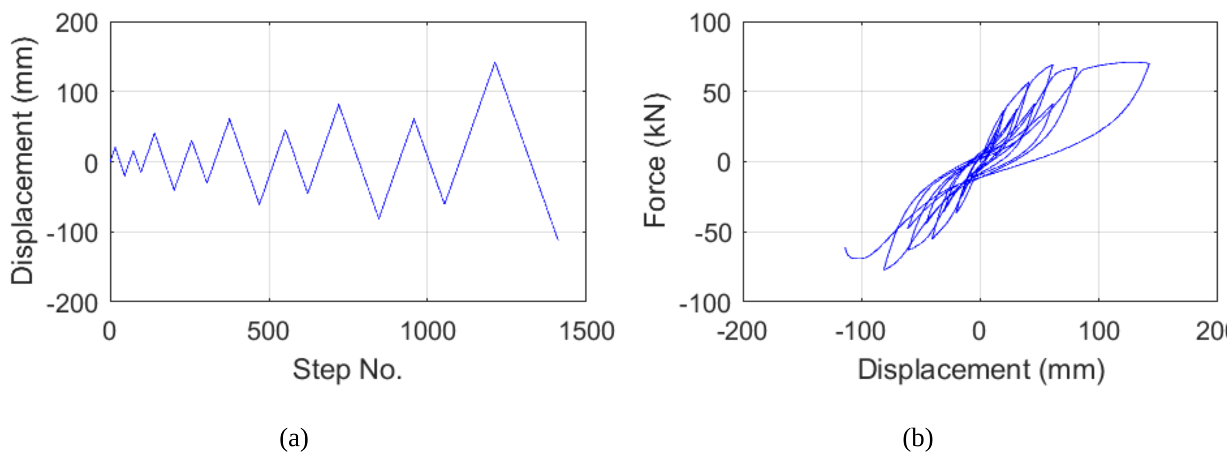


Figure 25: (a) Standard M-CASHEW2 Protocol, (b) Standard M-CASHEW2 Cyclic Response

4.3.2 Global Model

Two global Timber3D models were proposed to define the upper bound and lower bound predictions of the time-history response of the structure: (i) the segmented model; and (iii) the FEMA-P807 opening model. The global model is less computationally intensive compared to the detailed M-CASHEW2 model, as well is more suitable for realistic wood structures with a more involved floor plan and wall layout.

The segmented approach was used for the first model. The Canadian Wood Design code (CWC, 2010) recommends that the openings and wall segments with aspect ratios greater than 3.5:1 are ignored; only the two solid 1.0 m blocked wall segments at the wall ends are assumed to contribute to the strength and stiffness of the system. The blocked wall segments were modeled with RESST shear springs based on the experimental blocked wood shear walls tests performed at UBC and calibrated to the EQ-99 full-scale shake table tests, in Chapter 3. The ultimate strength and stiffness of the hysteretic material model were scaled linearly to the wall length.

The perforated wall approach is used for the second model. The FEMA P-807 guidelines recommend the use of an opening factor multiplied by the ultimate strength to account for the strength and stiffness contributions from the coupling beam behavior of the wall pier headers and sills around the openings. The schematic in Figure 27 shows how the opening factor is calculated; this factor is then multiplied by the ultimate strength of a wall of the same length without openings.

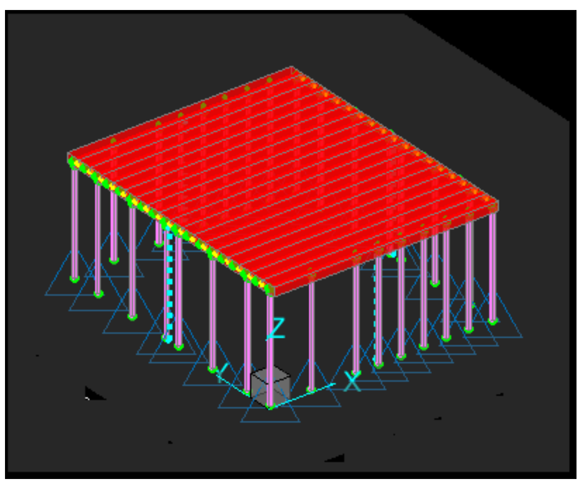
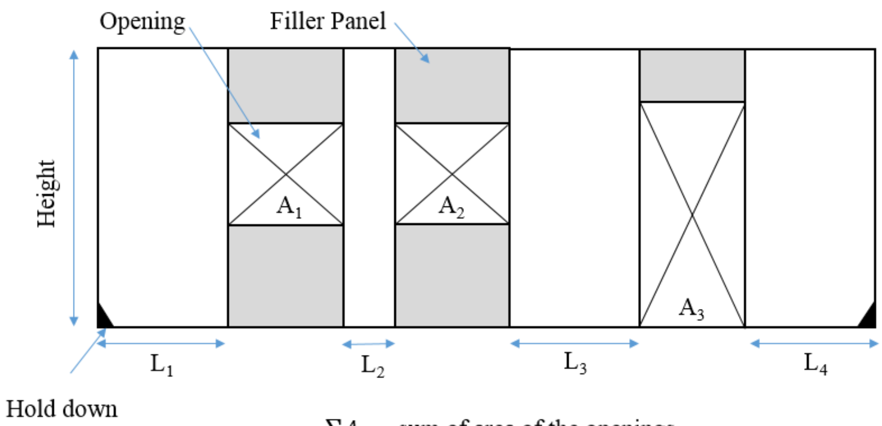


Figure 26: Timber3D global model of Classroom



$\sum A_i$ = sum of area of the openings
$\sum L_i$ = sum of length of full-height wall segments
H = floor to ceiling wall height

$$\text{Opening Factor} = 0.92\alpha - 0.72\alpha^2 + 0.80\alpha^3$$

$$\alpha = \frac{1}{1 + \frac{\sum A_i}{H \sum L_i}}$$

Figure 27: FEMA P-807 Opening Factor

Due to the different nailing schedules of the full height sheathing and the sheathing above and below the openings the FEMA P-807 opening factor cannot be simply applied. If the wall was entirely blocked or unblocked OSB the structure would have a resistance of 135kN and 53kN, respectively. The recommended ultimate resistance was calculated:

$$R_{Model} = R_{LowerBound} - R_{SegmentedUnblocked\ wall} + R_{SegmentedBlocked\ wall}$$

Were $R_{Lowerbound}$ was calculated based on the ultimate capacity for unblocked wood based on experimental testing of walls and the FEMA P-807 opening factor guidelines, $R_{SegmentedUnblockedwall}$ and $R_{SegmentedBlockedwall}$ is the resistance scaled to the 2.0m length per side for the unblocked wall prototype and blocked wall prototype, respectively. A schematic used to describe the recommended ultimate resistance is shown in Figure 28.

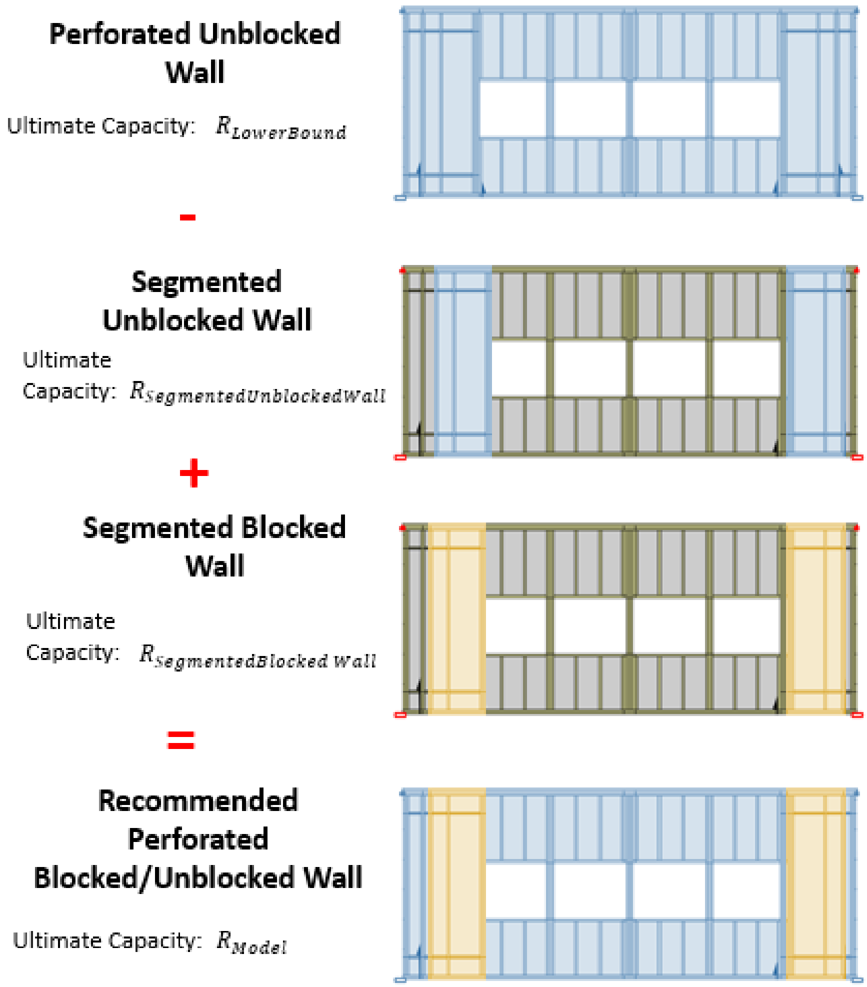


Figure 28: Recommended Perforated Wall Ultimate Capacity

The recommended modeling resistance to account for the openings based on empirical data is between the upper and lower bound solutions.

Table 8: Perforated Wall System – FEMA P-807 Opening Factor

Perforated Wall System (FEMA P-807 Opening Factor)		
Upper Bound Blocked Wall	**Lower Bound Unblocked Wall**	**Modeling Recommendation**
135kN	53kN	91kN
54%W	21%W	36%W

The M-CASHEW2 model predicted a higher ultimate capacity than the calculation of resistance using the results from the experimental walls tests and the FEMA P-807 opening factor. The higher capacity may have been caused by the detailed modeling of each sheathing nail and holdowns in the wall system.

GWB was installed on the interior walls of the test specimen and were accounted for in the numerical model using the superposition method. The stiffness and strength hysteretic parameters were linearly scaled to the length of the solid wall segments; the inner segment with the openings were not included. The gypsum wall parameters were based on data obtained from the tests conducted as part of the CUREE project, the cyclic wall tests from UBC as part of the testing program for the School Seismic Retrofit Guidelines (EERF, 2009), tests performed by Bahmani and van de Lindt (2016), as well as the recommendations from the FEMA P-807 and the technical committee review for the on-going ATC-116 project.

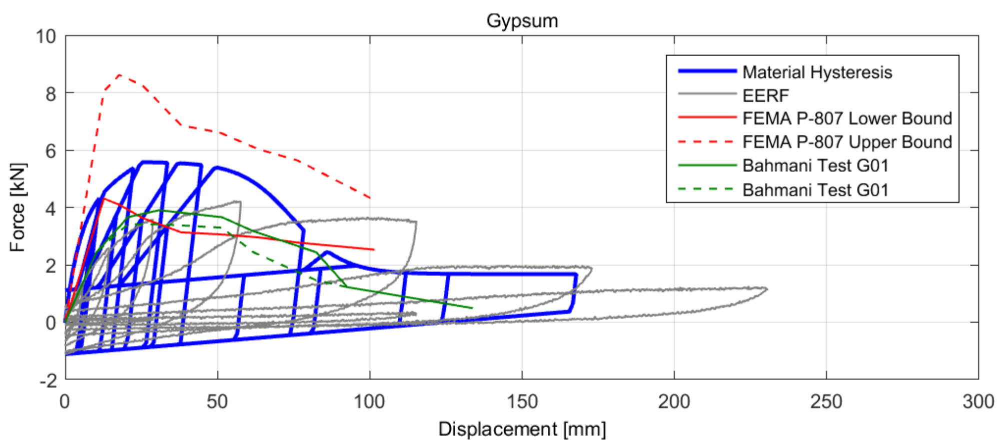
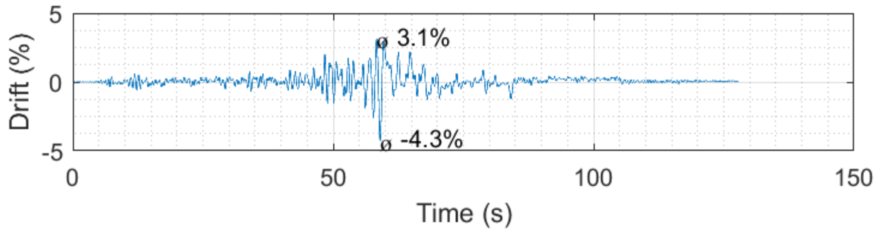


Figure 29: Gypsum Material Model compared to experimental data (8ft wall segment)

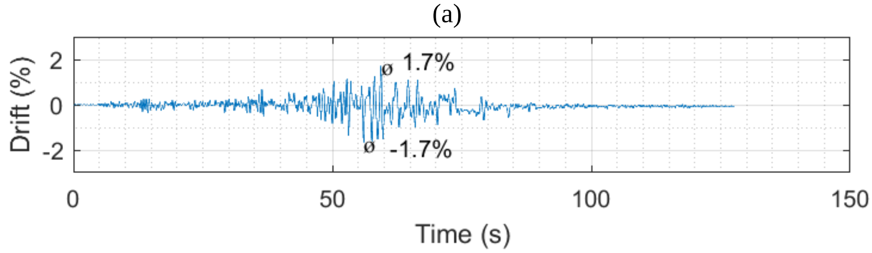
The comparison of ultimate capacity (kN and percentage of the weight) for the segmented and perforated wall approach is summarized in Table 9. The predicted time-history drift response is shown in Figure 30 for the segmented, FEMA P-807 and Timber3D model and the maximum interstorey drift is summarized in Table 9.

Table 9: Upper-bound and lower-bound ultimate capacity of classroom model

Segmented Approach		Perforated Wall Approach	
Unfactored Code Resistance (R_o=1.7)	**Timber 3D Model (4.0m Blocked Wall)**	**Perforated Wall System (FEMA P-807 Opening Factor – Modeling Recommendation)**	**M-CASHEW2 Global Model**
56.0 kN	71.1 kN	93.0 kN	152.0 kN
22%W	28%W	37%W	61%W



(a)



(b)

Figure 30: Prediction of the time history to the pretest acceleration output of the shake table for: (a) the segmented method Timber3D model, (b) the recommended FEMA P-807 Timber3D model

Table 10: Summary of maximum interstorey drift for Classroom model

Model Name	Maximum Absolute Drift
M-CASHEW2 Global Model	0.98%
Perforated FEMA P-807 Model	1.7%
Segmented Model	4.3%

4.4 Comparison of Numerical Prediction and Experimental Results

The test consisted of running the shake table for the TohokuSIT ground motion scaled at 75%, 100%, and 100% for the first, second and third run, respectively. In the first test the structure reached a peak interstorey drift of 1.5%. In the second test the gypsum wall boards were severely damaged; in areas, the GWB panels separated from the studs. The plywood panel framing the window buckled on one side. An interstorey peak drift of 2.8% was observed for the second test. It should be noted that most of the drift was localized to the middle 2439mm (96in.) tall blocked shear wall panels. The 380mm (15in.) panels above and below were much stiffer the middle section and appeared to remain elastic throughout the test. In the third test the structure was extensively damaged; the peak drift was 8.3%. The middle window section separated from the walls at the higher drift levels and therefore, appeared to not contribute to the resistance. Edge and interior nails in the blocked shear wall panels were sheared in half. The studs were misaligned in some places.

To compare the experimental results to the numerical prediction the shake table records for Run 1, 2 and 3 were imputed into the model consecutively. This better represents the testing procedure, as the structure was not repaired between the runs. The predictions of the response were compared for the detailed MCASHEW model and the global model separately.

4.4.1 Detailed Model

The comparison of the numerical and experimental displacement time-history and hysteretic response for Run 1 and Run 2 are shown in Figure 31 and Figure 32, respectively. The drift was calculated over the full height of the specimen (3175mm) for the both the experimental data and the numerical results. The time-history response of the model and test specimen show close to the same dynamic behaviour. The hysteretic damping seems to match reasonably well; however further calibration of the damping and degradation parameters may provide a closer match.

Several of the sheathing nails completely sheared in half after the third test. A way to better model this failure mechanism should be investigated to calibrate the model to the third test. It was challenging to capture the damage for the third run in the detailed model. Furthermore, the buckling and tearing of the sheathing panels was not captures as the panels are modeled with elastic shear elements. By making sub-elements of the sheathing panels attached with material springs the tearing and buckling mechanism may be able to be sufficiently captured.

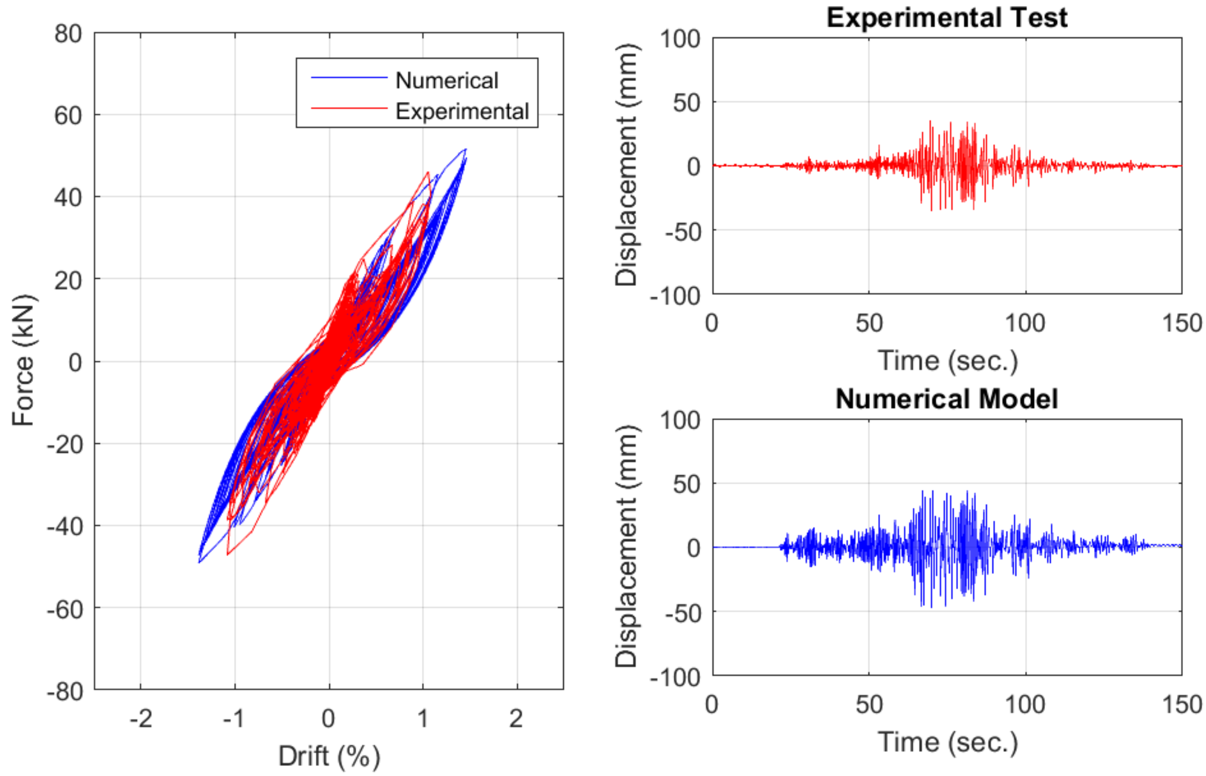


Figure 31: Detailed Numerical Model and Experimental (a) hysteresis (b) relative displacement time history for Run 1

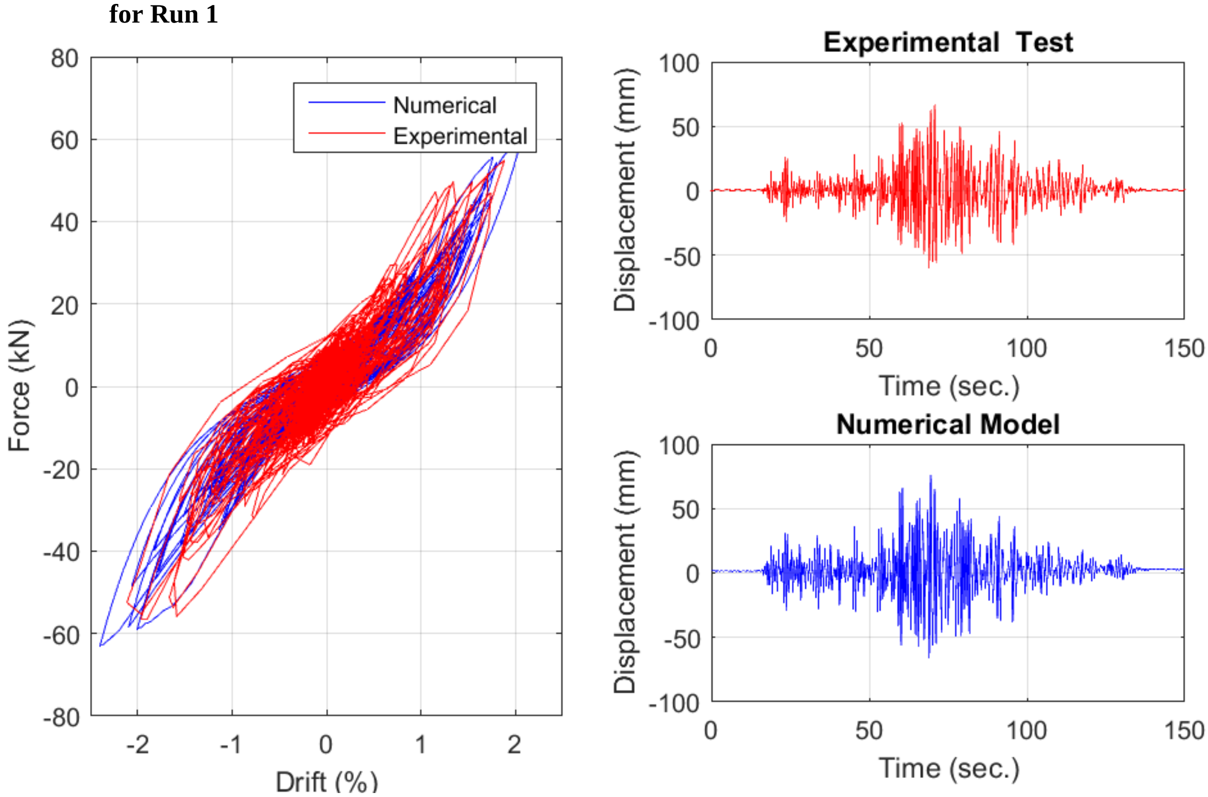


Figure 32: Detailed Numerical Model and Experimental (a) hysteresis (b) relative displacement time history for Run 2

4.4.2 Global Model

The Timber3D model is based on the recommended model with openings from the FEMA P-807 guidelines, as described above. The shear wall springs were reduced to 96in. in height to better represent the localized drift observed during the test. The drift was calculated on the wall height of 96in., rather than the full height of the structure (125in.). A comparison of the numerical and experimental displacement time-history and hysteretic response for Run 1, Run 2 and Run 3 is shown in Figure 33, Figure 34 and Figure 35 , respectively. The material hysteretic parameters were calibrated to reduce the hysteretic damping and achieve a slightly better time history and hysteresis match. Rayleigh damping of 1.0% was used for the first and second mode.

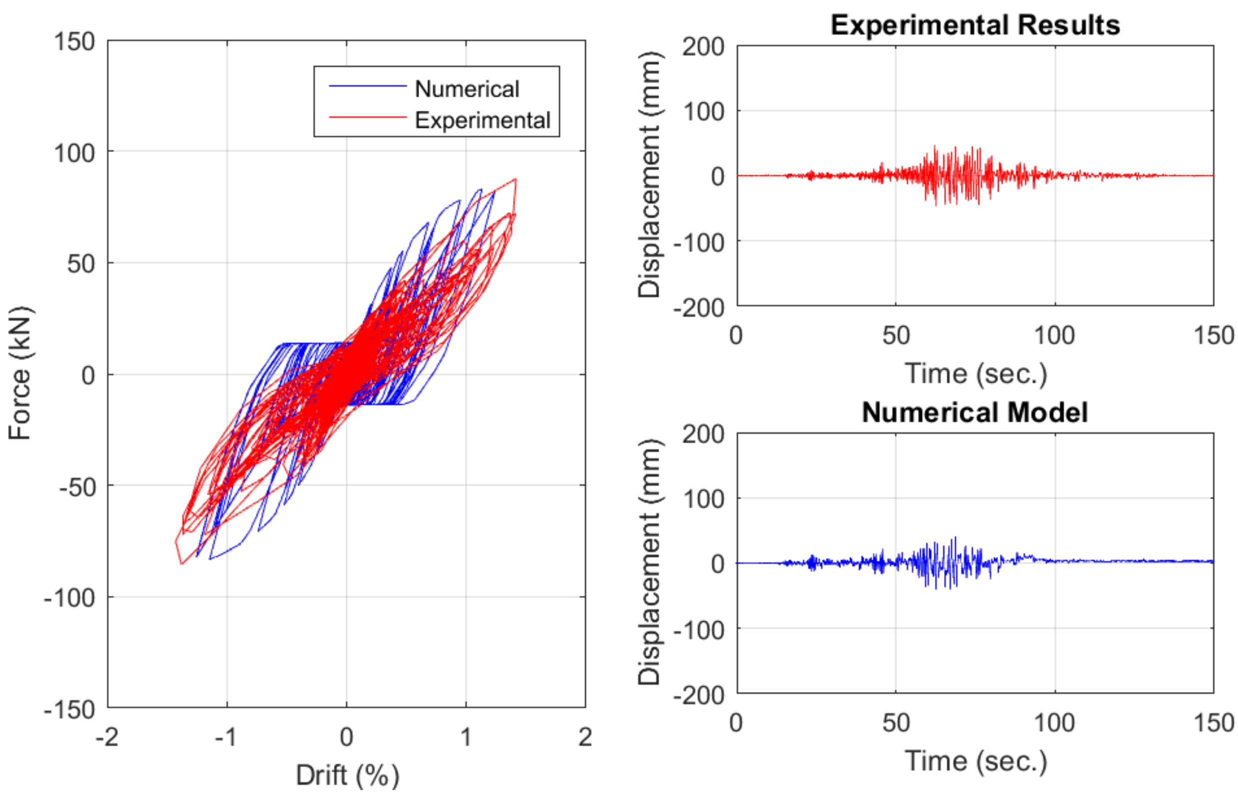


Figure 33: Global Numerical Model and Experimental (a) hysteresis (b) displacement time history for Run 1

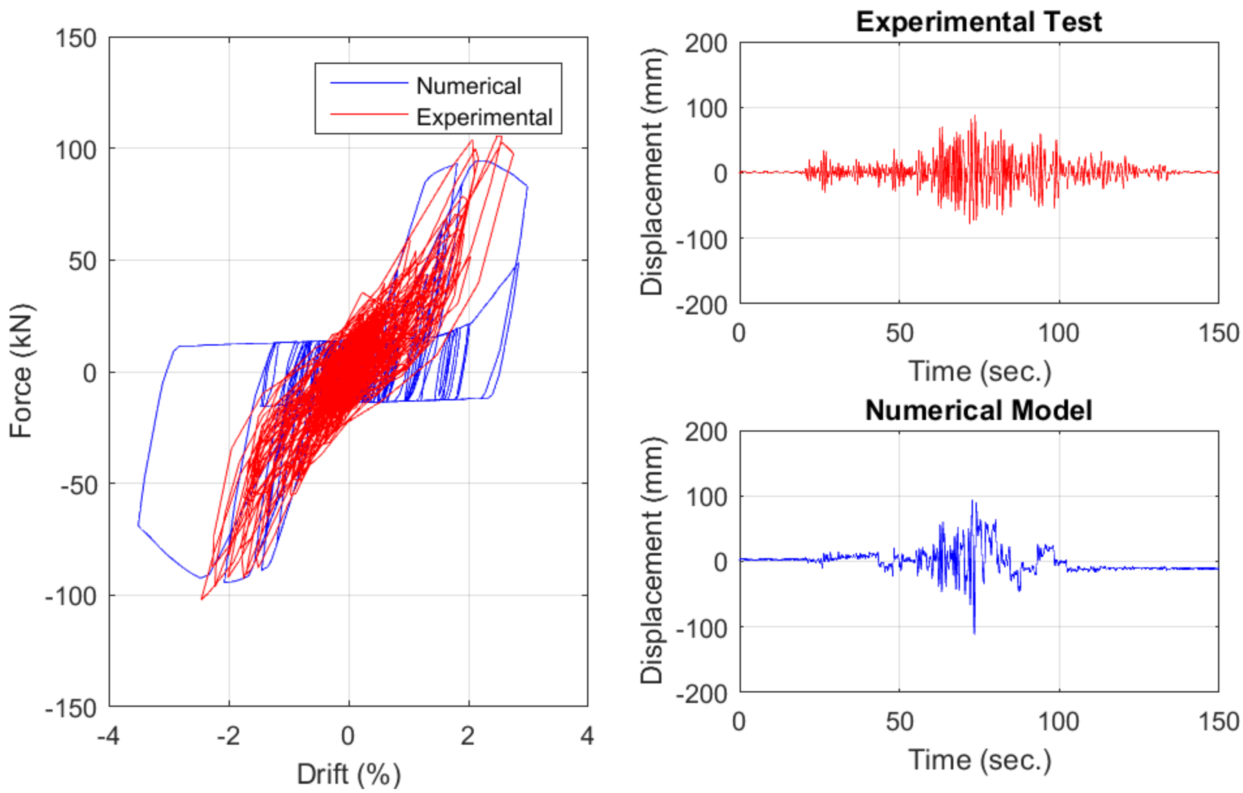


Figure 34: Global Numerical Model and Experimental (a) hysteresis (b) displacement time history for Run 2

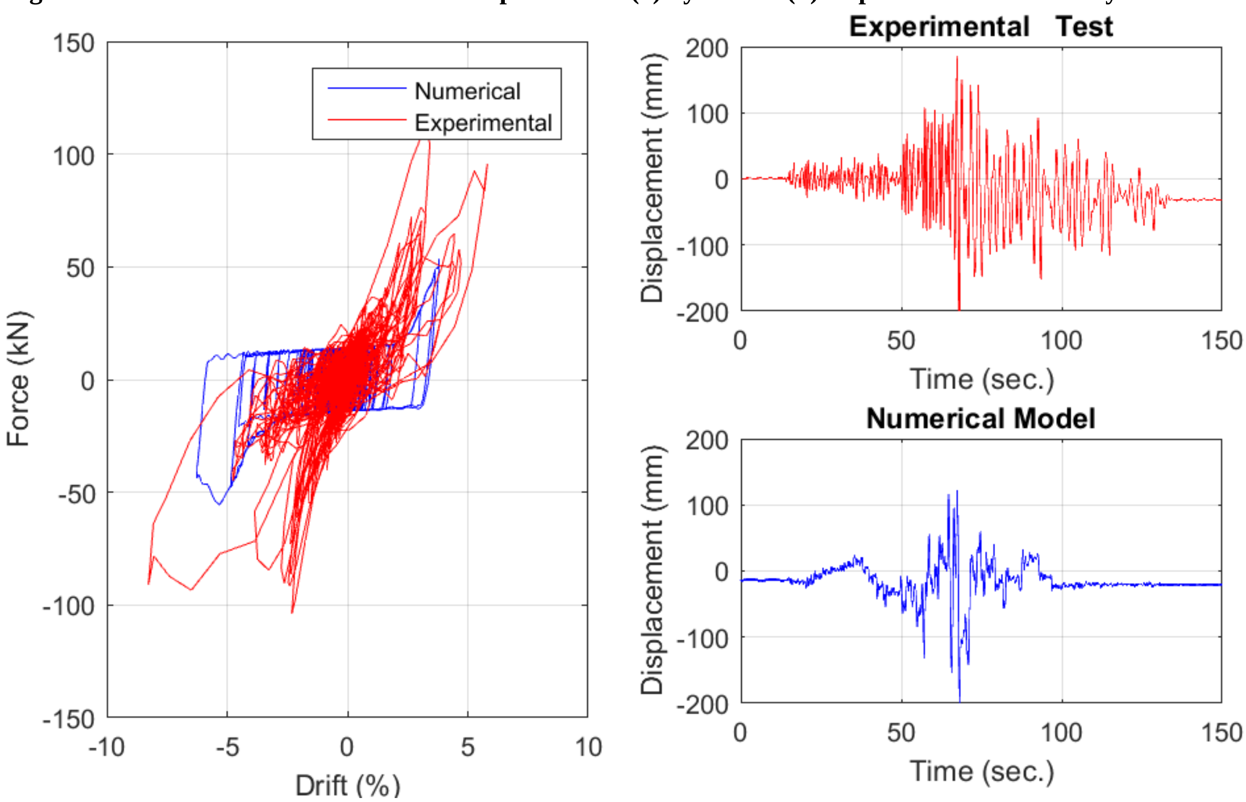


Figure 35: Global Numerical Model and Experimental (a) hysteresis (b) displacement time history for Run 3

The global numerical model could also predict the maximum absolute drift with reasonable accuracy. The model should include the non-structural sheathing walls and the strength and stiffness contributions of the openings. Due to the simplifications of the global and hysteretic material model it is difficult to capture the accumulative damage from previous runs. The structure experienced high drift levels close to collapse by the third run. When the structure is at high drift levels hysteretic damping governs damping within the structure; W. Pang (2015) suggests that close to zero percent Rayleigh damping be used for modeling collapse. The RESST model was not able to capture the stiffness and strength degradation and the pinching behaviour with as much accuracy as the EPHM model. Therefore, the hysteresis for the numerical model is shaped differently than the experimental results. The RESST model, however is less computationally intensive comparted the EPHM, while still accounting for the residual strength existing in the walls after degradation. The detailed M-CASHEW2 model can capture the strength or stiffness degradation, however is very computationally intensive.

4.5 Study of Long Duration Effects with Detailed Model

The influence of ground motion duration on the performance of structures is not well understood. It is difficult to isolate duration effects from the other shaking parameters (i.e. magnitude, frequency content); often higher magnitude earthquakes correspond with longer duration ground motion. Furthermore, up to 10 years ago, prior to the Tohoku 2011 and Maule 2010 earthquakes, it was challenging to produce significant results due to the limited database of available ground motion records. There also has not been good agreement between scientist on how to define ‘duration’ itself. More recently the tendency is to use the duration definition related to the amount of energy released during the shaking, such as ‘significant duration’. Current seismic design practice and loading protocols for component tests do not explicitly consider the effect of duration. Performance based engineering methodologies can implicitly consider duration through the qualitative ground motion selection for a given location. In geological locations where crustal and subduction earthquakes have a significant hazard, such as found in south-western British Columbia, Canada, the effect of duration may be a cause for concern in regards to significant damage and collapse (i.e. in Victoria, B.C. at 1sec period structure subduction, subcrustal and crustal contributes to 60%, 22% and 17% of the total hazard, respectively).

A study by Chandramohan et al. (in press) found that the probability of structural collapse is higher for long duration ground motions compared to short duration ground motions considering spectrally equivalent sets of records for a ductile steel moment frame building. Spectrally equivalent set of records were used to isolate the event of duration from other shaking parameters. A similar study (in press) was conducted on a reinforced concrete bridge pier and the effect of duration was quantified as a 17% decrease on collapse capacity when considering the long duration set rather than the short suite of ground motion. An additional study by Chanadramohan et al. (2016) found that the mean annual frequency of collapse of the same steel moment frame building was underestimated by 29%, 59% and 7% for Seattle, WA, Eugene, OR, and San Francisco, CA, respectively, when using typical-duration ground motions from the PEER NGA-West2 database (as compared to ground motions selected using source-specific probability distributions of the durations of the ground motions anticipated at the site). The probability of collapse was more significantly underestimated for sites where subduction earthquake sources govern the hazard.

There has been little research in determining the effect of duration and subduction earthquakes on light frame wood structures. After seismic events, such as the Northridge earthquakes, the research was focused on addressing the deficiencies observed in the post-earthquake evaluations. The earthquakes were crustal strike-slip, as common to California, and thus, the cyclic-testing protocols developed better represent the characteristics of crustal seismic events. The validated detailed M-CASHEW2 model was used to investigate the effect of the duration of ground motions on the performance of light-frame wood structures, The main parameter of interest for the selection of the ground motions used in this study was the significant duration, which is defined as the 5-95% of the accumulation of the integral (Chandramohan, Baker, & Deierlein, in press):

$$D_{s5-95} = \int_0^{t\_max} a(t)^2 dt \tag{1}$$

where $a(t)$ represents the acceleration time history of the record and t_{max} represents the length of the record. Long duration ground motions are defined in this study as a ground motion with a significant duration longer than 30s.

The intention of this study was to compare the effects of long duration vs. short duration motions; to best perform the comparison a spectrally equivalent short (based on minimizing sum of squares errors between the two response spectra) duration motion was selected for the long duration motion. For a preliminary study, non-linear time-history analysis of two spectrally equivalent pairs was completed. The comparison of the response spectra and time history for the short duration and long duration pairs are shown in Figure 36 and Figure 37 for the KOBE_KAK090/Tohoku_MYG0161103111446-EW records and the SFERN_PDL120 /Tohoku_MYG0161103111446-EW records. The ground motions were scaled to the 2% in 50 years' total hazard level for Vancouver, BC. The scaling factor, magnitude of earthquake, hypocentral distance, V_{s30} and significant duration for the ground motions are summarized in Table 11.

Table 11: Ground motion record properties

	Short Duration Motion 1 KOBE_KAK090	**Short Duration Motion 2 SFERN_PDL120**	**Long Duration Motion 1 Tohoku_MYG0161103111446-EW**
Scale Factor	1.35	3.18	1.10
Magnitude	6.9	6.6	9.0
Hypocentral Distance (km)	30.10	34.18	114.00
Vs30 (m/s)	312.0	452.9	580.0
D_{5-95} (sec)	12.86	17.45	107.00

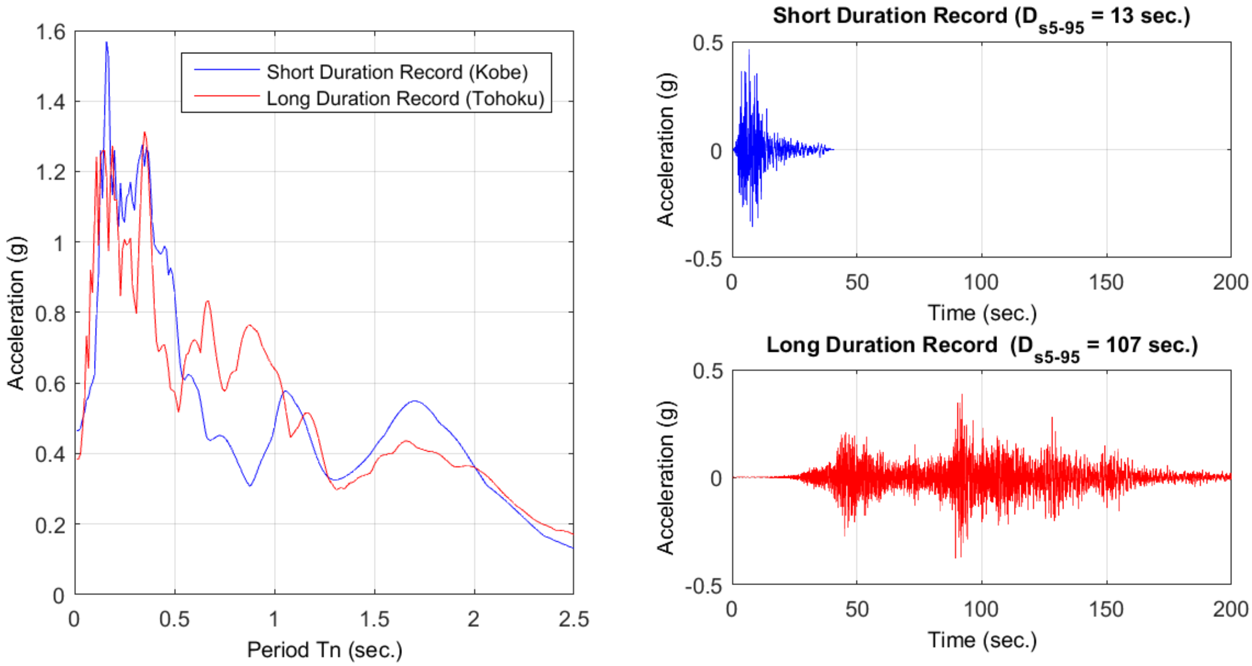


Figure 36: Kobe and Tohoku spectrally equivalent records (a) response spectra (5% damping) and (b) time history of short and long duration records

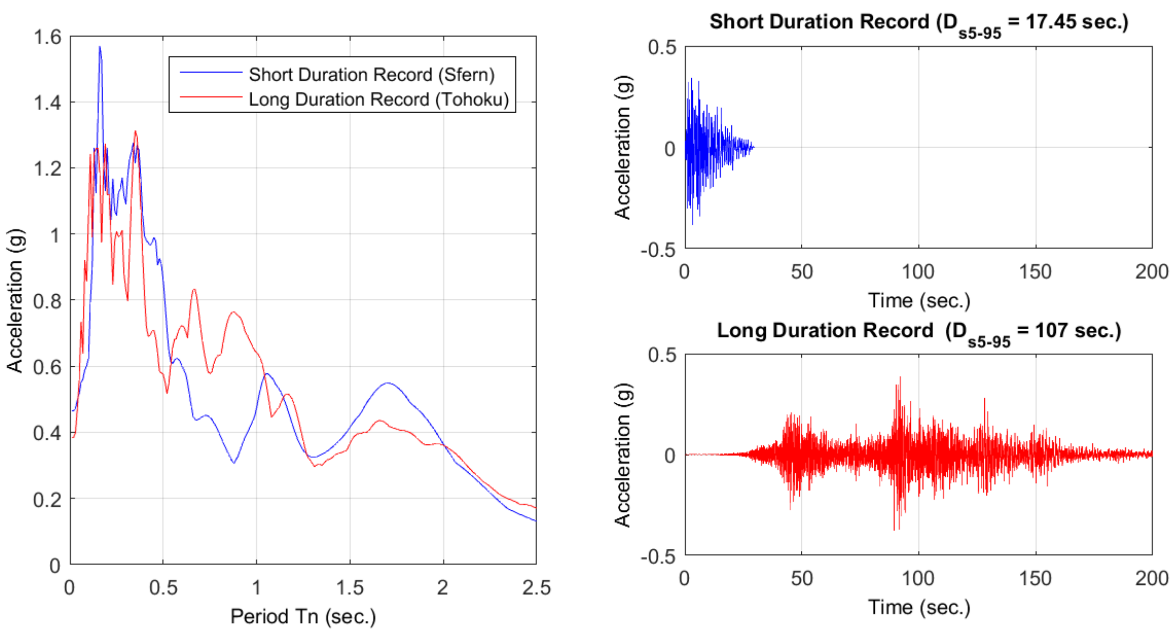


Figure 37: Sfern and Tohoku spectrally equivalent records (a) response spectra (5% damping) and (b) time history of short and long duration records

A comparison of the force-drift hysteretic and time-history response of the long and short duration ground motions pairs is shown in Figure 38 and Figure 39 based on the detailed M-CASHEW2 classroom model validated in the previous section. At the design hazard level, the long duration ground motion caused 32% and 27% more drift than the first and second especially equivalent short duration motion, respectively. This suggests that the margin against collapse may be lower when this type of system is subjected to long duration motions. A more comprehensive analysis program should be completed with a wider selection of various ground motions scaled to a range of hazard levels to have a better understanding of the effect of ground motion duration on seismic behaviour and expected collapse. Further full-scale testing with different sheathing configurations and openings are to be completed, as described in Appendix G. The additional testing program will involve shake table tests with short duration and long duration especially equivalent pairs.

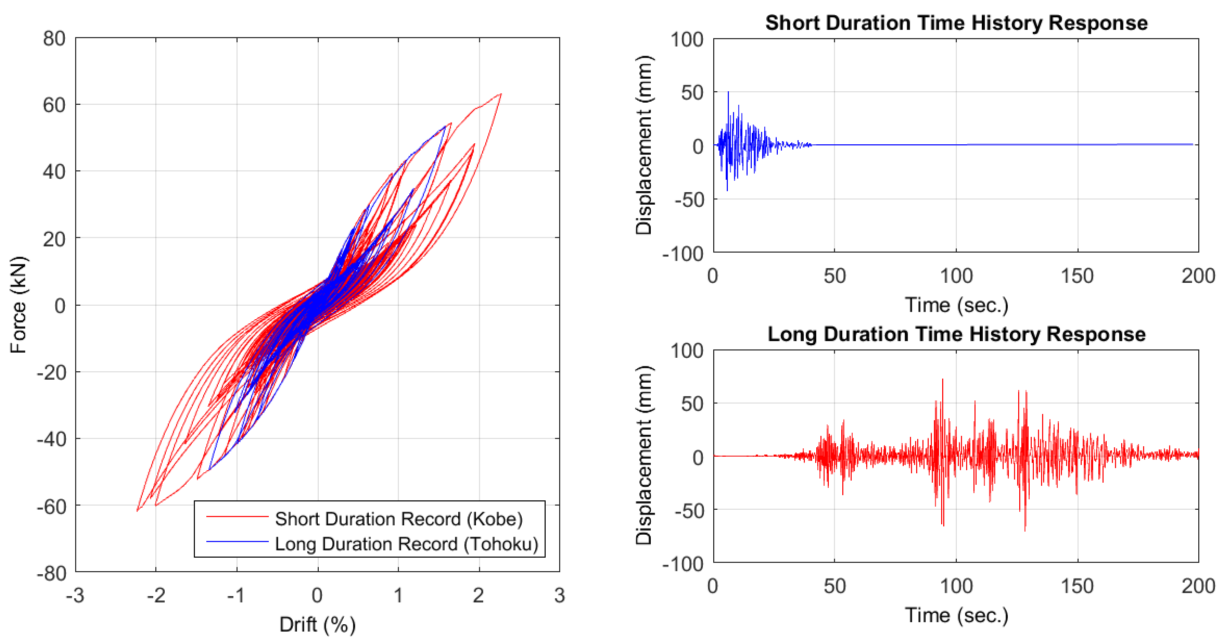


Figure 38: Comparison of numerical analysis results for Kobe (Short) and Tohoku (Long) spectrally equivalent ground motions (a) hysteresis, (b) displacement time-history

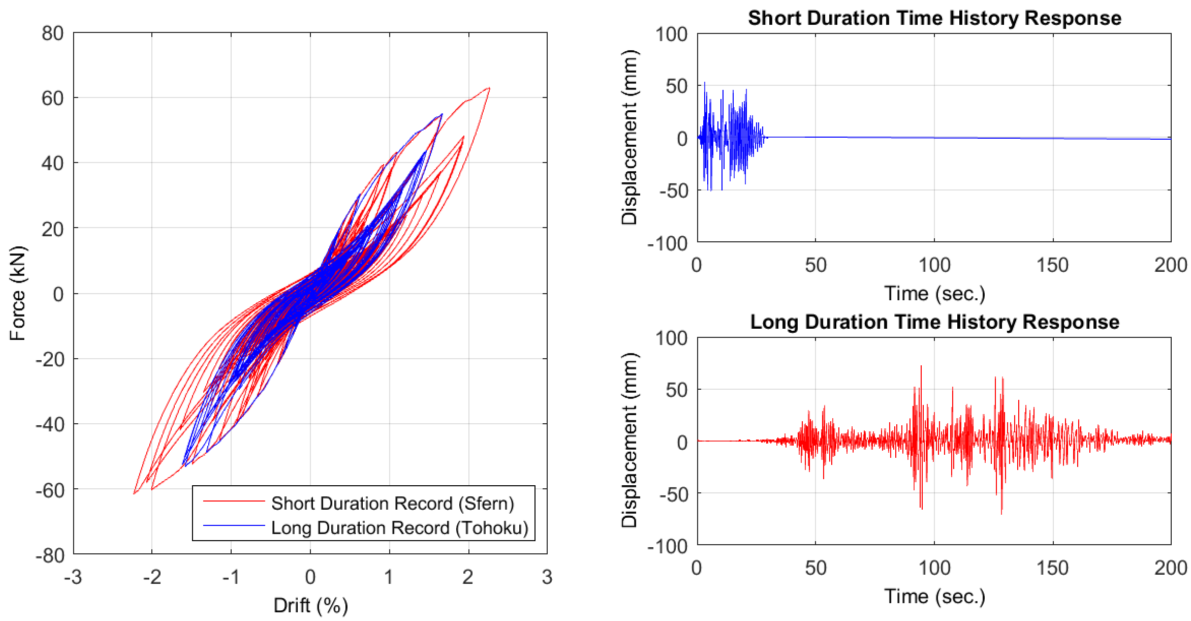


Figure 39: Comparison of numerical analysis results for Sfern (Short) and Tohoku (Long) spectrally equivalent ground motions (a) hysteresis, (b) displacement time-history

4.6 Summary

The detailed M-CASHEW2 model could predict the cyclic and time history response with considerable accuracy. Further calibration is required to fully capture the degradation and damping characteristics at the high drift levels when the structure is significantly damaged.

The M-CASHEW2 model was also used to investigate the effect of ground motion duration on the seismic response. The model has sufficient detailing and defined cyclic degradation to be able to capture the effect of duration. At the 2% in 50-year hazard level for Vancouver, the long duration ground motion caused about 30% more drift than the spectrally equivalent short duration motion.

The global numerical model was also able to predict maximum absolute drifts of the response accurately. The model should include the non-structural sheathing walls and the strength and stiffness contributions of the openings. Due to the simplifications of the global and hysteretic material model it is difficult to capture the accumulative damage from previous runs. The more detailed M-CASHEW2 model can capture the strength or stiffness degradation, however is very computationally intensive, and therefore has more limited application. Additional modeling and analysis for the classroom model tested with a different opening and shear wall configuration has been included in Appendix F.

Chapter 5: Seismic Assessment and Retrofit

5.1 Introduction

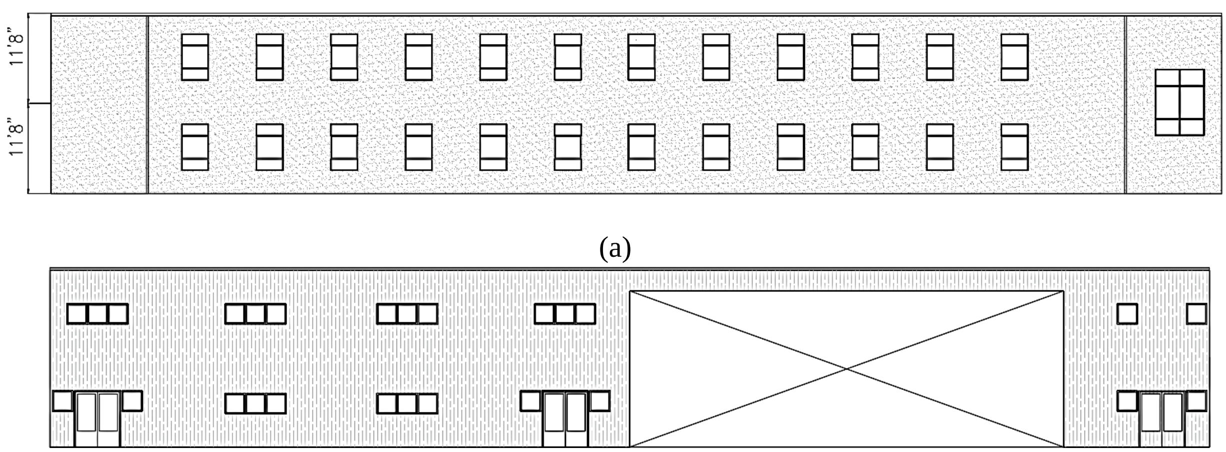
The global numerical modeling methods validated in Chapter 3 and Chapter 4 were applied to predict the seismic performance of a typical light-frame wood school building block in Vancouver, BC constructed in the 1950s. By applying the same numerical modeling methods that were calibrated to the experimental tests the model should be able to predict of the seismic behaviour of the existing structure. The model was also used to evaluate the performance of proposed retrofit options and investigate how these retrofits alter the seismic behaviour of the structure. The study was completed with non-linear time history analysis using biaxial SAPWood models and the three-dimensional Timber3D models. This chapter focuses on comparing the detailed modeling to simplified analysis tools, as well as investigates the expected collapse mechanisms of the structure.

5.2 Numerical Modeling

The seismic behavior of a two-storey wood frame school block was investigated. The structure represents typical 1950-1960 light-frame wood construction in the lower mainland of British Columbia. The foundation of the building is slab on grade. The exterior walls are sheathed with horizontal shiplap with a combination of vertical shiplap and stucco finishing. The interior walls are sheathed on both sides with gypsum wall board. The roof and suspended floors are horizontal shiplap on joints spanning to the stud walls. The clear storey height is 3.5m. The schematic of the first and second floor and the elevation view are shown in Figure 40 and Figure 41. The effective seismic weight for the first and second floor are estimated to be 545kN and 642kN, respectively. The school is assumed to be on Site Class C soil and soil structure-interaction is not explicitly considered.

The school was initially modeled as a biaxial shear model in the analysis program, SAPWood. The diaphragm was assumed to be perfectly rigid with one rotational and two in-plane translational degrees of freedom for the first-floor diaphragm and roof. This modeling simplification significantly reduces the computational time of the analysis. The shear walls were modeled with zero-height non-linear SDOF shear springs. The viscous damping was taken as 1.0% Rayleigh damping; it was assumed that much of the damping is accounted for through hysteretic damping.

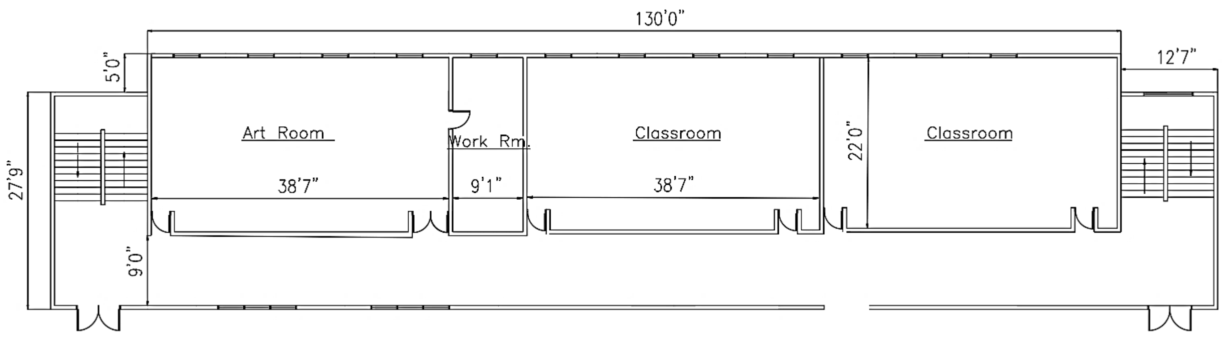
The school block was also modeled using the three-dimensional Timber3D model, as shown in Figure 42. The diaphragm was modeled with 3D frame elements and the shear wall behavior were modeled with non-linear shear spring link elements. The computations time and effort is increased compared to the biaxial shear model. The Timber3D numerical model gives more stable predictions; the lateral behavior of the model seems to be less sensitive to changes in the material models. Timber3D models have proven to be able to accurately predict global collapse (Pang, Ziaei, & Filiatrault, 2012). The viscous damping was taken as 1.0% Rayleigh damping assigned to modes 1 and 2.



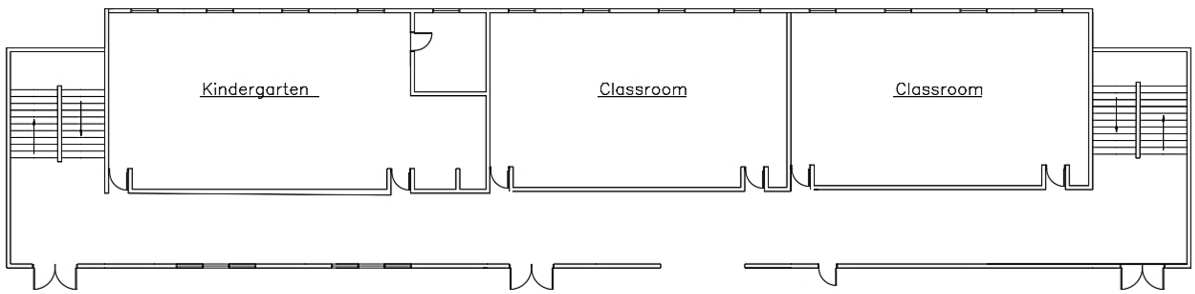
(a)

(b)

Figure 40: Elevation View of Institutional Archetype, (a) North, (b) South



(a)



(b)

Figure 41: Plan View of Institutional Archetype: (a) second floor, (b) first floor

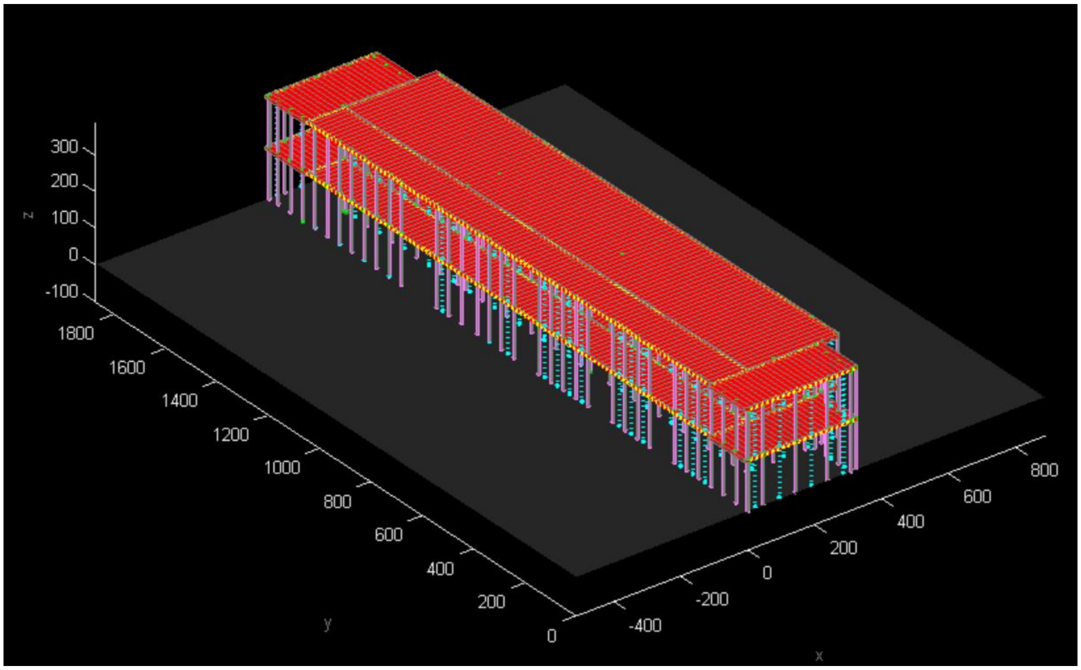
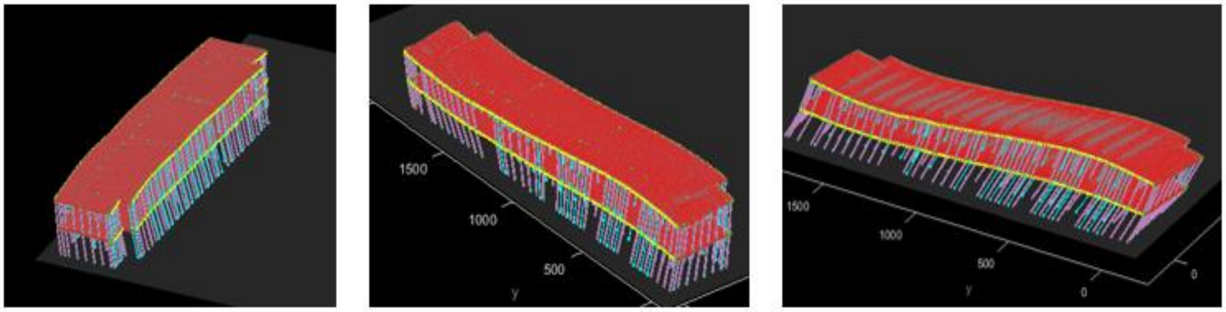


Figure 42: Modelling light-frame school block with Timber 3D

The first three periods of the building model are 0.60s, 0.46s, and 0.38s, which correspond to translational mode in the North-South direction, torsional model and translational mode in the East-West direction, respectively (Figure 43).



(a) (b) (c)

Figure 43: Modes of Vibration: (a) north-south, (b) torsional, (c) east-west

The performance of the building block was estimated with the Seismic Retrofit Analyzer Version 3.0 (SRG3) as part of the BC School Seismic Retrofit Program. The weight of the dead load was calculated referencing CSA O86-10 (CWC, 2010) and the factored resistance of the shear walls were based on recommendations from SRG3. The resistance as a percentage of the weight, storey height (3500mm), community (Vancouver), soil type (Class C) and design drift limit (3.5%) was imputed into the SRG3 calculator for each prototype. The exterior shiplap and interior gypsum walls are modelled with prototype W-4 and W-3, respectively. Table 12 summarizes the percent resistance in the N/S and E/W direction and the respective probability of drift exceedance and risk category. The overall risk of the existing block is H1. A description of the retrofit priority ranking is given in Table 13; structures with a Probability of Drift Exceedance (PDE) less than 2% do not require a retrofit, structures evaluated with a PDE greater

than 2% are Medium Risk or one of the three High Risk categories (H1, H2 or H3), H1 being the most structurally deficient category. The existing stucco finish was included in the existing and retrofitted models of the school block. To access the existing building and develop the retrofit options in SRG analyzer the contribution of strength and stiffness from the stucco finishing was not considered.

Table 12: SRG3 initial lateral resistance system assessment

Prototype No.	Prototype Description	Resistance %W	Probability of Drift Exceedance	Retrofit Priority Ranking
E/W Direction				
W-4	Horizontal Boards	5.8%	7.00%	H2
W-3	Gypsum Wallboard	4.6%	5.00%	H3
N/S Direction				
W-4	Horizontal Boards	2.6%	19.10%	H1
W-3	Gypsum Wallboard	2.4%	12.00%	H1
		Maximum PDE	**19.10%**	**H1**
			Existing Block Retrofit Ranking Priority	**H1**

Table 13: Retrofit Priority Ranking Description

Probability of Drift Exceedance (PDE)	Retrofit Priority Ranking
PDE > 10%	**H1**
10% ≥ PDE > 7%	**H2**
7% ≥ PDE > 5%	**H3**
5% ≥ PDE > 2%	**M**
PDE ≤ 2%	**No Retrofit Required**

5.2.1 Wall Hysteresis Models

The behavior of the shear walls was modeled with the MSTEW material hysteresis model for the SAPWood model. The Timber3D analysis program has implemented the RESST material model. This material model has a more appropriate backbone curve and residual strength definition for the light-frame materials. The Timber3D shear walls were modeled with a combination of the MSTEW and RESST material models. The material models parameters for 8ft. segments of the existing gypsum wall board, traditional stucco and horizontal siding walls are given in Table 14.

Table 14: Wall Hysteresis Parameters (per 8ft. wall)

RESST Material Model												
	K_0 kN/mm kip/in.	**r_1**	**r_2**	**r_3**	**r_4**	**F_x kN kip**	**f_1**	**f_2**	**f_3**	**D_x**	**α**	**β**
Gypsum Wall Board (2)	0.89 5.1	0.07	0.46	1.01	0.010	5.87 1.32	3.02 0.68	0.18	0.3	82 3.23	0.80	1.10
Traditional Stucco Construction (2)	2.63 15.0	0.13	-0.05	1.45	0.005	40.2 9.04	1.97 0.442	0.09	0.1	119 4.70	0.38	1.09
MSTEW Material Model												
	K_0 kN/mm kip/in.	**r_1**	**r_2**	**r_3**	**r_4**	**F_O kN kip**	**F_I kN kip**	**D_u mm in.**	**α**	**β**		
Horizontal Siding (1,2)	0.21 1.18	0.1	-0.95	1.01	0.035	1.60 0.36	0.6 0.136	241 9.5	0.45	27 1.06		
Gypsum Wall Board (1)	0.89 5.11	0.07	-0.04	1.01	0.01	4.00 0.90	1.11 0.25	25.4 1.0	0.8	1.1		
Traditional Stucco Construction (1)	1.75 10	0.13	-0.06	1.45	0.005	6.67 1.50	2.40 0.54	20.3 0.8	0.38	1.09		

(1) Material Model for SAPWood V2.0
(2) Material Model for Timber3D

The stucco external finishing was modeled based on the stucco tests referenced in the FEMA P-807 (FEMA, 2012) document and the tests performed by Bahmani and van de Lindt (2016) and Sofali (2008). Bahmani and van de Lindt (2016) conducted reverse cyclic tests on 2.4x2.4m (8’x8’) stud walls with one layer of 22.2 mm (7/8 in.) thick stucco. The stucco was constructed to emulate the construction methods of the 1920’s to 1950’s consisting of five sub layers: a weather barrier layer, wire lath, a scratch coat, a brown coat, and a finish coat. The stucco specimens were fully cured before testing and had 28-day compressive strength from 17.2 to 20.7 MPa (2.5 to 3.0 ksi) and a unit weight of 478 N/m2 (10 psf).

Sofali (2008) conducted stucco wall tests based on traditional construction. The regular stucco shear wall had stapled wire lath over single layer of building paper secured in place using horizontal wire stones at 6 in. spacing. The 1.0 in. welded wire lath (Structalath) was attached with 11 gauge, 1in. long, 1 1/4 in. wide crown staples at 12 in. on center to the framing members. The stucco boundaries were confined by a 3/4 in. aluminum stop that was screwed around the form. A two-coat system of 24MPa with the total thickness of ¾ in. was applied.

Figure 44 shows the envelope curves from the tests by Bahmani and van de Lindt (2016), Sofali (2008), the upper and lower bound recommendations from the FEMA P-807 guidelines and the ASCE 41-13 (ASCE/SEI, 2013) default curve. It should be noted that the first point reported by FEMA P-807 is at 0.5% drift, and thus the initial stiffness and yielding drift cannot be determined from this curve. The FEMA P-807 stucco model is reduced to zero resistance at 1.5% drift; this assumption seems to be overly conservative when compared to the results from the cyclic tests and therefore, for the stucco model the degrading portion of the backbone curve has been extended.

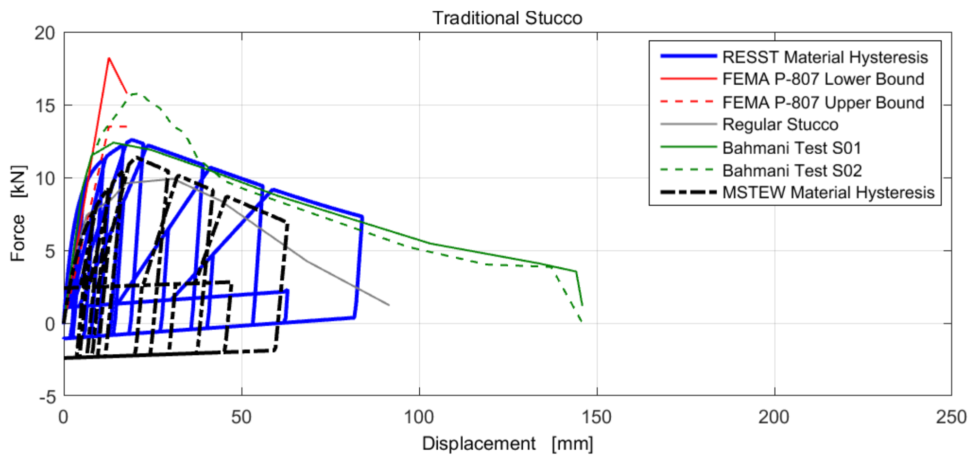


Figure 44: Stucco Material Model compared to experimental data (8ft. wall)

The gypsum wall parameters were based on data obtained from the tests conducted as part of the CUREE project, the cyclic wall tests from the University of British Columbia as part of the testing program for the School Seismic Retrofit Guidelines (EERF, 2009), tests performed by Bahmani and van de Lindt (2016), as well as the recommendations from the FEMA P-807 and the technical committee review for the on-going ATC-116 project. The backbone curves and hysteresis for the experimental tests and material models is shown in Figure 45.

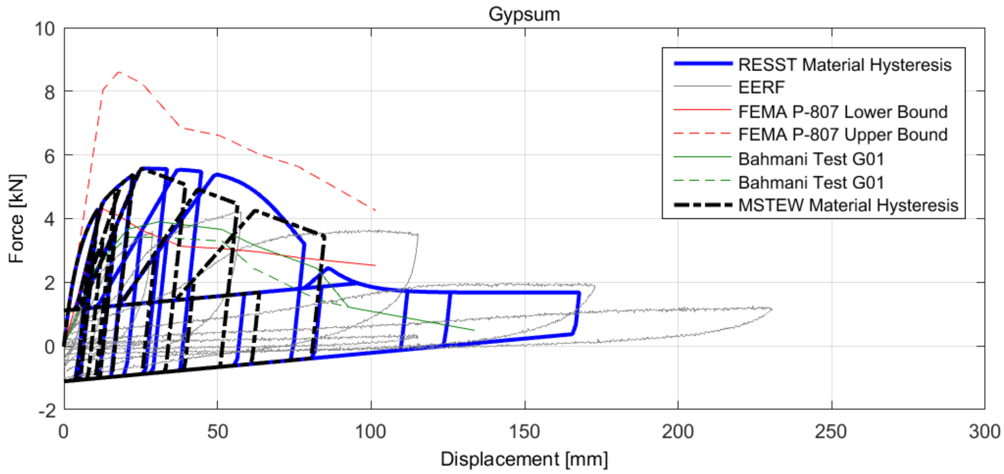


Figure 45: Gypsum Material Model compared to experimental data (8ft. wall)

The horizontal wood siding model was based on wall test data conducted in the 1950s in the Forest Products Laboratory, the cyclic wall tests from the University of British Columbia as part of the testing program for the School Seismic Retrofit Guidelines (EERF, 2009), tests performed by Bahmani and van de Lindt (2016), as well as the recommendations from FEMA P-807 and the ATC-116 project.

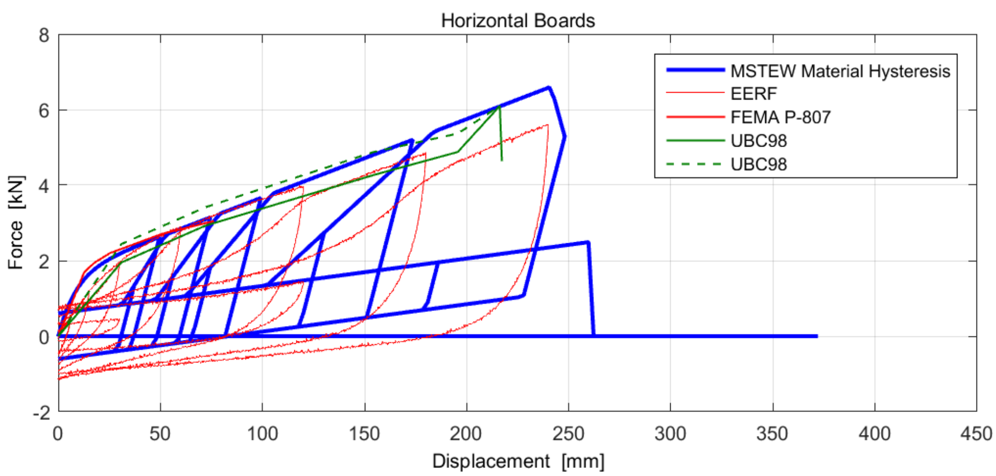


Figure 46: Shiplap Material Model compared to experimental data (8ft. wall)

5.3 Retrofit Options

The performance of six main retrofit options have been evaluated using the SAPWood model, as well as the three dimensional Timber3D numerical model of the school block archetype including (1) a shear wall retrofit, (2) an exterior stucco retrofit, (3) CLT panel walls, (4) special steel moment frames, and (5) a distributed knee-brace system. A summary of the material parameters used in the models are given in Table 15.

Table 15: Wall Hysteresis Parameters (per 8ft. wall)

RESST Material Model

	K_0 **kN/mm** **kip/in.**	**r_1**	**r_2**	**r_3**	**r_4**	**F_x** **kN** **kip**	**f_1**	**f_2**	**f_3**	**D_x**	**α**	**β**
Engineered Blocked Wood Panel (2)	1.57 9.0	0.01	-0.23	1.01	0.030	41.2 9.26	4.31 0.97	0.13	0.3	121 4.77	0.76	1.15
New Stucco Construction (2)	2.63 15.0	0.13	-0.05	1.45	0.005	40.2 9.04	1.97 0.442	0.09	0.1	119 4.70	0.38	1.09

MSTEW Material Model

	K_0 **kN/mm** **kip/in.**	**r_1**	**r_2**	**r_3**	**r_4**	**F_O** **kN** **kip**	**F_I** **kN** **kip**	**D_u** **mm** **in.**	**α**	**β**
CLT panels (1, 2 ,3)	0.35 2.02	0.078	-2.62	1.50	0.015	27.0 6.08	0.60 0.136	175 6.9	0.7	1.07
Distributed Knee-Brace System (1, 2)	0.26 1.5	0.06	-0.31	1.40	0.056	9.43 2.12	2.90 0.65	126 4.95	0.9	1.05
Engineered Blocked Wood Panel (1, 2)	1.58 9.0	0.01	-0.23	1.01	0.01	32.56 7.32	4.00 0.9	97 3.8	0.8	1.5
New Stucco Construction (1, 2)	2.63 15.0	0.055	-0.04	1.45	0.005	13.3 3.0	5.34 1.2	43 1.7	0.38	1.09

Bilinear Material Model

	K_1 **kN/mm** **kip/in.**	**r**	**D_y**
Special Steel Moment Frames (1, 2)	1.91 10.9	0.113	19.05 0.75

(1) Material Model for SAPWood
(2) Material Model for Timber3D
(3) Single CLT Panel 2ft in length

The retrofit options were defined based on the seismic performance analyzer as part of the School Retrofit Guidelines. The lateral drift resisting system (LDRS) should meet (i) the maximum drift limit (Design Drift Limit); (ii) the minimum capacity for a probability of drift exceedance (PDE) of 2% in 50 years. The Design Drift Limits (DDL) is based on the prototype, municipality, site class and storey height. The lateral capacity of the retrofit must be equal to the demand for a PDE of 2% in 50 years to meet the Life Safety performance objective. The toolbox method provides a procedure for performing a retrofit design of a block that has mixed LDRSs (different prototypes, new or existing materials). The Toolbox Method (Ventura, Finn, & Bebamzadeh, 2012) treats each LDRS separately and then provides a method for accumulating the performance contribution from each LDRS to determine the overall block performance. Appendix C summarizes the Toolbox Method in more detail. Table 16 summarizes the demands, required resistance of retrofit in terms of resistance and per unit (i.e. metre, frames).

Table 16: Retrofit option requirements

	Retrofit 1: Blocked Shear Wall	**Retrofit 2: New Stucco Exterior**	**Retrofit 3: CLT Panels**	**Retrofit 4: Steel Moment Frame**	**Retrofit 5: Distributed Knee System**
SRG Prototype	W-1 Blocked Shear Wall	C-4 Squat Shear Concrete Wall	W-1 Blocked Shear Wall	S-9 Ductile Steel Frame	W-1 Blocked Shear Wall
Required Resistance	10.90%W	18.10%W	10.90%W	13.70%W	9.90%W
PDE	2.00%	2.00%	2.00%	2.00%	2.00%
DDL	3.50%	2.00%	3.50%	4.00%	4.00%
Required Resistance (Factored)	157.0 kN 35.3 kips	260.6 kN 58.6 kips	157.0 kN 35.3 kips	131.7 kN 29.6 kips	142.8 kN 32.1 kips
R_o Factor	1.7	1.3	1.7	1.5	1.7
Required Resistance (Unfactored)	92.5 kN 20.8 kips	200.6 kN 45.1 kips	92.5 kN 20.8 kips	131.7 kN 29.6 kips	84.1 kN 18.9 kips
Required Resistance with Toolbox	57.8 kN 13.0 kips	125.9 kN 28.3 kips	57.8 kN 13.0 kips	82.3 kN 18.5 kips	52.5 kN 11.8 kips
Required Retrofit Units	3.7 m 12.3 ft.	15.3 m 50.3 ft.	3.0 panels	2.0 frames	5 4-frame assemblies

Table 17 summarizes the first three modes of vibration for the retrofit options. The first, second and third mode represent the north-south, torsional and east-west modes of vibration, respectively.

Table 17: First three modes of vibration for retrofit options of school block

Retrofit 1: Blocked Shear Wall	**Retrofit 2: New Stucco Exterior**	**Retrofit 3: CLT Panels**	**Retrofit 4: Steel Moment Frame**	**Retrofit 5: Distributed Knee System**
0.58sec.	0.58sec.	0.58sec.	0.6sec.	0.57sec.
0.44sec	0.42sec.	0.45sec.	0.45sec.	0.49sec.
0.31sec	0.31sec.	0.31sec.	0.31sec.	0.31sec.

5.3.1 Retrofit #1: Add Shearwalls

One of the most efficient methods of increasing the lateral resistance of an existing light-frame wood structure is to strengthen its existing shearwalls. This can usually be done with minimal disruption to the building. The existing components can be utilized and the floor plan of the building can remain unchanged. To increase the capacity of existing shearwalls, extra nailing can be added to the existing panels and frames; however more typically, the walls will need to be resheathed. If the existing wall is unblocked, then new solid blocking will need to be installed at all sheathing edges. Also, hold-downs should be installed and possibly new anchor bolts if the existing foundation connections are inadequate. In some cases, a new grade beam will need to be installed below the shearwalls if the existing is insufficient for the higher loads that will be transferred from the stronger, retrofitted shearwalls. This will increase the cost of the retrofit and will require much more work and time.

Many older wood buildings have floor and/or roof diaphragms sheathed with shiplap or tongue and groove decking which may not provide enough capacity to resist seismically induced forces. A typical retrofit in this situation would be to resheath the diaphragm with new plywood. Flat metal straps (drag struts/chords) must also be added along the diaphragm perimeter and any drag lines. This will ensure forces are “collected” from the diaphragm and redistributed into the shearwalls.

In the case of the archetype school the existing shiplap diaphragm would be replaced with new plywood sheathing. Additional blocked plywood shear walls would be constructed, as shown in Figure 47, to provide additional strength and stiffness. The required resistance for the retrofit was estimated using the “LDRS Retrofit Design Results” SRG3 calculator. The target performance was assumed as 2% in 50 years’ non-exceedance of a maximum interstorey drift of 3.5%. The required resistance recommended is at least 10.9% of the total seismic weight of the structure or 57.8kN in both shaking directions by including the contribution of the existing lateral resistance. The “Toolbox” method was used to account for the contribution of the existing lateral systems. The walls are included in the numerical models. The blocked shear wall hysteretic model is shown in Figure 48. This model was calibrated to the previous residential and classroom testing, as described in Chapter 3 and Chapter 4. A total of 3.7m of the blocked shear wall is recommended in both shaking directions.

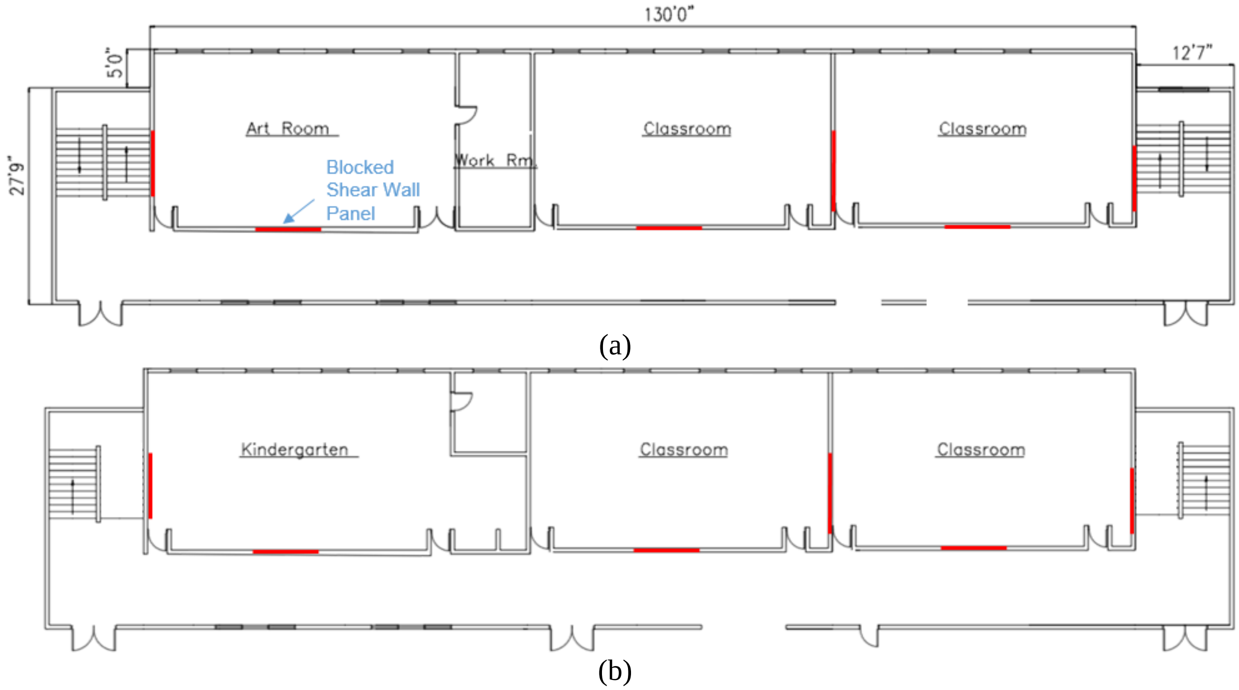


Figure 47: Blocked shear wall retrofit solution for Institutional Archetype for (a) Floor 1, (b) Floor 2

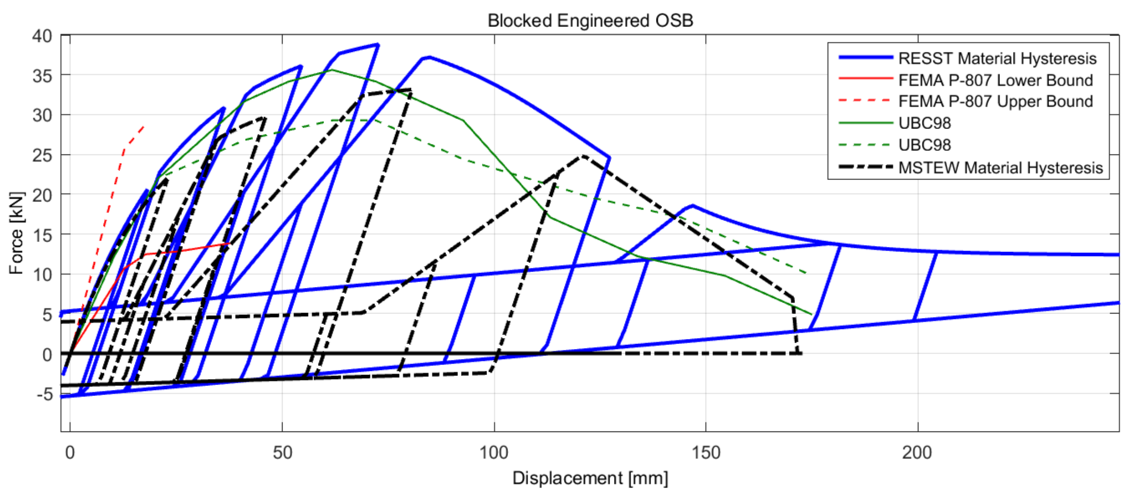


Figure 48: Blocked Engineered shear wall material model compared to experimental data (8ft. wall)

5.3.2 Retrofit #2: Add new stucco finishing for exterior walls

New stucco construction has been found to perform with high strength and ductility, as shown by studies conducted by Taylor et al. (2003) as part of the EQ-99 project and Sofali (2008). A possible retrofit solution could be to remove and replace the existing exterior finishes with new stucco construction. The material hysteresis model is shown in Figure 49 compared to the experimental data. Figure 50 shows the schematic of the retrofit for the first and second floor of the school block.

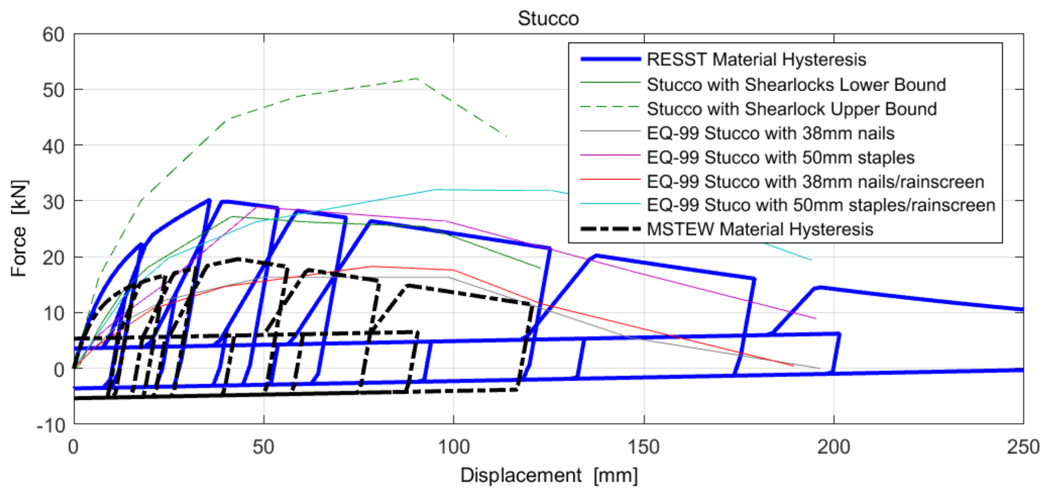
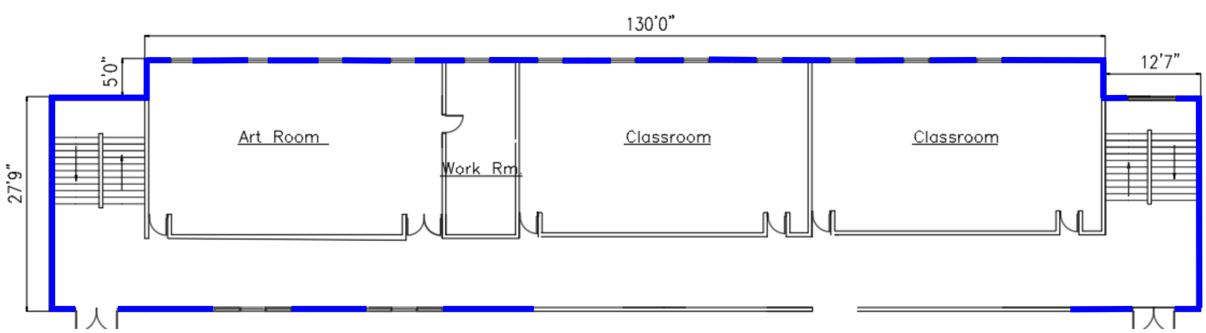
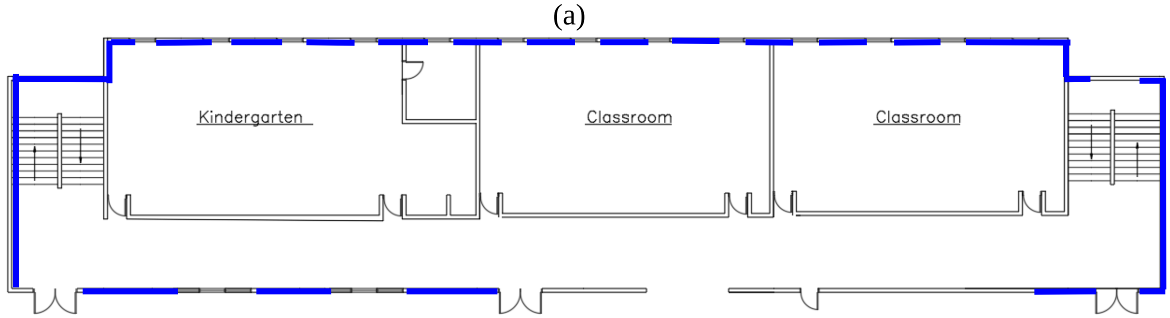


Figure 49: New stucco construction shear wall material model compared to experimental data (8ft. wall)



(a)



(b)

Figure 50: Proposed stucco retrofit solution for Institutional Archetype for (a) Floor 1, (b) Floor 2

5.3.3 Retrofit #3: CLT Panels.

Recent research has been focused on establishing CLT rocking walls as a viable retrofit solution for light-frame wood structures. CLT panels are commonly used as an engineering material in Europe, and are beginning to be more common in Canada and the United States. As part of the NEES-Soft project and proof of concept for the FEMA P-807 documents, a CLT rocking wall retrofit was tested numerically and experimentally. At the University of Alabama 610mm (2ft.) long CLT panels were tested by van de Lindt et al. (2013). The test hysteresis and a calibrated MSTEW material model is shown in Figure 51 (Jennings E. , et al., 2015) .

In the NEES-Soft project the CLT wall retrofit met the performance criteria by providing adequate strength to the soft storey (4% drift limit at a higher intensity than designed (S_a = 1.14g)), as well did not shift the damage to the upper storeys, as in accordance the relative stiffness method of FEMA P-807. The CLT panels were designed to rock and behave primary in rigid body motion. Vertically slotted holes at the top shear transfer connection were installed to allow for free rocking. The primary energy dissipation of the walls is in the mechanical connections, brackets and hold-downs (Popovski, Schneider, & Schweinsteiger, 2010). The 16mm diameter threaded rods at each end of the CLT walls were designed to resist the overturning moment and yield for ductility. A metal connector and 6.5 mm diameter self- tapping wood screws were used as shear connectors between the CLT panel and the foundation.

The CLT retrofit proposed provides an initial resistance as percentage of the weight equivalent to the engineered blocked wood shear walls retrofit solution (Retrofit #1). The schematic of the retrofit is shown in Figure 53. Three panels are recommended for each shaking direction for the first storey and second storey.

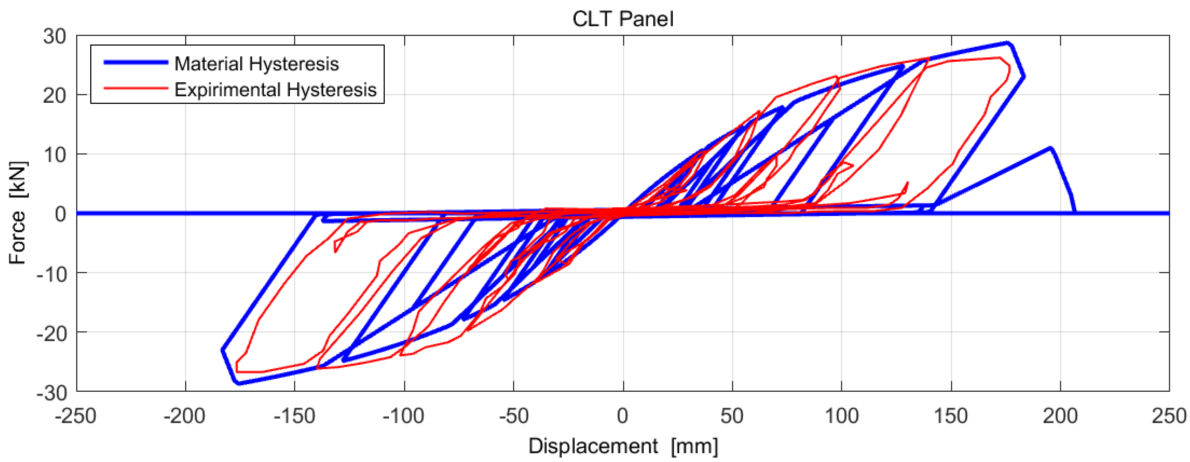


Figure 51: Experimental and Numerical Hysteresis for single CLT panel wall

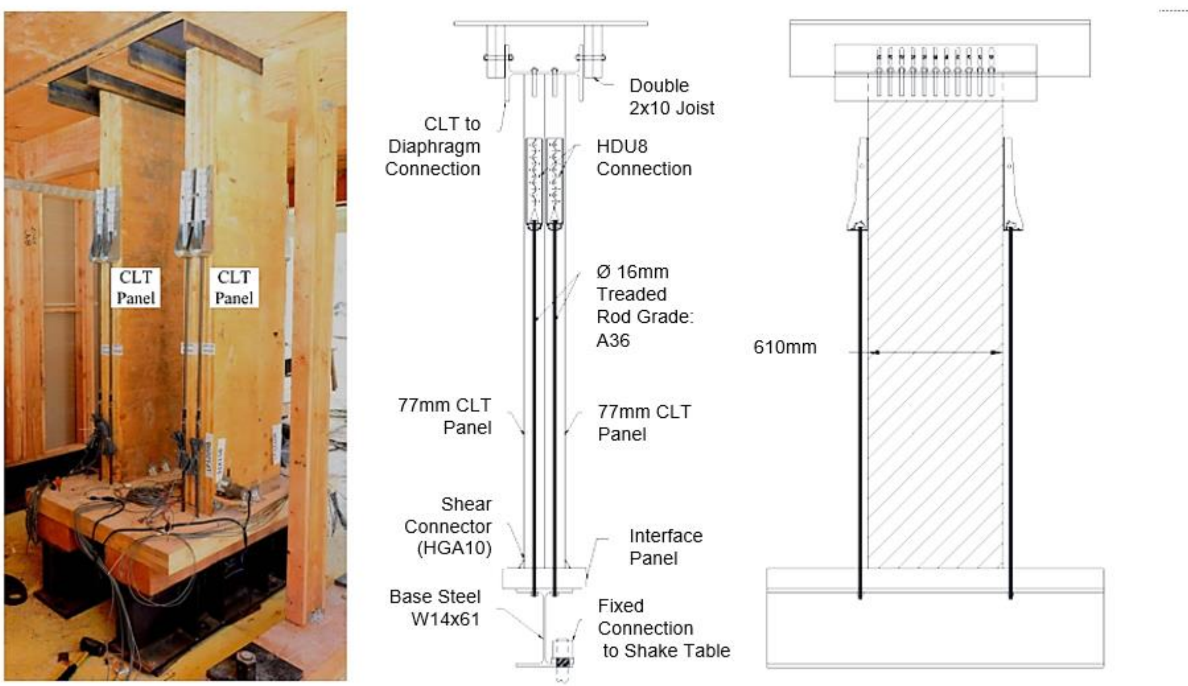


Figure 52: Cross Laminated Timber (CLT) rocking walls for retrofit solution: a) Installed in first storey for full-scale testing and b) elevation and design details (Bahmani, et al., 2014).

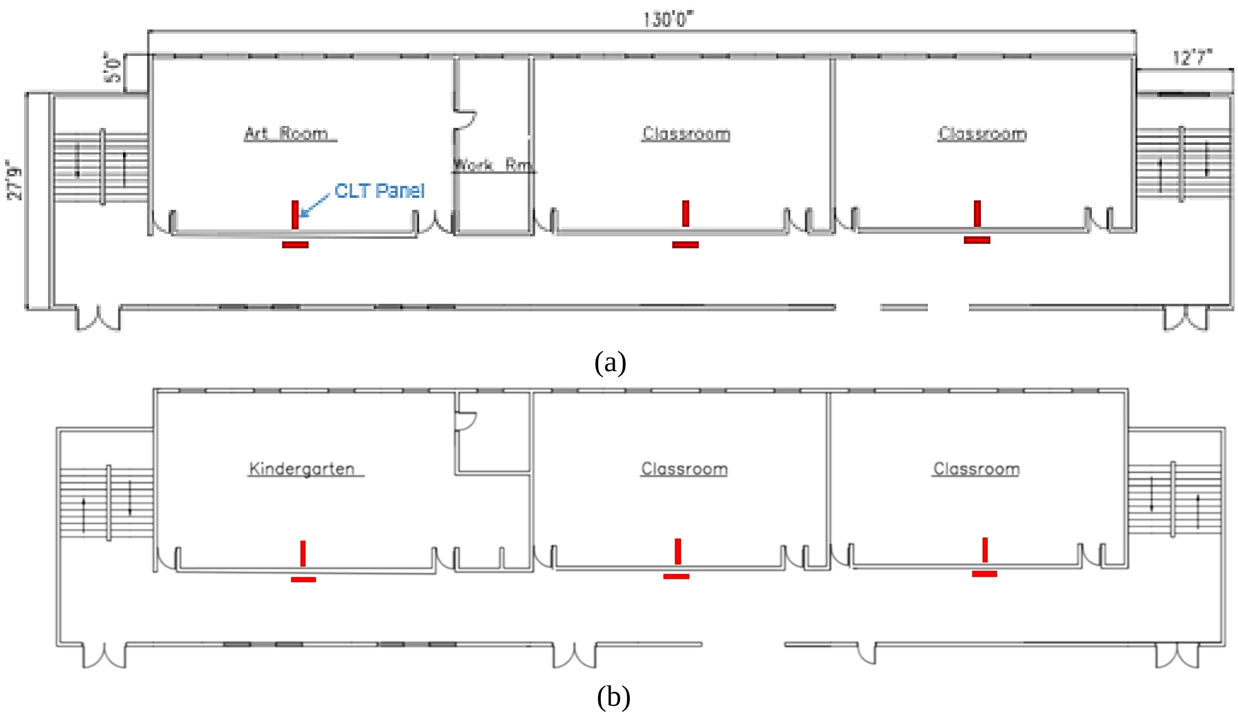


Figure 53: Proposed CLT retrofit solution for Institutional Archetype for (a) Floor 1, (b) Floor 2

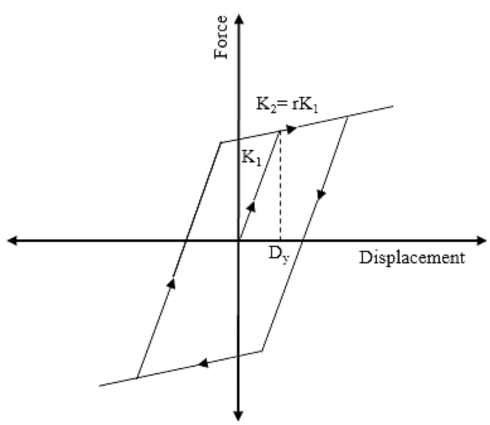
5.3.4 Retrofit #4: Steel Moment Frame

Special moment frames (SMF) and inverted moment frames are viable retrofit options for light-frame wood structures. Full-scale testing and analysis of the systems indicate the retrofit can meet performance requirements regards to strength, ductility and relative stiffness (Bahmani, et al., 2014). Pinned-ended SMFs, such as the Strong-Frame SMF system, as shown in Figure 54, were designed to be suitable as a retrofit solution. These frames have minimal interference with garage openings and other architectural details. The beam-to-column connections are designed so that the plastic hinge occurs away from the column and eliminates the potential for lateral torsional buckling of the beam. SMF are easily assembled on-site. It has snug-tight bolted connections that do not require specific training to install. There are no welded connections which reduces the cost associated with certified welders, field inspection and fire risk. Shear forces from the first-floor diaphragm can be transferred to the foundation by connecting the beam with a wood nailer to the floor diaphragm. Finally, the base connection is pinned, therefore no moment is produced at the column-to-foundation connection and foundation would only need to be retrofitted to resist the vertical and shear forces (Bahmani P. , 2015) (Pryor & Murray, 2013).

As the part of the NEES-Soft Project the FEMA P-807 methodology was implemented to design and retrofit the structure with a single steel special moment frame (SMF) in each orthogonal direction in the first storey only. The frames were placed to reduce torsion as much as possible without interfering with garage parking space. The first-storey SMF retrofit was capable of meeting FEMA P-807 requirements at a shaking intensity of 1.1g.

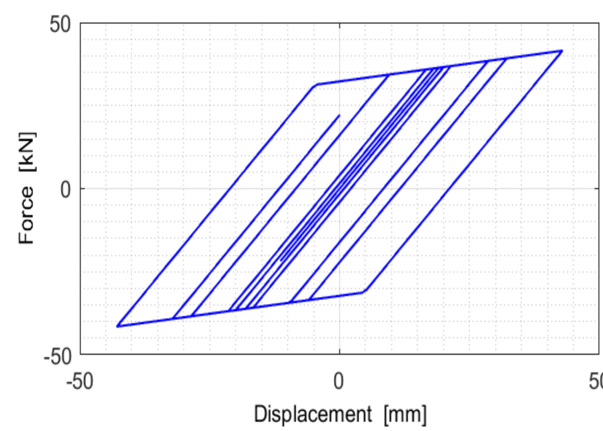
The proposed retrofit solution uses two SMF Simpson Strong Tie frames with W12x35 sized beams and W10x30 sized columns in both shaking directions, as shown in the schematic in Figure 55. The bilinear material hysteresis for the frames is shown in Figure 54.

The required resistance for the retrofit was estimated using the "LDRS Retrofit Design Results" SRG3 calculator. The ductile steel moment frame (S-9) is the most comparable prototype to the SMF. The target performance was assumed as 2% in 50 years' non-exceedance of a maximum interstorey drift of 4.0%. The retrofit recommendation is a resistance of at least 13.7% of the total seismic weight of the structure or 112.4kN in both shaking directions. Two SMF frames in both shaking directions are recommended for the retrofit in the first floor only, as shown in the schematic in Figure 56.



K_1	initial stiffness
F_0	post-yield stiffness ratio ($r = K_2/K_1$)
D_y	yield displacement

(a)



(b)

Figure 54: (a) Details for Bilinear material model (b) Bilinear material model for SMF for Col.:W10×30 Beam:W12×35SMF

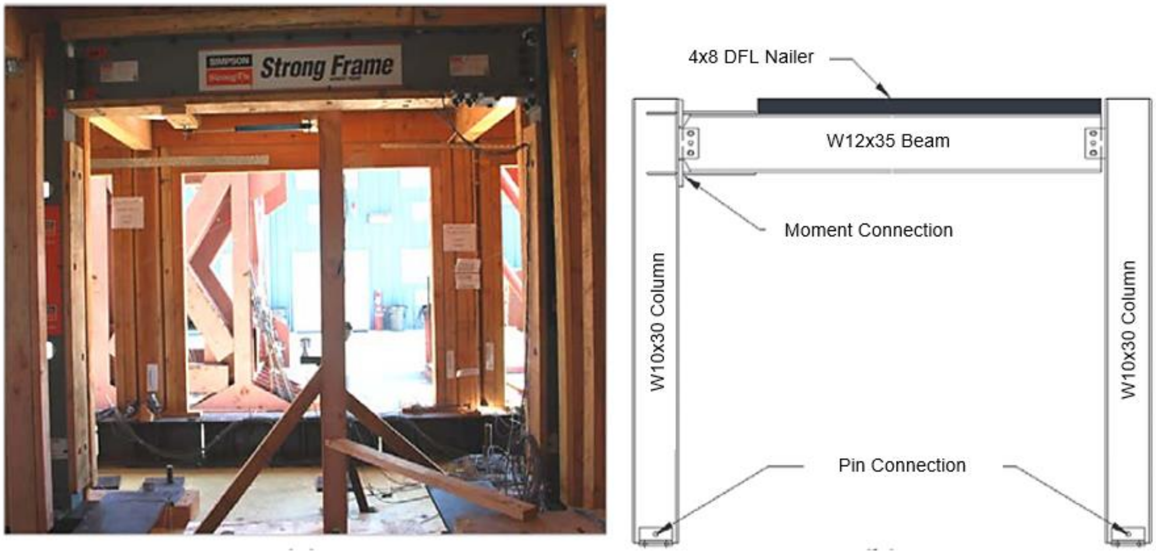
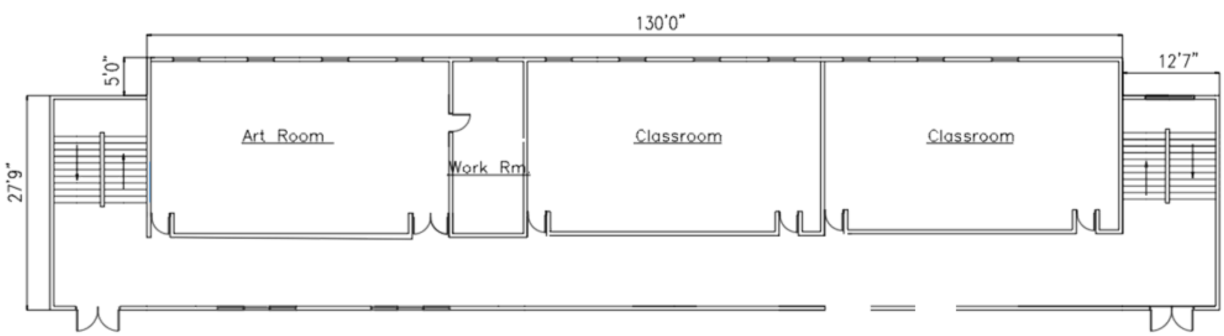
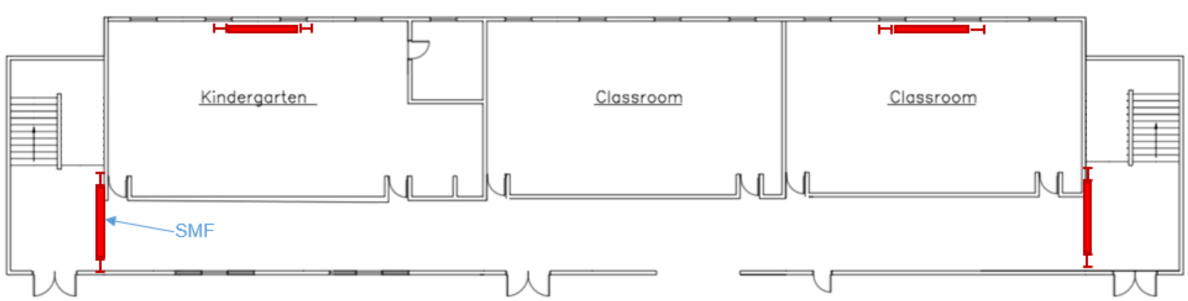


Figure 55: Strong Frame SMF a) Installed in first "soft" storey retrofit full scale test b) elevation of details (Bahmani, et al., 2014)



(a)



(b)

Figure 56: Proposed SMF retrofit solution for Institutional Archetype for (a) Floor 1, (b) Floor 2

5.3.5 Retrofit #5: Distributed Knee System

The DKB (Distributed Knee-Braced) system was tested as a possible retrofit solution for the NEES-Soft project for performance-based design and the FEMA-P807 Guidelines. This system would likely result in a reduction of retrofit design, construction time and cost. Each individual knee-brace frame is constructed using an additional stud connected to the existing stud, a Simpson Strong- Tie© A35 connector between the stud and bottom plate, a Simpson Strong-Tie© H2A between the stud to joist connection; and two new diagonal 2x4 wood members between the reinforced stud and joist fastened with 8d framing nails, as shown in Figure 57. The knee-brace connections to the studs and joists were designed at a lower capacity to protect the other framing members and connections by acting as the system fuse. Individual knee braced systems should be installed on several frames. This means that the existing walls and floor members that did not contribute to the lateral resistance are utilized and the foundation demands are reduced due to the distribution of the resistance (Gershfeld M. , et al., 2014).

Reversed-cyclic testing, numerical modeling, hybrid testing, and shake table testing was used to validate the performance of the DKB system. The system was found to provide sufficient strength at very high drift levels and has potential to be a viable retrofit solution with further development and research (Gershfeld M. , et al., 2014).

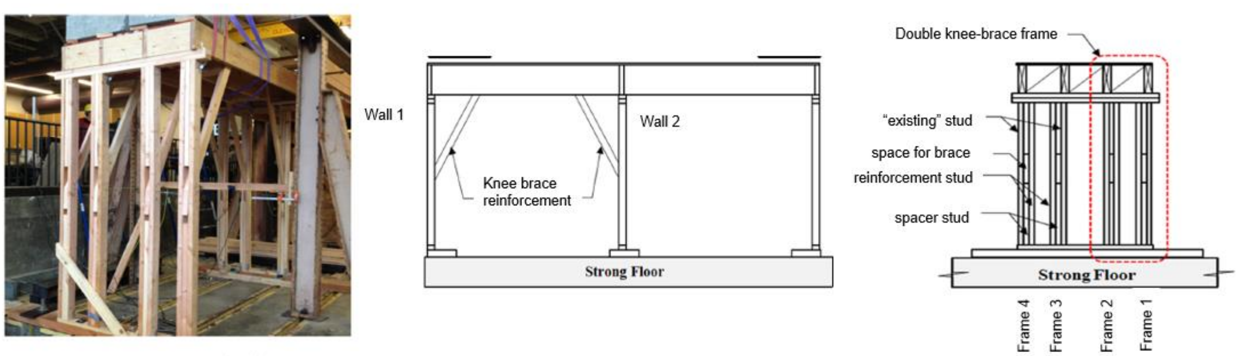


Figure 57: DKB System: a) Testing of system b) elevation view of details (Gershfeld M. , et al., 2014)

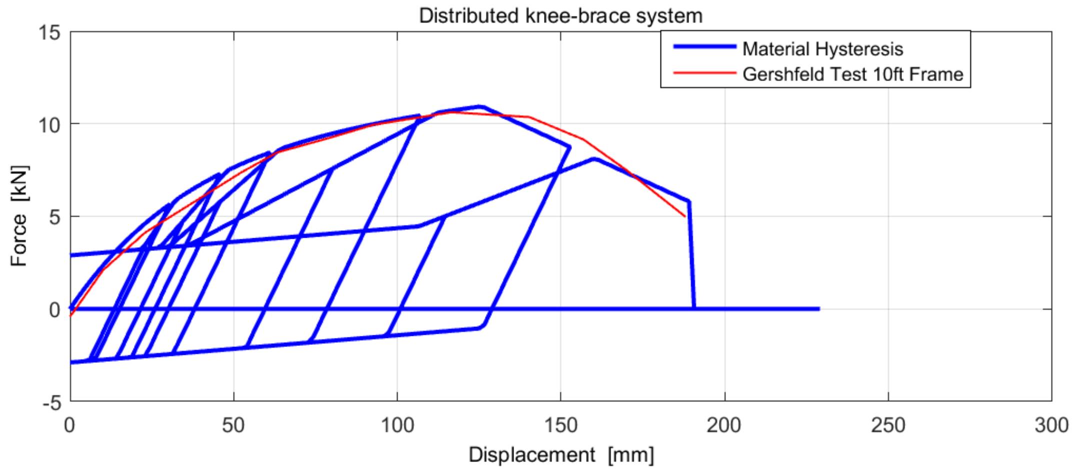
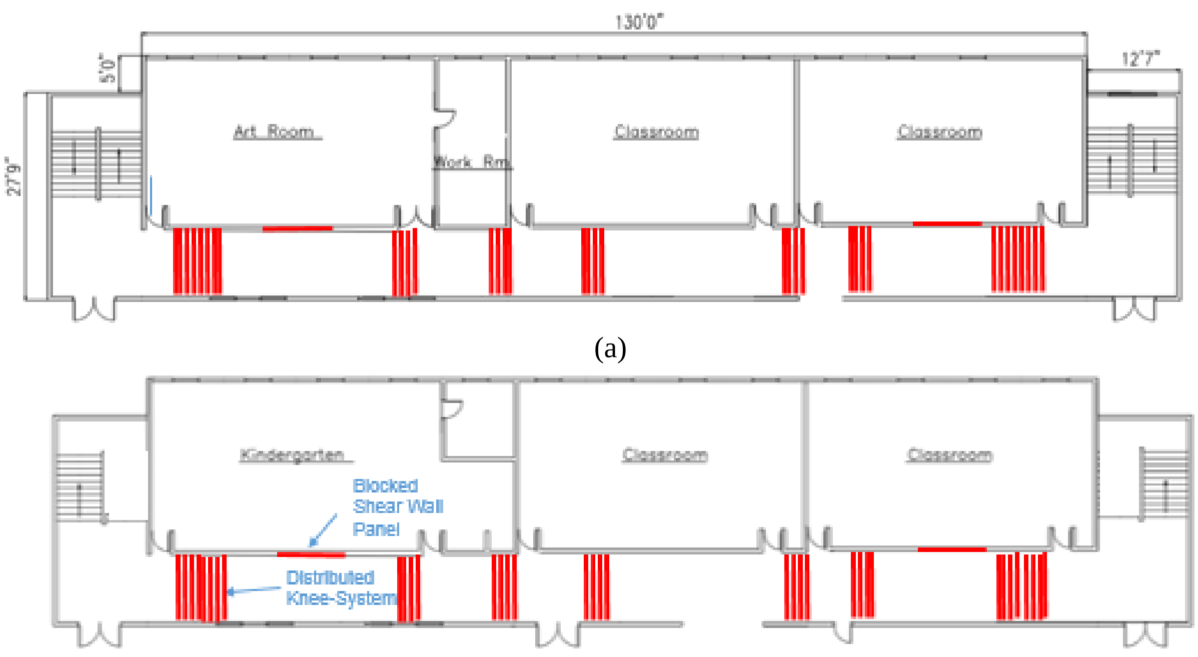


Figure 58: Experimental and numerical hysteresis for distributed knee system for 10ft. four-frame assembly

A combination of the distributed knee system and shear walls was recommended for the retrofit, as shown in the schematic in Figure 59. A total of 9 distributed frame assembles are recommended in the E/W shaking direction. The blocked shear wall length recommended is based on the 10.9%W resistance from the SRG3 calculator.



(b)

Figure 59: Proposed distributed knee system and blocked shear wall panel retrofit solution for Institutional Archetype for (a) Floor 1, (b) Floor 2

5.4 Ground Motion Selection and Scaling

To determine and compare the performance of the proposed retrofit solutions a time history analysis of a suite of two-dimensional ground motions scaled to four hazard levels in Vancouver, B.C. was completed. The suite of ground motions included crustal, subcrustal and subduction records. The intensity levels and corresponding targeted performance objectives are based on the recommendations of the NEESWood project team. Table 18 shows the exceedance probability and the return period of the hazard levels. Table 19 gives the expected performance in terms of the probability of non-exceedance of a determined maximum interstorey drift that correspond with different damage states at the four hazard levels. Christovasillis et al. (2007), the NEESWood Project Team (Pang et al. 2010) and Applied Technology Council Project 63 (ATC 2009) considered 7% interstorey drift to be a responsible, slightly conservative collapse criterion for wood frame buildings.

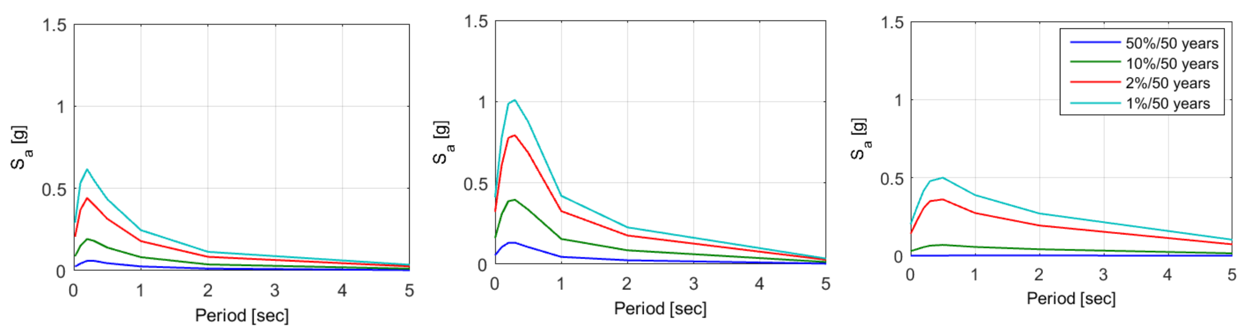
Table 18: Seismic Hazard for Level 1-4 performance objectives

	Exceedance Probability of Hazard	**Return Period**
Level 1 (Short Return Period)	50% / 50 years	72 years
Level 2	10 % / 50 years	475 years
Level 3 (Maximum Considered Earthquake)	2 % / 50 years	2475 years
Level 4 (Rare Events)	1 % / 50 years	4975 years

Table 19: Targeted performance and damage expectation at Hazard Level 1 – 4

Hazard Level	Target Peak Interstorey Drift	Non-Exceedance Probability of Target Drift	Damage Expectations
Level 1 (Short Return Period)	1%	50%	• Minor splitting and cracking of sill plates • Slight Sheathing nail withdraw • Hairline cracking of GWB • Diagonal crack propagation from door/window openings of GWB • Cracking at ceiling-to-wall interface
Level 2	2%	50%	• Permanent differential movement of adjacent panels • Corner sheathing nail pullout • Splitting/cracking of sill plates • Crushing of corners of GWB • Cracking of GWB taped/mud joints
Level 3 (Maximum Credible Earthquake)	4%	80%	• Severe splitting of sill plates and cracking of studs above anchor bolts • Partial withdraw and damage of sheathing nails • Severe damage\failure of anchor bolts • Separation of GWB corners in ceiling • Buckling of GWB at openings
Level 4 (Rare Events)	7%	50%	• Severe damage across edge nail line • Separation of sheathing • Vertical post uplift • Failure of anchor bolts • Large separates/dislodged of GWB

The suite of two-dimensional ground motions was selected and scaled to the different seismic intensity levels for each type of earthquake source separately. Figure 60 shows the spectra for the Level 1- Level 4 earthquake hazards for the three earthquake sources. The seismic hazard data for Vancouver, British Columbia was generated from EZ-RISK analysis (Risk Engineering 2008).



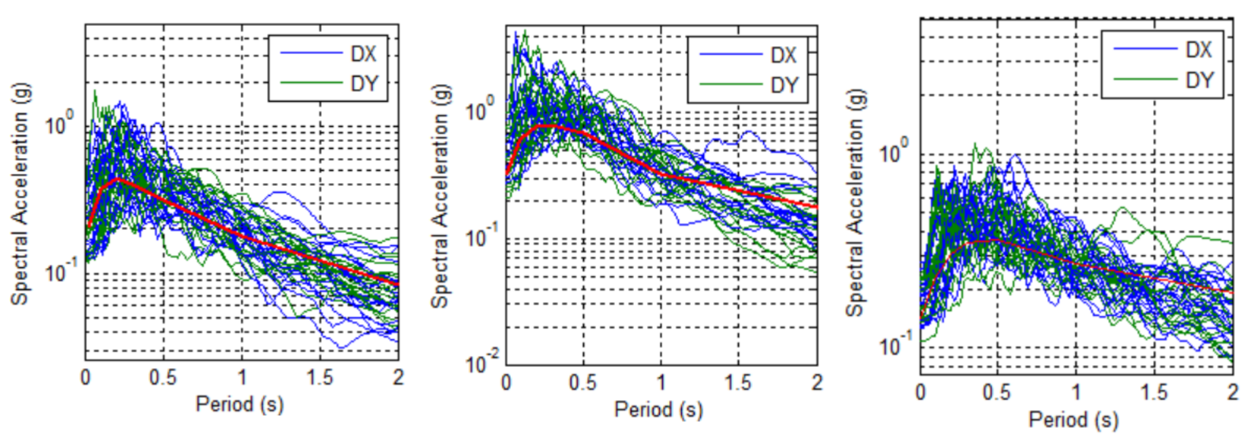
(a) (b) (c)

Figure 60: Vancouver, B.C. Level 1 – 4 Spectral Acceleration (5% damping) for (a) crustal, (b) subcrustal, (c) subduction earthquakes

The ground motions were selected so that the scaled records were above 70% of the target for the period range 1.0s to 2.0s for the 2% in 50-year hazard level. The ground motions records were chosen from the following sources: PEER-NGA database (Chou et al., 2008); K-NET (Kinoshita 1998); KiK-net (Aoi et al. 2000); and COSMOS database (Archuleta et al. 2006). The geomean of the two spectra acceleration horizontal ground motion components was calculated as:

$$SA_{GM} = \sqrt{SA_{NS} \times SA_{EW}} \qquad [1]$$

The scaling factor was determined by minimizing the mean squared error (MSE) between the targeted spectra acceleration (SA) hazard levels and the SA of the geomean of the ground motions for the period range 0.1-1.5sec. This procedure is in accordance to the recommendations from NBCC (2015) and the technical report for PEER Ground Motion Database (PEER, CALTRANS, CGS, 2010). Figure 61 shows the scaled crustal, subcrustal and, subduction motions in the x and y direction of shaking and the target spectra.



(a) (b) (c)

Figure 61: Vancouver, B.C. 2% in 50 years' spectra for (a) crustal, (b) subcrustal, (c) subduction earthquakes

5.5 Results for Bilinear Model

The peak interstorey drift distributions, as shown in Figure 62, for the existing structure and the retrofit options are based on the results from the SAPWood 2D NLTHA. The distributions are a lognormal fit of the maximum ultimate interstorey drift data. The non-exceedance probability at the design drift limits and medium drift level at the four performance levels (50% in 50 years, 10% in 50 years, 2% in 50 years, and 1% in 50 years hazard levels) are summarized in Table 20. The aspect ratio for the lateral dimension/height is approximately equal to 1, therefore the dynamic behavior would be primarily shear dominated and the biaxial model should capture the principal lateral behavior of the building block. The median peak drifts at Levels 1 and 2 were considerably lower than the 1 and 2% drift limits. The medium drift at Level 3 was lower than the objective for the existing building. Retrofit 2, 3, 4, and 5 pass all four performance objectives. Retrofit 1 failed the 3rd performance objective.

Table 20: Summary of 2D NLTHA Results for Existing School Block and Retrofit Options (Red=Fail, Green=Pass)

Seismic Hazard	Level	1	2	3	4
	Ground Motion	**50%/50 yr.**	**10%/50 yr.**	**2%/50 yr.**	**1%/50 yrs.**
Performance Expectation	**Drift Limit**	**1**	**2**	**4**	**7**
	Non-exceedance Probability Limit	**50**	**50**	**80**	**50**
Existing School Block	**Median drift**	0.08	0.56	2.56	4.0
	Non-exceedance Probability at drift limit	96.5	97.0	73.0	76.0
	Pass				
Retrofit 1 School Block	**Median drift**	0.08	0.53	2.34	3.8
	Non-exceedance Probability at drift limit	96.8	97.6	77.5	77
	Pass				
Retrofit 2 School Block	**Median drift**	0.06	0.39	1.59	2.75
	Non-exceedance Probability at drift limit	97.0	99.0	96.0	90.0
	Pass				
Retrofit 3 School Block	**Median drift**	0.08	0.54	2.3	3.6
	Non-exceedance Probability at drift limit	96	97	80	80
	Pass				
Retrofit 4 School Block	**Median drift**	0.08	0.44	1.39	2.16
	Non-exceedance Probability at drift limit	97	99	98	98
	Pass				
Retrofit 5 School Block	**Median drift**	0.07	0.52	2.09	3.43
	Non-exceedance Probability at drift limit	97	97	86	83
	Pass				

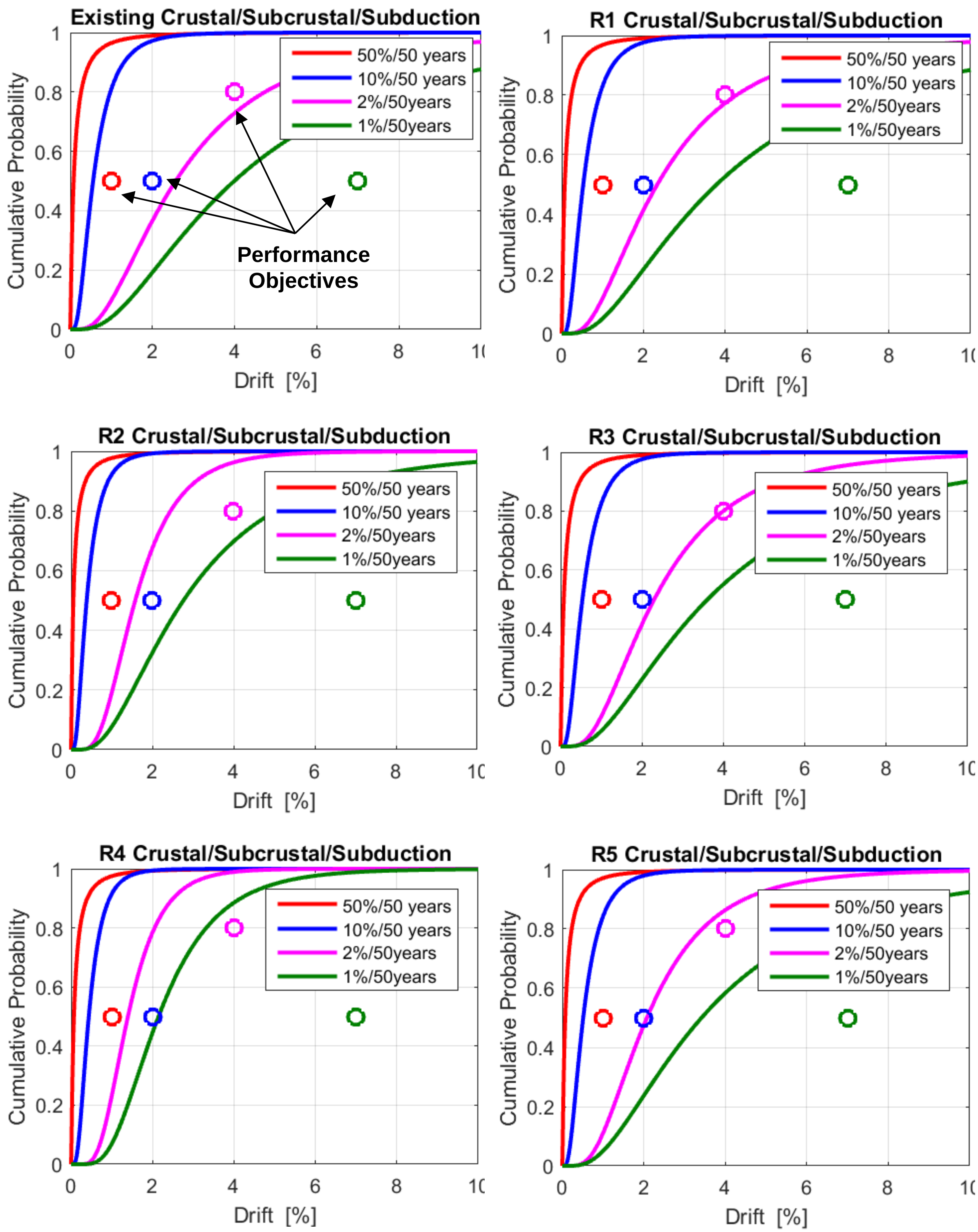


Figure 62: Peak interstorey drift distributions for the Existing Structure, Retrofit 1, Retrofit 2, Retrofit 3, Retrofit 4, Retrofit 5

5.6 Results for 3D Model

A NLTHA of the Timber3D models were run at the 2% in 50 years' hazard level to confirm the retrofitted building performance. A comparison of non-exceedance probability distributions (lognormal fit) for the existing building block and the retrofit options are shown in Figure 63. The non-exceedance probability at the design drift limit (4.0% drift) and medium drift level at the 2% in 50-year performance level are summarized in Table 21. The retrofit options met the requirements of the third performance criteria using the 3D model

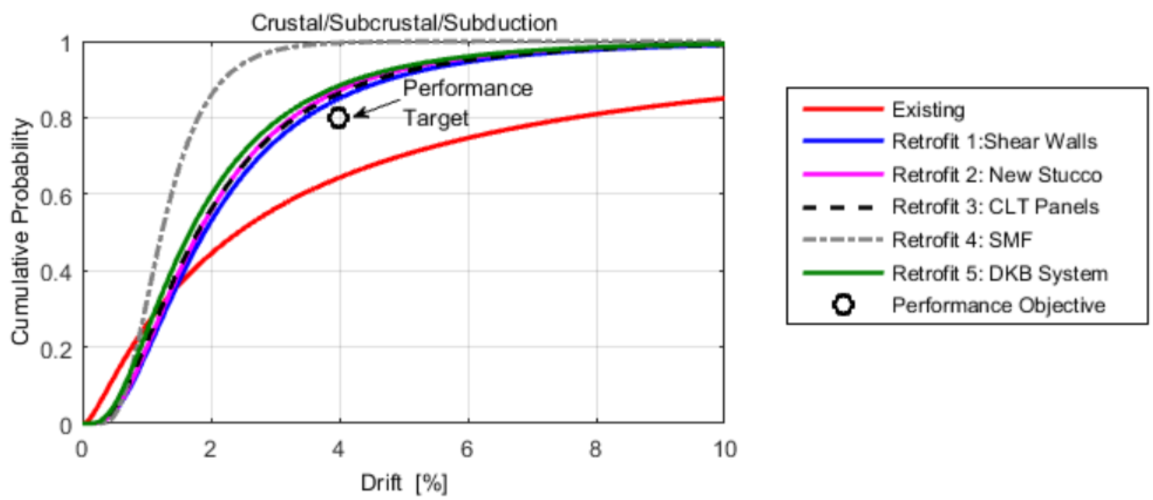


Figure 63: Comparison of non-exceedance probability distributions from NLTHA of Existing Building and Retrofit Options

If the existing stucco is not included in the assessment of the existing and retrofitted buildings the seismic response may change. NLTHA was run for the existing structure without stucco at the 2% in 50-year hazard level; the median drift was 2.45 and the non-exceedance probability at the design drift was 64%. The first, second and third modes of vibration were 0.60sec., 0.46sec. and 0.38sec., respectively. It would be important to view the condition of existing the stucco before including it in the model; if the connection between the stucco and walls is significantly deteriorated it would not contribute to the shear resistance of the structure.

The three-dimensional model predicted higher medium drift levels and a non-exceedance probability at the design drift than the biaxial model. The difference, however, was not significant; the building's dynamic behaviour is primarily shear dominated.

Table 21: Summary of 3D NLTHA Results for Existing School Block and Retrofit Options (Red=Fail, Green=Pass)

Seismic Hazard	Level	3
	Ground Motion	2%/50 yr.
Performance Expectation	Drift Limit	4
	Non-exceedance Probability Limit	80
Existing School Block	Median drift	2.40
	Non-exceedance Probability at drift limit	65.5
	Pass	
Retrofit 1 School Block	Median drift	1.91
	Non-exceedance Probability at drift limit	85.0
	Pass	
Retrofit 2 School Block	Median drift	1.81
	Non-exceedance Probability at drift limit	87.0
	Pass	
Retrofit 3 School Block	Median drift	1.80
	Non-exceedance Probability at drift limit	86.0
	Pass	
Retrofit 4 School Block	Median drift	1.24
	Non-exceedance Probability at drift limit	99.0
	Pass	
Retrofit 5 School Block	Median drift	1.67
	Non-exceedance Probability at drift limit	88.5
	Pass	

5.7 Collapse Mechanism

The deformed shape of the existing building block at incipient of collapse is shown in Figure 64. The building collapsed in a side-sway mechanism; second order effects such as p-delta effect propagated the collapse of the structure. The first floor acted as a soft-storey, where the first floor deformed significantly more than the upper floor and the upper floor remained nearly elastic. The 2001 Geilo earthquake subcrustal motion scaled to 2% in 50 years Vancouver hazard caused collapse. The PGA of the ground motion was 0.5g. Figure 64 and Figure 65 shows the time-history response of the displacement at the top of the first story in the N-S shaking and E-W shaking direction. The time history responses for nodes on the opposite corners of the building block, in blue and red in Figure 65, show nearly equivalent lateral response, therefore the diaphragm behaved near rigid and torsion did not influence the response.

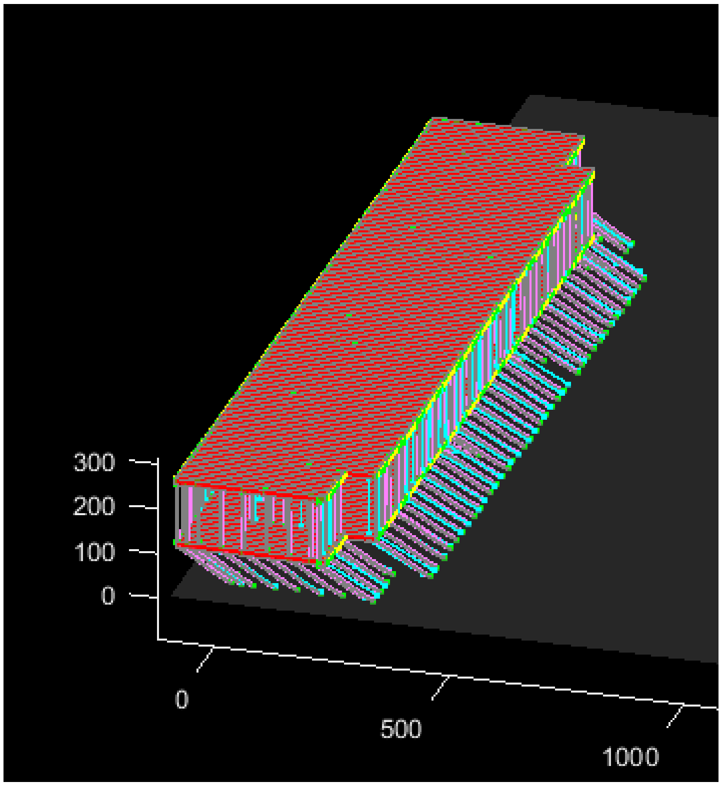
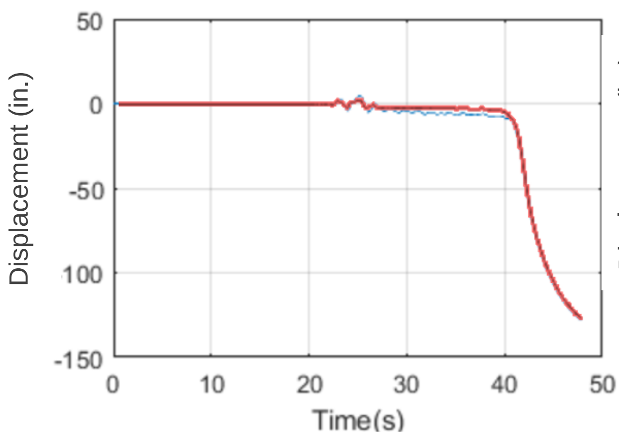
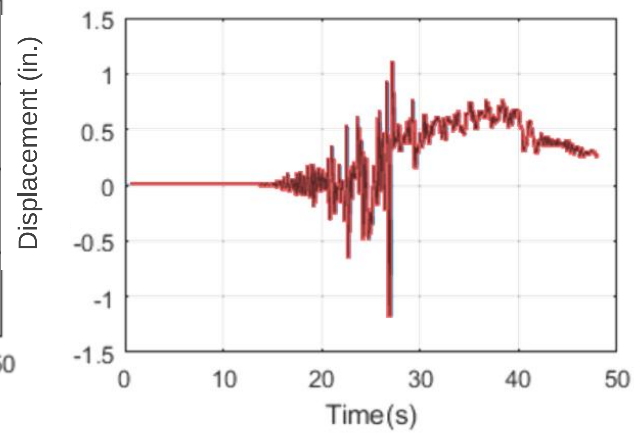


Figure 64: Deformed shape at incipient of collapse of school block



(a)



(b)

Figure 65:Time-History response of displacement at top of first storey (a) the N-S direction (b) the E-W direction

5.8 Discussion

The biaxial model has been documented as being able to predict the lateral response of the building at low drift levels. Therefore, to evaluate the performance of the structure at serviceability levels (Level 1 and Level 2) the biaxial model is recommended to reduce the modeling and computational time and effort. The Timber3D model can predict the response when deformations are large, close to and at collapse. For collapse prevention checks the Timber 3D should be used. It is recommended for the life safety performance level (Level 3) the target peak interstorey drift is limited to the displacement at or close to the peak force in the material backbone curves if the biaxial model is used for the prediction. If target peak interstorey drift is past within the degrading portion of the backbone curve the response would be better captured with the Timber3D model. To better characterise the expected probability of collapse of the retrofit options a full incremental dynamic analysis of the models should be completed.

5.1 Summary

The numerical models of the retrofit options provide an objective method evaluate the expected performance of the structure in different seismic events. The modeling methods validated with the shake table results in Chapter 3 and Chapter 4 were applied to achieve a reasonable estimation of the lateral behaviour in the design earthquake. The existing structure would most likely be heavily damaged and has a probability of experiencing a side sway collapse at the 2% in 50 year hazard level earthquake greater than 20%. This means that the structure should be retrofitted to achieve a more acceptable expected performance.

The proposed retrofits met the performance objectives based on the numerical modeling results. The special steel moments frame had better performance at the 2% in 50years (based on the Timber3D analysis). A complete performance-based loss estimation of the different retrofit options would provide a more comprehensive comparison of the retrofits, however is not the focus of this study.

Chapter 6: Summary and Conclusions

6.1 Summary

In this thesis, the use of numerical models to predict and evaluate the seismic performance of light-frame wood structures was investigated.

A three-dimensional model of a two-storey, light-frame timber was created in the numerical modeling program, Timber3D. This model was validated with the shake table test conducted at the University of British Columbia (UBC) of two-story full-scale light-frame timber houses as part of the Earthquake-99 Testing Program. A variety of sheathing configurations and detailing were used to represent common construction practices in decades prior to and after the implementation of seismic guidelines for light-frame wood structures. The material hysteretic modelling parameters for the wood walls were based on experimental testing and recommendations in literature. The strength and stiffness contribution from the shear walls with openings was accounted for using the FEMA P-807 opening factor. The non-structural sheathing material was included in the model and significantly changed the lateral behaviour of the structure. The models could predict the time-history response of the drift with responsible accuracy over a wide range of drift levels from serviceability to near collapse.

A prediction for the dynamic behavior of a one-storey light-frame structure was completed in two parts: (1) a detailed M-CASHEW2 model and (2) a global Timber3D model. A full-scale wood frame classroom was tested on the linear shake table at the UBC EERF facility. The testing program was performed to evaluate the effect of non-structural finishing, openings in shear walls and ground motion duration on the seismic performance of light-frame wood structures. The full-scale classroom was subjected to a long duration motion recorded in the 2011 M_w = 9.0 Tohoku, Japan earthquake scaled to 70%, 100% and 100% for the first, second and third run, respectively. In the detailed numerical model (M-CASHEW2) each nail, stud, sheathing panel, and hold-down was modeled explicitly. Cyclic and monotonic analysis were completed to characterize the lateral behavior of the structure and time history analysis was completed. The hysteretic and time-history response of the structure was accurately predicted for the first two runs. The degradation in the material models and the damping characteristics may need to calibrated to the third run where the structure was significantly damaged and was at the onset of collapse. A preliminary study investigating the effect of ground motion duration was completed using the validated detailed numerical model. The results suggested that for spectrally equivalent short duration and long duration ground motion pairs the structure would experience more damage and higher absolute drift during a long duration seismic event. The global Timber3D model was also used to complete a time-history analysis of the structure. Several models were created to capture the upper and lower bound predictions of the lateral response. The segmented approach, where only the solid shear walls were modeled overestimated the absolute interstorey drift. The cyclic response of the M-CASHEW2 model was fit to the RESST material model, simplified to a shear spring and inserted into the global Timber3D model. This model represented the upper bound response and underestimated the global drift when compared to the experimental results. The structure was also modeled using the FEMA P-807 openings factors to account for the contribution of the strength and stiffness of the openings. This global Timber3D numerical model could predict the hysteretic and time history response with considerable accuracy and was validated with the EQ-99 full-scale house shake table testing program. The model included the non-structural sheathing walls. Due to the simplifications of the global and hysteretic material model it is difficult to capture the accumulative damage from previous runs. The structure experienced high drift levels close to collapse by the third run. When the structure is at high drift levels hysteretic damping governs damping within the structure.

The seismic behavior of a two-storey wood frame school block was also investigated. The structure represents typical 1950-1960 light-frame wood construction in the lower mainland of British Columbia. The performance of the building block was estimated with the Seismic Retrofit Analyzer Version 3.0 (SRG3) as part of the British Columbia Ministry of Education Seismic Mitigation Program. Several retrofit options were proposed and investigated, including (1) a shear wall retrofit, (2) an exterior stucco retrofit, (3) CLT panel walls, (4) special steel moment frames, and (5) a distributed knee-brace system. The building block was modeled using (i) a biaxial model in SAPWood and (ii) a three-dimensional model in Timber3D. The performance of the structure was evaluated by the non-exceedance probability at the design drift limits (1%, 2%, 4%, 7%) for the four hazard levels (50% in 50 years, 10% in 50 years, 2% in 50 years, and 1% in 50 years' hazard levels). The SAPWood and the Timber3D model showed that the retrofit options met the target performance criteria.

6.2 Conclusion

The main goal of the study was to investigate the use of numerical models to predict the seismic performance of light-frame wood structures. Based on the body of work presented in this thesis it can be concluded that:

1. The three-dimensional Timber3D model can give accurate predictions of the performance of light-frame wood buildings. Models with different sheathing types, construction practices and opening configurations were validated with experimental testing.

2. The sheathing above and below openings and non-structural finishing significantly contribute to the strength and stiffness of the structure and should be included for performance based assessment and design. The opening factor included in the FEMA-P807 guidelines gives a reasonable prediction of the strength contribution for a global model.

3. The detailed M-CASHEW2 model can give accurate predictions of the dynamic response of a light-frame wood wall. The model could capture the accumulated degradation after multiple shake-table runs.

4. The global Timber3D model can be used to predict the expected seismic behaviour of light-frame wood structures for a range of performance objectives. The SRG analyser tool can be used to initially evaluate an existing structure and determine the strength requirements needed for the design retrofit options. The Timber3D analysis results indicated that the use of wood structural panels, new stucco envelope, CLT panels, steel SMF's, and DKB systems can be an effective technique to retrofit an existing, structurally deficient wood-frame building.

6.3 Contributions

The contributions of this thesis to the field of structural and earthquake engineering include validating a numerical model with full-scale shake table tests for a variety of construction types typical to North America, and the lower Mainland of British Columbia. Hysteretic parameters for wood-frame walls for new and archaic materials were defined based on compiled experimental data and referenced recommendations. The modeling methodology for combining materials and accounting for the openings was detailed and validated with the experimental results with two separate full-scale testing programs.

The validated modeling methodology was then applied to predict the seismic performance of an existing school block in Vancouver, British Columbia built in the 1950s with archaic materials and construction practices. Several retrofitting techniques for the seismically deficient building were proposed and assessed. The simplified modeling tool used in the performance-based Seismic Retrofit Guidelines (SRG) (Ventura, Bebamzadeh, Fairhurst, Taylor, & Fiam, 2015) was verified compared to the more complicated three-dimensional model. The collapse mechanism and deformation limits of wood-frame buildings subjected to earthquakes were investigated by conducting non-linear time history analysis.

The detailed numerical model could predict give a good prediction of the response of a full-scale classroom test with long duration study. A preliminary study suggests that a structure will experience a higher level of drift and degradation in a long duration seismic event comparted to a short duration event.

6.4 Suggestions for Future Work

1. Complete a more comprehensive analysis of ground motion duration effect. Additional shake table tests using short and long duration motions and different sheathing configurations should be completed. The testing should be complemented with detailed (M-CASHEW2) and global (Timber3D) numerical modelling and a full incremental dynamic analysis to investigate the effect of ground motion duration on the likelihood of collapse.

2. A sensitivity analysis on the cyclic behaviour of the nails for the M-CASHEW2 detailed modeling parameters should be completed to capture the degradation from multiple consecutive ground motions. A after-shock study could be completed with the validated model.

3. Refining and calibrating modeling technique of wall systems with opening in M-CASHEW2 and Timber3D for additional testing completed on the full-scale classroom.

4. Investigate the performance of supplemental damping devices as a retrofit technique. The Timber3D source code would need to be altered to implement this in the existing Timber3D model.

5. Complete a complete incremental dynamic analysis of the school block 3D model to estimate collapse probabilities and evaluate collapse drift and mechanisms.

6. Compare the performance and collapse probability of designs based on the Direct Displacement Method, the School Retrofit Analyzer, the FEMA P-807 Guidelines, the Performance Based Design Method and the current code-specified force-based procedures.

7. Investigate using performance-based loss estimation framework to provide quantitative comparisons of various building types/retrofit options.

References

Adams, J., Halchuck, S., Allen, T., & Rogers, G. (2015). Canada's 5th Generation Seismic Hazard Model, As Prepared for the 2015 National Building Code of Canada. *11th Canadian Conference of Earthquake Engineering.* Victoria, Canada .

Adebar, P., Davis, H., Spilchen, W., & Sacks, A. ”. (2003). *US Patent No. 6668501*.

AISC . (2005). *Seismic Provisions for Structural Steel Buildings, Including Supplement No. 1, ANSI/AISC 341-05, ANSI/AISC 341s1-05.* Chicago: American Institute of Steel Construction.

AISC. (2011). *Seismic Provisions for Structural Steel Buildings, ANSI/AISC 341-10* . Chicago, Illinoi: American Institute of Steel Construction.

APEGBC. (2013). *Seismic Retrofit Guidelines, 2nd Edition.* Burnaby, British Columbia, Canada: Association of Professional Engineers and Geoscientists.

Applied Technology Council, ,. (2008). Here today—Here tomorrow, earthquake safety for soft-story buildings. *San Francisco Dept. of Building Inspection*. San Francisco , U.S. Retrieved from San Francisco Dept. of Building Inspection.

ASCE. (2013). *Seismic Evaluation of Existing Buildings, ASCE/SEI 41-13.* Reston, Virginia: American Society of Civil Engineers .

Asiz, A., Chui, Y. H., Smith, I., & Zhou, L. (2009). *Development of Advanced System Design Procedures for the Canadian Wood Design Standard.* Fredericton, New Brunswick,: Value to Wood Project.

Bahmani, P. (2015). *Performance-Based Seismic Retrofit (PBSR) Methodology for Multi-Story Buildings with Full-Scale Experimental Validation.* Colorado State University.

Bahmani, P., van de Lindt, J. W., Gershfeld, M., Mochizuki, G. L., & Pryor, S. E. (2014). Experimental Seismic Behavior of a Full-Scale Four-Story Soft-Story Woodframe Building with Retrofits I: Building Design, Retrofit Methodology, and Numerical Validation. *J. Struc. of Eng.*

Bahmani, P., van de Lindt, J. W., Gershfeld, M., Mochizuki, G. L., Pryor, S. E., & Rammer, D. (2014). Experimental Seismic Behavior of a Full-Scale Four-Story Soft-Story Woodframe Building with

Retrofits I: Building Design, Retrofit Methodology, and Numerical Validation. *Journal of Structure Engineering*, 10.1061/(ASCE)ST.1943-541X.0001207 , E4014003.

Bahmani, P., van de Lindt, J. W., Gershfeld, M., Mochizuki, G., Pryor, S., & Rammer, D. (2014). Experimental Seismic Behavior of a Full-Scale Four-Story Soft-Story Woodframe Building with Retrofits I: Building Design, Retrofit Methodology, and Numerical Validation. *J. of Struct. Eng*.

Blasetti, A., Hoffman, R., & Dinehart, D. (2008). Simplified hysteretic finite-element model for wood and viscoelastic polymer connections for the dynamic analysis of shear walls. *J. Struct. Eng., 134*, 77-86.

Bruneau, M. (1990). Preliminary report of structural damage from the Loma Prieta (San Francisco) earthquake of 1989 and pertinence to Canadian structural engineering practice. *Canadian Journal of Civil Engineering, 17*(2), 198-208.

CAPSS. (2010). *A Community Action Plan for Seismic Safety*. Redwood City, California: Applied Technology Council.

Ceccotti, A., & Karacabeyli, E. (2000). Dynamic analysis of nailed wood-frame shear walls. *12th World Conference on Earthquake Engineering (12WCEE)*, (pp. 1-8). Auckland, New Zealand.

Chandramohan, R., Baker, J. W., & Deierlein, G. (2016). Impact of hazard-consistent ground motion duration in structural collapse risk assessment . *Earthquake Engineering & Structural Dynamics* .

Chandramohan, R., Baker, J. W., & Deierlein, G. G. (in press). Quantifying the influence of ground motion duration on structural collapse capacity using spectrally equivalent records. *Earthquake Spectra.*

Chandramohan, R., Baker, J. W., & Deierlein, G. (in press). Quantifying the influence of ground motion duration on structural collapse capacity using spectrally equivalent records. *Earthquake Spectra*.

Chen, Z., Nott, A., Chui, Y. H., Doudak, G., & Ni, C. (2014). Experimental Study on the Contribution of GWB to the Lateral Performance of Wood Shearwalls. *World Conference on Timber Engineering (WCTE 2014).* Quebec City, Canada.

Christovasilis, I. (2010). *Numerical and Experimental Investigations of the Seismic Response of Light-Frame Wood Structures.* University of Buffalo.

Christovasilis, I., Filiatrault, A., & Wanitkorkul, A. (2007). *Benchmark Test Seismic Testing of a Full-Scale Two-Story Light-Frame Wood Building: NEESWood* . NEESWood Project Report .

Collins, M., Kasal, B., Paevere, P., & Foliente, G. C. (2005). Three- dimensional model of light frame wood buildings. I: Model description. *J. Struct. Eng.*, 131:4(676), 676–683.

CUREe. (1998). *CUREe-Caltech Woodframe Project Newsletter*. Retrieved from http://www.curee.org/projects/woodframe/

CWC. (2010). *Wood Design Manual 2010.* Ottawa, ON.: Canadian Wood Council.

Dolan, J. D. (1989). *The dynamic response of timber shear walls.* Vancouver, Canada: Ph.D thesis, University of British Columbia.

EERF. (2009). *Innovative retrofit strategies for wood frame walls, Technical Report, Seismic Retrofit Guidelines.* Vancouver: Earthquake Engineering Research facility, University of British Columbia.

EERI. (1996). *Northridge Earthquake of January 17, 1994 Reconnaissance Report, Vol. 2, Holmes, W. and Somers, P., editors.* . Oakland, California: Earthquake Engineering Research Institute.

FEMA. (2006). *Homebuilders' Guide to Earthquake-Resistant Design and Construction.* Washington, D.C. : National Institute of Building Sciences .

FEMA. (2009). *Quantification of Building Seismic Performance Factors, FEMA P-695* . Washington, D.C: Federal Emergency Management Agency.

FEMA. (2012). *Seismic Evaluation and Retrofit of Multi-Unit Wood-Frame Buildings With Weak First Stories.* Washington, D.C.: FEMA P-807,.

Filiatrault, A., Christovasilis, I., Wanitkorkul, A., & van de Lindt, J. (2010). Experimental Seismic Response of a Full-Scale Light-Frame Wood Building. *J. Struct. Eng.*, 135(3), 246-254.

Filiatrault, A., Fischer, D., Folz, B., & Uang, C.-M. (2002). Seismic Testing of a Two-Story Woodframe House: Influence of Wall Finish Materials. *J. Struct. Eng.*, 128(10), 1337-1345.

Fischer, D., Filiatrault, A., Folz, B., Uang, C., & Seible, F. (2001). *Shake Table Test of a Two-story House*. Richmond, CA: CUREE Publication No. W-06.

Folz, B., & Filiatrault, A. (2001). Cyclic analysis of wood shear walls. *J. Struc. Eng.*, *127*(4), 433-441.

Folz, B., & Filiatrault, A. (2002). *A Computer Program for Seismic Analysis of Woodframe Structures.* Richmond, CA: CUREE Report W-21.

Folz, B., & Filiatrault, A. (2004b). Seismic analysis of woodframe structures. II: Model implementation and verification. *J. Stuct. Eng*, 1361-1370.

Gatto, K., & Uang, C. M. (2002). *Cyclic Response of Woodframe Shear Walls: Loading Protocol and Rate of Loading Effects.* CUREe Publication No. W-16.

Gershfeld, M., Chadwell, C., van de Lindt, J., Pang, W., Ziaei, E. F., & Gordon, S. (2013). Distributed knee-brace (DKB) system as a complete or supplemental retrofit for soft-story low-rise wood-frame buildings. . *Structural Engineers Association of California Convention.* San Diego.

Gershfeld, M., Chadwell, C., van de Lindt, J., Pang, W., Ziaei, E., Amini, M., . . . Jennings, E. (2014). Distributed Knee-Braced (DKB) System as a Complete or Supplemental Retrofit of Soft-Story Wood-Frame Buildings. *Structures Congress*(1), 1437–1447.

Harris, S. K., & Egan, J. A. (1992). Effects of Ground Conditions on the Damage to Four-Story Corner partment Buildings. In *The Loma Prieta, California, Earthquake of October 17, 1989 Strong Ground Motion and Ground Failure - Marina District* (pp. 181-194). Washington, D.C.: United States Geological Survey .

Heine, C. (1997). *The Effect of Tie-Down Anchorage on Long Shear Walls with Openings.* Blacksburg, Virginia: Virginia Polytechnic Institute and State University.

ICBO. (1988). *Uniform Building Code, 1988 edition.* Whittier, CA: International Conference of Building Officials.

ICBO. (1994). *Uniform Building Code.* Whittier, CA.: International Conference of Building Officials.

ICC. (2012). *International Existing Building Code.* Washington, D.C.: International Code Council.

Itani, R. Y., & Cheung, C. K. (1984). Nonlinear analysis of sheathed wood diaphragms. *J. Struc. Eng.*, *110*(9), 2137-2147.

Jennings, E., & van de Lindt, J. (2012). Numerical Retrofit Study of Light-Frame Wood Buildings Using Shape Memory Alloy Devices as Seismic Response Modification Devices. *Journal of Structural Engineering*.

Jennings, E., Lindt, v. d., J.W., S., X., Pang, W., & Ziaei, E. (2014). Full-Scale Hybrid Testing of a Soft-Story Woodframe Building Seismically Retrofitted using Shape Memory Alloy Devices in Scissor-Jack. *10th National Conference in Earthquake Engineerig*. Anchorage, AK: Earthquake Engineering Research Institute .

Jennings, E., van de Lindt, J. W., Ziaei, E., Mochizuki, G., Pang, W., & Shao, X. (2014). Retrofit of a soft-story woodframe building using SMA devices with full-scale hybrid test verification. *Engineering Structures, 80*, 469-485.

Jennings, E., van de Lindt, J., Ziaei, E., Bahmani, P., Park, S., Shao, X., . . . Gershfeld, M. (2015). Full-Scale Experimental Verification of Soft-Story-Only Retrofits ofWood-Frame Buildings using Hybrid Testing. *Journal of Earthquake Engineering*, 19:410–430.

Karacabeyli, E., & Douglas, B. (2013). *CLT Handbook: Cross-laminated Timber*. Vancouver, BC, Canada: FPInnovations.

Kasal, B., & Leichti, R. J. (1992). Nonlinear finite-element model for light-frame stud walls. *J. Struct.*, *118*(11), 3122-3135.

Kasal, B., & Leichti, R. J. (1992). Nonlinear finite-element model for light-frame stud walls. *J. Struct. Eng.*, 3122-3135.

Kharrazi, M. (2001). *Vibration Characteresitics of Single - Family Woodframe Buildings.* Vancouver, Canada: MASc Thesis, University of British Columbia .

Kharrazi, M., Ventura, C., Prion, G., & Lord, J. T. (2002). Experimental Evaluation of Seismic Response of Woodframe Residential Construction . *4th Structural Specialty Conference of the CSCE.* Montreal, Canada.

Kinoshita, S. (1998). Kyoshin net (K-net). *Seismological Research Letters, 69*(4), 309-332.

Krawinkler, H., Parisi, F., Ibarra, L., Ayoub, A., & Medina, R. (2003). *Development of a Testing Protocol for Woodframe Structures.* Richmond, CA: Consortium of Universities for Research in Earthquake Engineering.

Li, Y., & Ellingwood, B. R. (2007). Reliability of woodframe residential construction subjected to earthquakes. *Structural Safety, 29*(4), 296-307.

Mosalam, K. M., Mahin, S., & Naito, C. (2002). *Seismic evaluation of asymmetric three-story woodframe building, CUREE Publication No.: W-19.* Richmond, CA: CUREE.

Pacific Fire Rating Bureau. (1971). *San Fernando Earthquake.*

Pang, W. C., & Rosowsky, D. V. (2010). Beam-spring model for timber diaphragm and shear walls. *Proceedings of the Institution of Civil Engineers-Structures and Buildings*, 227-244.

Pang, W. C., Rosowsky, D. V., Pei, S., & van de Lindt, J. W. (2007). Evolutionary Parameter Hysteretic Model for Wood Shear Walls. *Journal of Structural Engineering*, *133*, 1118-1129.

Pang, W., & Hassanzadeh, M. (2010). Next generation numerical model for non-linear in-plane analysis of wood-frame shear walls. *World Conference on Timber Engineering.* Trento Province, Italy.

Pang, W., & Shirazi, S. M. (2013). Corotational model for cyclic analysis of light-frame wood shear walls and diaphragms. *ASCE Journal of Structural Engineering* , 139 (8): 1303-1317.

Pang, W., Hassanzadeh, S., & Seyed, M. (2012). Corotational Model for the Cyclic Analysis of Light Frame Wood Shear Walls and Diaphragms. *J. Struct. Eng.*, *139*, 450.

Pang, W., Rosowsky, D. V., Ellingwood, B. R., & Wang, Y. (2009). Seismic Fragility Analysis and Retrofit of Conventional Residential Wood-Frame Structures in the Central United States. *Journal of Structural Engineering*, *135*(3), 262–267.

Pang, W., Rosowsky, D. V., Ellingwood, B. R., & Wang, Y. (2009). Seismic Fragility Analysis and Retrofit of Conventional Residential Wood-Frame Structures in the Central United States. *Journal of Structural Engineering, 135*(3), 262-271.

Pang, W., Rosowsky, V., D., Ellingwood, B. R., & Wang, Y. (2009). Seismic Fragility Analysis and Retrofit of Conventional Residential Wood-Frame Structures in the Central United States. *Journal of Structural Engineering, 135*(3), 262-271.

Pang, W., Ziaei, E., & Filiatrault, A. (2012). A 3D model for collapse analysis of Soft-story Light-frame wood building. *World Conference on Timber Engineering.* Auckland, New Zealand.

Pang, W., Ziaei, E., Shao, X., Jennings, E., van de Lindt, J., Gershfeld, M., & Symans, M. (2014). A Three-Dimension Model for Slow Hybrid Testing of Retrofits for Soft-Story Wood-Frame Buildings. *10th U.S. National Conference on Earthquake Engineering.* Anchorage, Alaska.

Pardoen, G. C., Walman, A., Kazanjy, R. P., Freund, E., & Hamilton, C. H. (2003). *Testing and Analysis of One-Story and Two-Story Shear Walls Under Cyclic Loading.* CUREe Publication No. W-25.

PEER, CALTRANS, CGS. (2010). *Technical Report for the PEER Ground Motion Database Web Application.*

Pei, S. (2007). *Loss Analysis and Loss Based Seismic Design for Woodframe Structures.* Colorado State University.

Pei, S., & van de Lindt, J. (2010). *User's Manual For SAPWood for Windows.* Colorado State University .

Pei, S., & van de Lindt, J. (2012). Coupled shear-bending formulation for seismic analysis of stacked shear wall systems. *Earthquake Engineering & Structural Dynamics*, 1549-1568.

Pei, S., & van de Lindt, J. W. (2011). Seismic Numerical Modeling of a Six-Story Light-Frame Wood Building: Comparison with Experiments. *Journal of Earthquake Engineering*, 924-941.

Pei, S., van de Lindt, J. W., Pryor, S. E., Shimizu, H., & Isoda, H. (2010). *Seismic testing of a full-scale six-story light-frame wood building: NEESWood Capstone test.* NEESWood Report NW-06.

Pei, S., van de Lindt, J. W., Wehbe, N., & Liu, H. (2013). Experimental Study of Collapse Limits for Wood Frame Shear Walls. *Journal of Structural Engineering*, 1489-1497.

Pei, S., van de Lindt, J., & Popovski, M. (2013). Approximate R-Factor for Cross-Laminated Timber Walls in Multistory Buildings. *J. Archit. Eng*, *19*(4), 245–255.

Pender, M., & Robertson, T. (1987). *Edgecumbe Earthquake: Reconnaissance Report.* Bull. New Zealand Society of Earthquake Engineering.

Popovski, M., Schneider, J., & Schweinsteiger, M. (2010). Lateral load resistance of cross laminated wood panels . *Proc. of the World Conference on Timber Engineering.* Riva del Garda, Italy.

Priestley, M. (1998). Displacement-based approaches to rational limit states design of new structures. *11th European Conference on Earthquake Engineering.* Paris, France.

Pryor, S. E., & Murray, T. M. (2013). Next generation partial strength steel moment frames for seismic resistance. *Research, Development, and Practice in Structural Engineering and Construction*, 27–32.

Rainer, J., & Karacabeyli, E. (2000). Wood-frame construction in past earthquakes. *12th World Conference on Earthquake Engineering.* Auckland, New Zealand.

Rose, J., & Keith, E. (1997). *Wood Structural Panel Shear Walls With Gypsum Wallboard and Window/Door Openings.* American Plywood Association.

Scawthorn, C., Kornfield, L., Seligson, H., & Rojahn, C. (2006). Estimated Losses from Scenario Earthquakes Affecting San Francisco: CAPSS – Part 2. . *8th US National Conference on Earthquake Engineering.* San Francisco.

Scawthorn, C., Kornfield, L., Seligson, H., & Rojahn, C. (April 2006). Estimated Losses from Scenario Earthquakes Affecting San Francisco: CAPSS – Part 2. *8th US National Conference on Earthquake Engineering.* San Francisco.

Shinde, J., & Symans, M. (2010). *Integration of Seismic Protection Systems in Performance-Based Seismic Design of Woodframed Structures.* NEESWood Project Report .

Sofali, V. (2008). *Evaluation of direct stucco-woodframe connectors in improved stucco shear walls* . Vancouver, B.C.: University of British Columbia .

Symans, M., Cofer, W., Du, Y., & Fridley, K. (2004). Seismic Behavior of Wood-framed Structures with Viscous Fluid Dampers. *Earthquake Spectra*, 20 (2): 451-482.

Symans, M., W.F., C., & Fridley, K. (2002). Base Isolation and Supplemental Damping Systems for Seismic Protection of Wood Structures: Literature Review. *Earthquake Spectra,* 18 (3): 549-572.

Tarabia, A. M., & Itani, R. Y. (1997). Seismic response of light-frame wood buildings. *J. Struct. Eng.*, 123:11(1470), 1470–1477.

Taylor, G. W., Prion, H. G., Ventura, C. E., & Kharrazi, M. (2003). *Static and dynamic earthquake testing of rainscreen stucco systems for Bristish Columbia residential wood frame construction* . Vancouver, BC: TBG Seismic Consultants Ltd.

TBG Seismic Constultants Ltd. (2002). *Earthquake 99 Project - Laboratory Tests Results.* Sidney, Canada: EERF UBC.

Tian, J. (2014). *Performance-Based Seismic Retrofit of Soft-Story Woodframe Buildings Using Energy-Dissipation Systems.* Rensselaer Polytechnic Institute.

Tian, J., Symans, M., Gershfeld, M., Bahmani, P., & van de Lindt, J. (2014). Seismic Performance of a Full-Scale Soft-Story Woodframed Building with Energy Dissipation Retrofit. *10th National Conference in Earthquake Engineering.* Anchorage, AK: Earthquake Engineering Research Institute.

Toothman, A. J. (2003). *Monotonic and Cyclic Performance of Light-Frame Shear Walls With Various Sheathing Materials* . Virginia Polytechnic Institute .

Vamvatsikos, D., & Cornell, C. A. (2002). Incremental dynamic analysis. *Earthquake Engineering & Structural Dynamics*, *31*(3), 491-514.

van de Lindt, J. W., Bahmani, P., Mochizuki, G., & Pryor, S. (2014). Experimental Seismic Behavior of a Full-Scale Four-Story Soft-Story Woodframe Building with Retrofits II: Shake Table Test Results. *J. of Struct. Eng.*

van de Lindt, J. W., Pei, S. P., C., W., & Hassansadeh, M. (2012b). Collapse testing and analysis of a light-frame wood garage wall. *J. Struct. Eng.*, 492–501.

van de Lindt, J. W., Pei, S., Pryor, S. E., Shimizu, H., & Isoda, H. (2010). Experimental seismic response of a full-scale six-story light-frame wood building. *Journal of Structural Engineering*, 1262-1272.

van de Lindt, J., & Potts, A. (2008). Shake Table Testing of a Superelastic Shape Memory Alloy Response Modification Device in a Wood Shearwall. *Journal of Structural Engineering*.

van de Lindt, J., Bahmani, P., Gershfeld, M., Kandukuri, G., Rammer, D., & Pei, S. (2013). Seismic Retrofit of Soft-Story Woodframe Buildings Using Cross Laminated Timber. *Proc. of ISEC-7.* Honolulu, HI.

van de Lindt, J., Bahmani, P., Pryor, S. E., Mochizuki, G., Gershfeld, M., Pang, W., . . . Rammer, D. (2014). Overview of the NEES-Soft Experimental Program for Seismic Risk Reduction of Soft-Story Woodframe Buildings. *Structures Congress*, 2875–85.

van de Lindt, J., Pei, S., Liu, H., & Filiatrault, A. (2010). Three-Dimensional Seismic Response of a Full-Scale Light-Frame Wood Building: Numerical Study. *J. Struct. Eng.,* , 136(1), 56–65.

van de Lindt, J., Symans, M., Pang, W., Shao, X., & Gershfeld, M. (2012). Seismic Risk Reduction for Soft-story Woodframe Building: The NEES-Soft Project. *121th World Conference on Timber Engineering.* Auckland, New Zealand.

Ventura, C. E., Bebamzedeh, A., Fairhurst, M., Taylor, G., & Fiam, W. D. (2015). Updates to the British Columbia Seismic Retrofit Guidelines, 3rd Edition. *11th Canadian Conference of Earthquake Engineering* . Victoria, Canada.

Ventura, C., Finn, W. .., Onur, T., Blanquera, A., & Rezai, M. (2005). Regional seismic risk in British Columbia — classification of buildings and development of damage probability functions. . *Canadian Journal of Civil Engineering*, 32(2):372–387.

Ventura, C., Taylor, G., Prion, H., & Kharrazi, M. (2002). Full-Scale Shake Table Studies of Woodframe Residential Construction. *7th US National Conferene on Earthquake Engineering* . Boston, USA.

Xu, J. a. (2009). Developement of a wood frame shear wall model in ABAQUS. *J. Struct. Eng.*, *135*(8), 977-984.

Ziaei, E., Shao, X., Jennings, E., van de Lindt, J., Gershfeld, M., & Symans, M. (2014). A Three-Dimension Model for Slow Hybrid Testing of Retrofits for Soft-Story Wood-Frame Buildings. *10th U.S. National Conference on Earthquake Engineering.*

Appendix A Analytical Programs

Three analytical programs were used in the studies: (1) SAPWood, (2) Timber3D and (3) M-CASHEW2.

SAPWood

SAPWood (Seismic Analysis Package for Woodframe structures) was developed as part of the NEESWood project. It is a toolbox to model light-frame wood structures. Four types of models are available in SAPWood: (1) a bi-axial structural model (by Folz and Filiatrault (2002) in the SAWS program) where there are 3DOF are defined in each storey and the diaphragm is assumed to be completely rigid; (2) a tri-axial model with six DOF in three-dimensional space; non-linear pure shear springs for shear walls and cumulative uplift of the hold-down rods and coupled interaction between lateral displacements and horizontal diaphragm rotation are incorporated in the tri-axial model (Pei and van de Lindt, 2009, van de Lindt et al. 2010); (3) a simplified 1 DOF lumped-mass shear wall model for uni-directional analysis and simplified design approaches; and (4) the SAPWood-Nail Pattern (NP) analysis model that allows for the ability to model wood shear walls down to the fastener level (similar to the CASHEW program).

The user manual and program is available online:

https://nees.org/resources/819/download/SAPWood_Users_Manual_V20.pdf

https://nees.org/resources/sapwood/supportingdocs

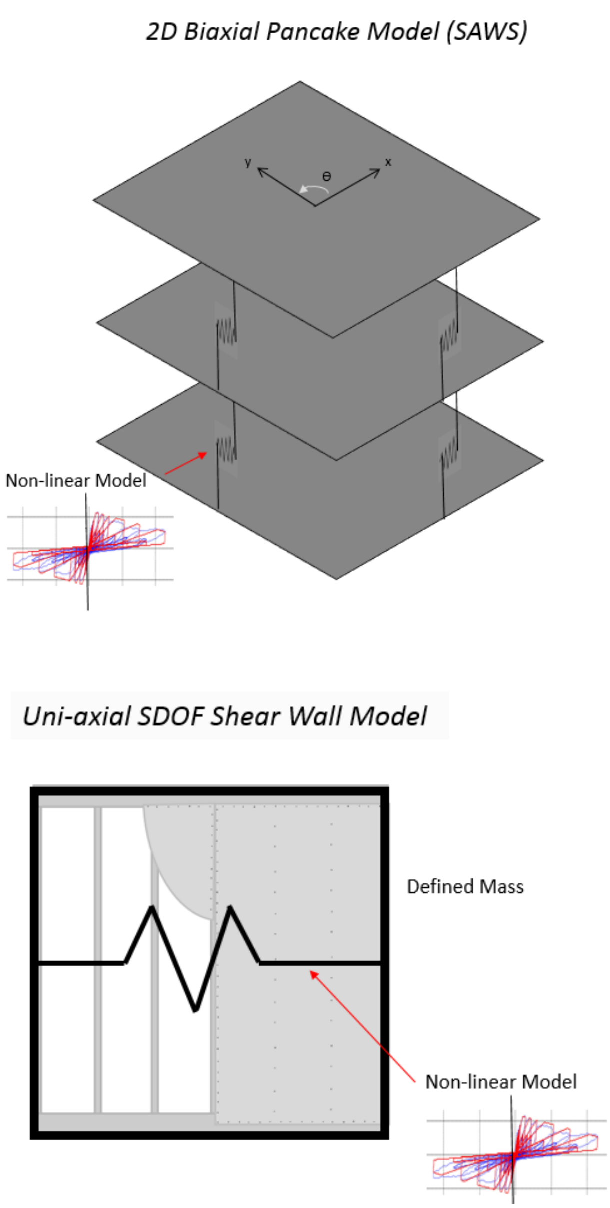
Useful references include:

- **Loss Analysis and Loss Based Seismic Design for Woodframe Structures** by: S. Pei
- **Seismic Numerical Modeling of a Six-Story Light-frame Wood Building: Comparison with Experiments** by: S. Pei; J. W. van de Lindt

- **Coupled Shear Bending Formulation for Seismic Analysis of Stacked Wood Shear Wall Systems** by: S. Pei; J. W. van de Lindt

- **Three-Dimensional Seismic Response of a Full-Scale Light-Frame Wood Building: Numerical Study** by: J. W. van de Lindt; S. Pei; H. Liu; A. Filiatrault

A schematic of the four type of model is shown in the figure below:



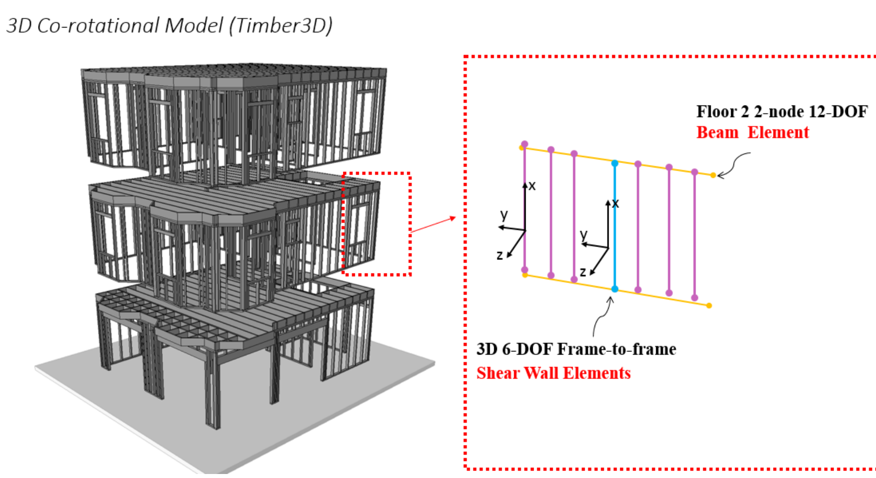
Timber3D

Timber3D is a Matlab and Simulink program for three-dimensional light-frame wood dynamic analysis. The model was developed using a co- rotational formulation and large displacement theory. The in-plane and out-of-plane motions of the diaphragms under large deformations is considered. The diaphragms are modeled with 3D two-node 12-DOF *frame elements* and can be used to model tension, compression, torsion and bending behavior mechanisms. The lateral stiffness of the walls is modelled with 3D, two-node, 6-DOF *link elements*. A nodal condensation technique is applied to condense the DOFs of the link elements and reduce the computational time. This model is appropriate for modeling full global collapse as it is based on large displacement theory.

Useful references include:

- **A Three-Dimension Model for Slow Hybrid Testing of Retrofits for Soft-Story Wood-Frame Buildings** by: W. Pang; E. Ziaei; X. Shao; E. Jennings; J. van de Lindt; M. Gershfeld; and M. Symans (2014)
- **A 3D Model of Collapse Analysis of Soft-story Light-frame Wood Buildings** by: W. Pang; E Ziaei; and A. Filiatrault

A schematic of the model is shown in the figure below:



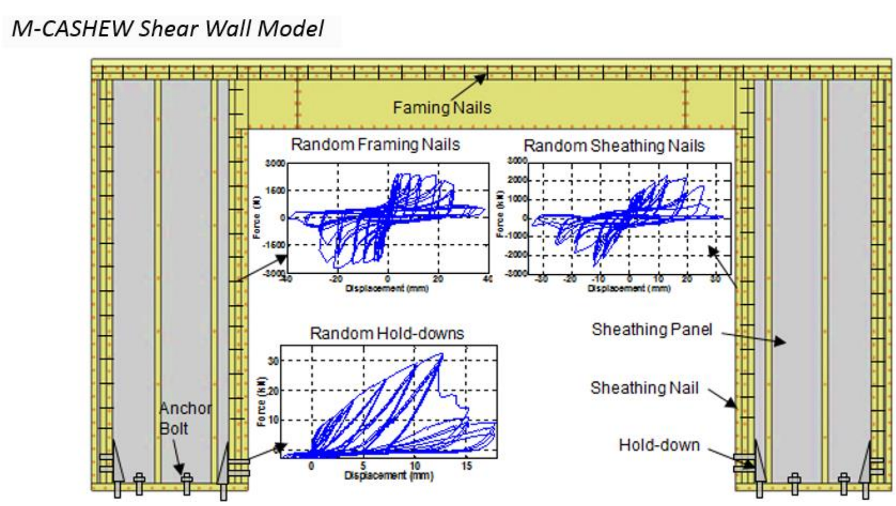
M-CASHEW2

M-CASHEW2 (MATLAB - Cyclic Analysis of Wood Shear Wall version 2) is a numerical modeling program used for detailed modeling of light-frame wood walls and diaphragms. Three main components are used in the model: (1) framing members (two-node 6-DOF planar-frame beam elements); (2) sheathing panels (5-DOF shear-panel elements), and (3) connectors/bearing contact elements such as nails, bolts and hold-downs (3-DOF link elements). The program is flexible for modeling for different sheathing (i.e. horizontal boards, OSB, GWB), opening configurations, nailing patterns, anchorage and vertical loading conditions of wood shear wall and diaphragm assemblies.

Useful references include:

- **Next Generation Numerical Model for Non-linear in-plane Analysis of Wood-frame Shear Walls** by: W. Pang; and S. M. H.
- **Collapse Testing and Analysis of a Light-frame Wood Garage Wall** by: J. van de Lindt; P. Shiling; W. Pang; S. M. H. Shirazi
- **Corotational Model for Cyclic Analysis of Light-frame Wood Shear Walls and Diaphragms** by: J. van de Lindt; P. Shiling; W. Pang; S. M. H. Shirazi

A schematic of the model is shown in the figure below:



Appendix B EQ-99 Woodframe House Drawings

The following are drawings of the Earthquake 99 Woodframe House project provided by TBG Seismic Consultant Ltd. These drawings include elevation and plan views of the subsystem and two-storey house test specimens.

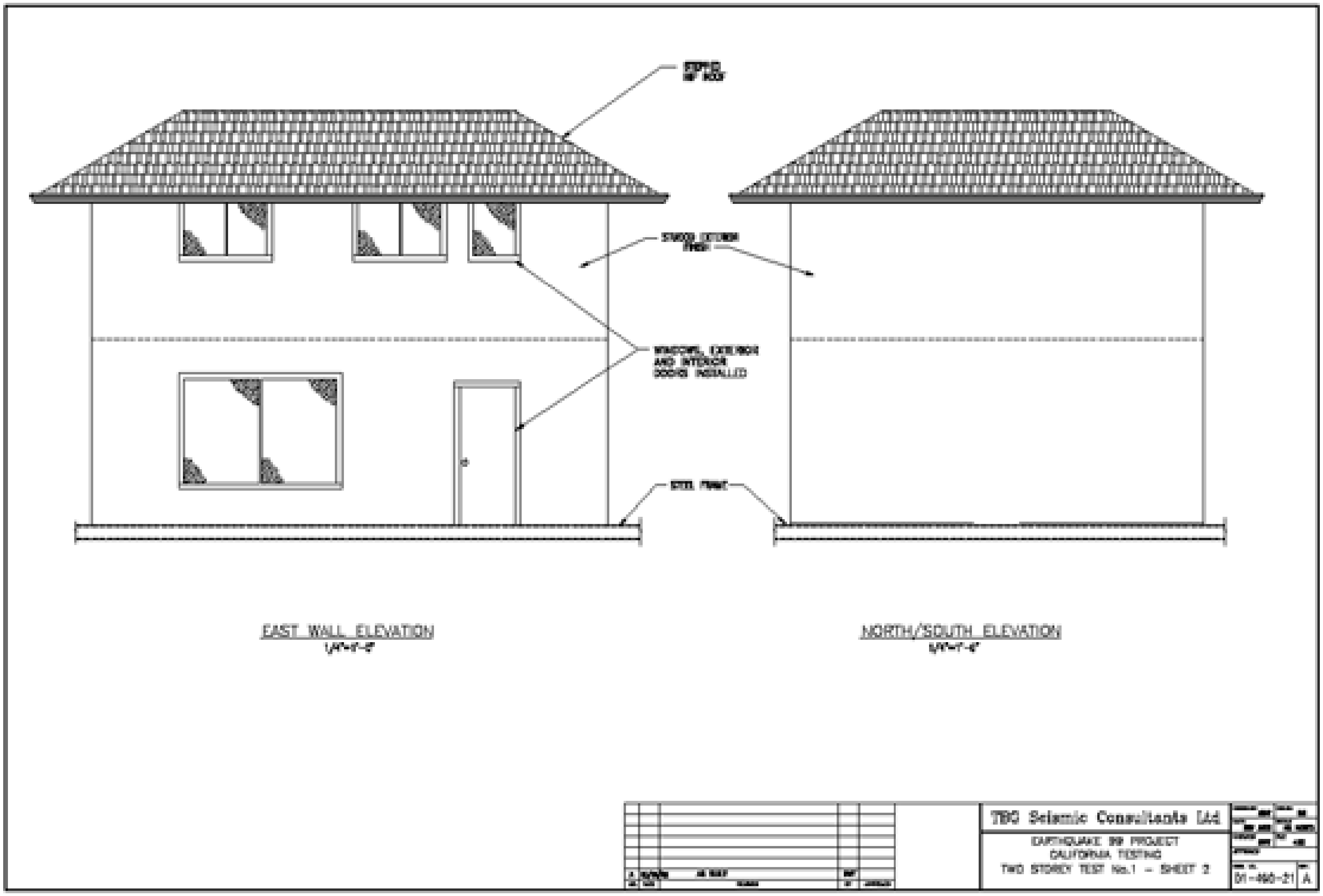


Fig. B.1: Two-storey woodframe house elevations view

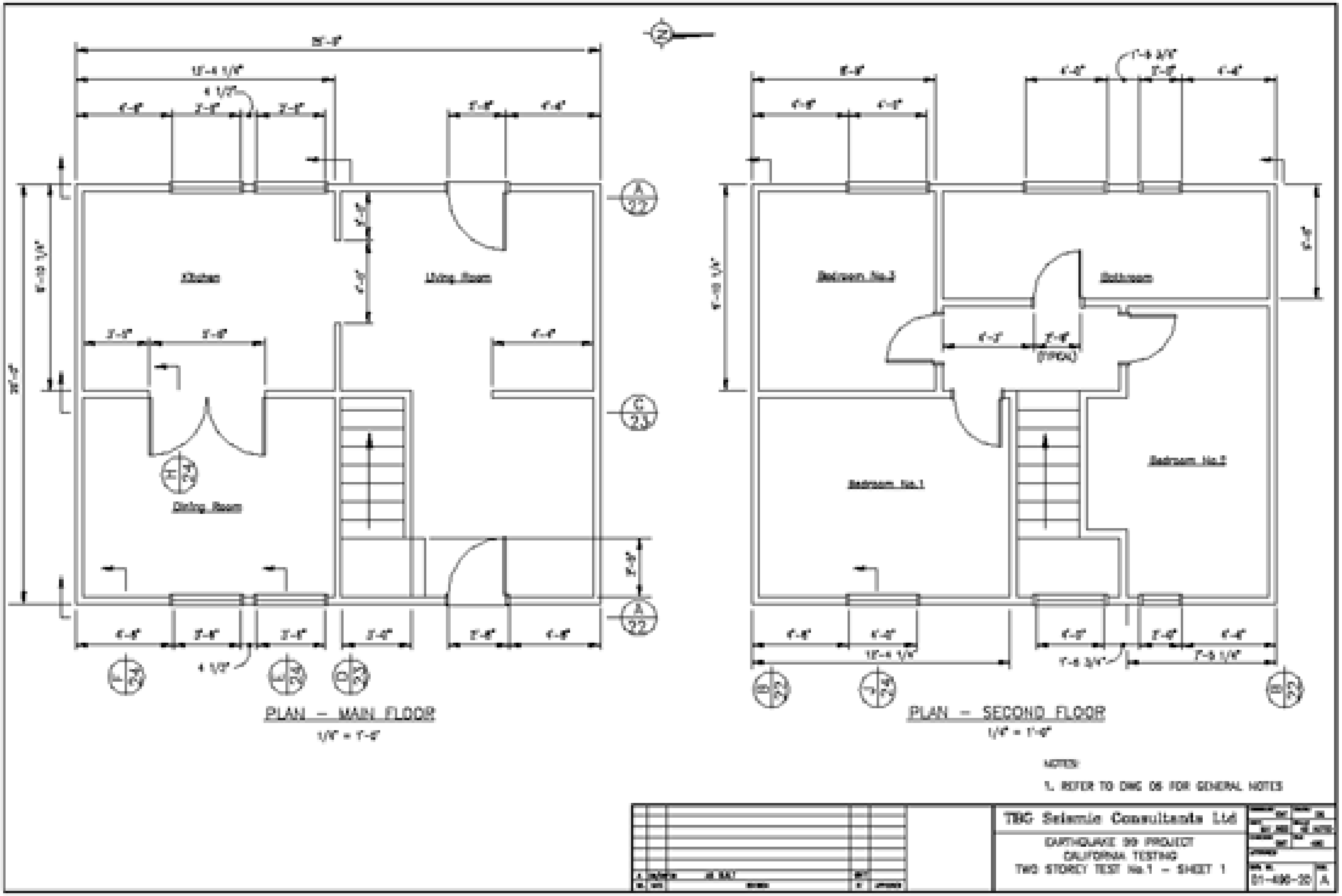


Fig. B.2: Two-storey woodframe house plan view

Appendix C Summary of EQ-99 Shake Table Tests

The following summaries of the Earthquake 99 shake table test documents for the 2-Storey woodframe house project provided M. Kharrazi (2002). The first run for Test 9 – Test 15 is included.

Department of Civil Engineering
The University of British Columbia

Summary of Earthquake 99 Project
2-Storey House Shake Table Test

Data Analysis Project Information:

Title = "Test 09 - Sherman Oaks Record"

Description = "Two Storey OSB panel wall system w/o stucco"

Date_of_Test = "28 July 2000"

Test_No = 9

Run_sequence = 1

Directed and Supervised by C.E.Ventura

Analysis done by M.K.H.Kharrazi / C.E.Ventura

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Title = "Test 09 - Sherman Oaks Record"

Description = "Two Storey OSB panel wall system w/o stucco"

Date_of_Test = "28 July 2000"

Section I

Shake Table Motions

This section presents the recorded shake table motions (base level motion) during this test.

The information presented here includes:

a) shake table actuator displacement (in cm)
b) shake table inferred velocity (in cm/sec)
c) shake table actuator acceleration (in g's)
d) shake table actuator load (in kN)

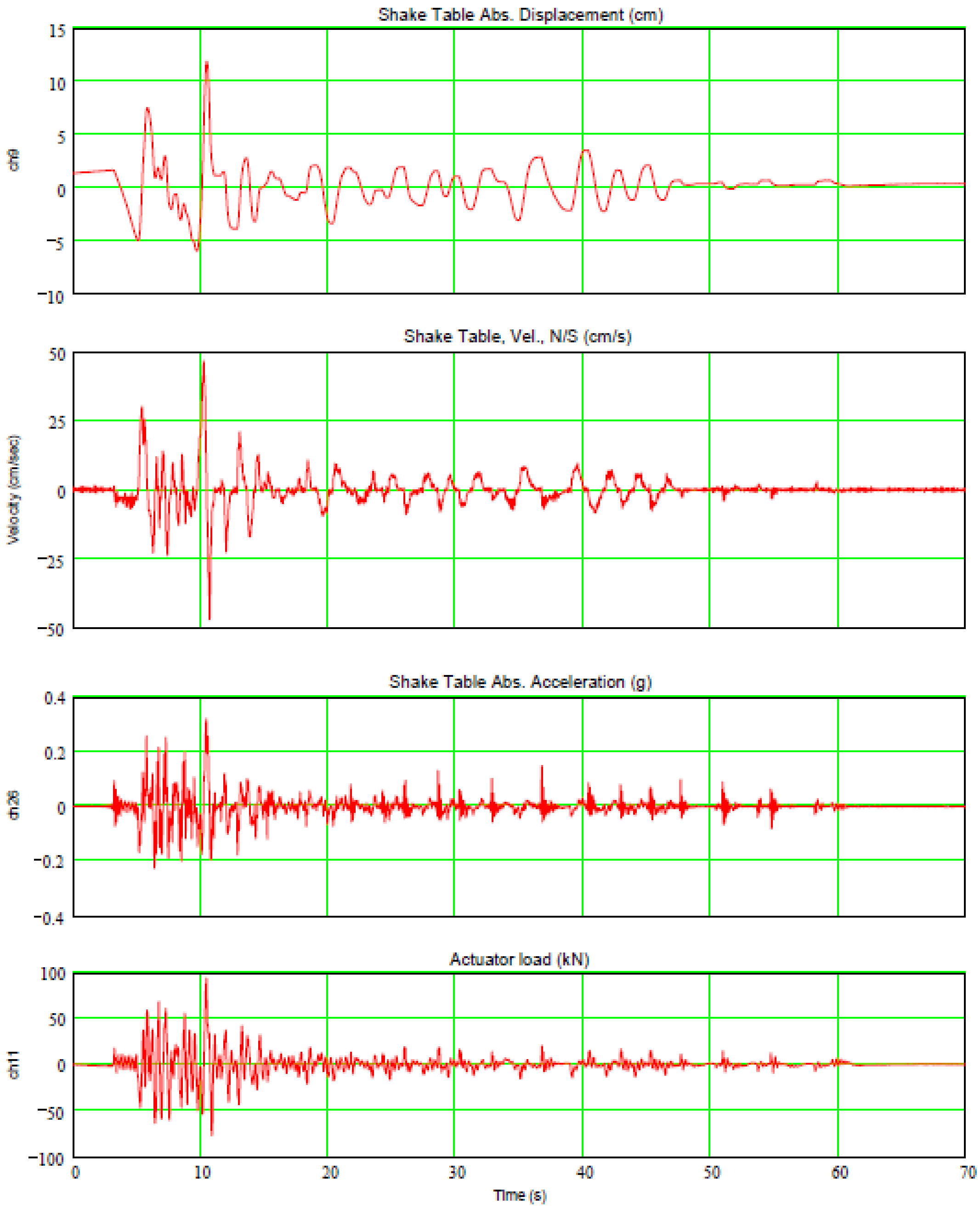
The recorded peak values (after signal conditioning) of these motions are:

a) Displacement:	Mdisp = 11.955	cm	=	Mdispl = 4.707	in
b) Velocity:	MVel = 46.932	cm/sec	=	MVell = 18.477	in/sec
c) Acceleration:	Maccl = 0.321	g			
d) Load:	MLoad = 94.727	kN	=	MLoadl = 21.296	kips

1. Shake Table Motions

Title = "Test 09 - Sherman Oaks Record" Date_of_Test = "28 July 2000"

Description = "Two Storey OSB panel wall system w/o stucco"



Title = "Test 09 - Sherman Oaks Record"

Description = "Two Storey OSB panel wall system w/o stucco"

Date_of_Test = "28 July 2000"

Section II

Horizontal Absolute Accelerations

The peak values of absolute acceleration are:

a. Roof Level (N/S direction)

East Wall	MARLE = 0.509	g
Center Wall	MARLC = 0.502	g
West Wall	MARLW = 0.428	g

b. 2nd Floor (N/S Direction)

East Wall	MA2LE = 0.403	g
Center Wall	MA2LC = 0.398	g
West Wall	MA2LW = 0.388	g

c. Base Level (N/S Direction)

East Wall	MABLE = 0.312	g
Center Wall	MABLC = 0.321	g
West Wall	MABLW = 0.312	g

Title = "Test 09 - Sherman Oaks Record"

Description = "Two Storey OSB panel wall system w/o stucco"

Date_of_Test = "28 July 2000"

Section III

Anchor Rod Loads

The peak values of the measured loads are as follow:

a. 2nd floor rods

NE North Rod	MNENR2 = 0	kN	or	MNENR2I = 0	kips
NE South Rod	MNESR2 = 0	kN	or	MNESR2I = 0	kips
SE North Rod	MSENR2 = 0	kN	or	MSENR2I = 0	kips
SE South Rod	MSESR2 = 0	kN	or	MSESR2I = 0	kips

b. Base level rods

NE North Rod	MNENRB = 10.871	kN	or	MNENRBI = 2.444	kips
NE South Rod	MNESRB = 12.49	kN	or	MNESRBI = 2.808	kips
SE North Rod	MSENRB = 10.959	kN	or	MSENRBI = 2.464	kips
SE South Rod	MSESRB = 13.291	kN	or	MSESRBI = 2.988	kips

Note: Any prestressed load in the anchors before the earthquake simulation test was removed by zeroing the beginning of each record. So the loads presented in the following plots should be interpreted as just the dynamic component of the rod loads, and not the total rod loads.

Title = "Test 09 - Sherman Oaks Record"

Description = "Two Storey OSB panel wall system w/o stucco"

Date_of_Test = "28 July 2000"

Section IV

Relative Displacements and Drifts

The following relative displacements with respect to the Base Level are considered here:

a) displacement of the 2nd floor, and
b) displacement of the Roof Level.

The drifts considered here are the *drift of the 2nd Floor* with respect to the Base Level and the *drift of the Roof Level* with respect to the 2nd Floor.

The peak values are:

a. Relative Displacements

a.i) Roof vs. Base Level

East Wall	MRBDE = 1.875	cm	or	MRBDEI = 0.738	in
Center Wall	MRBDC = 1.981	cm	or	MRBDCI = 0.78	in
West Wall	MRBDW = 1.929	cm	or	MRBDWI = 0.759	in

a.ii) 2nd Floor vs. Base Level

East Wall	M2BDE = 1.071	cm	or	M2BDEI = 0.421	in
Center Wall	M2BDC = 1.407	cm	or	M2BDCI = 0.554	in
West Wall	M2BDW = 1.177	cm	or	M2BDWI = 0.463	in

b. Drifts

b.i) Roof vs. 2nd Floor

East Wall	MRBDDE = 0.898	cm	or	MRBDDEI = 0.354	in
Center Wall	MRBDDC = 0.715	cm	or	MRBDDCI = 0.282	in
West Wall	MRBDDW = 0.846	cm	or	MRBDDWI = 0.333	in

b.ii) 2nd Floor vs. Base Level

East Wall	M2BDDE = 1.071	cm	or	M2BDDEI = 0.421	in
Center Wall	M2BDDC = 1.407	cm	or	M2BDDCI = 0.554	in
West Wall	M2BDDW = 1.177	cm	or	M2BDDWI = 0.463	in

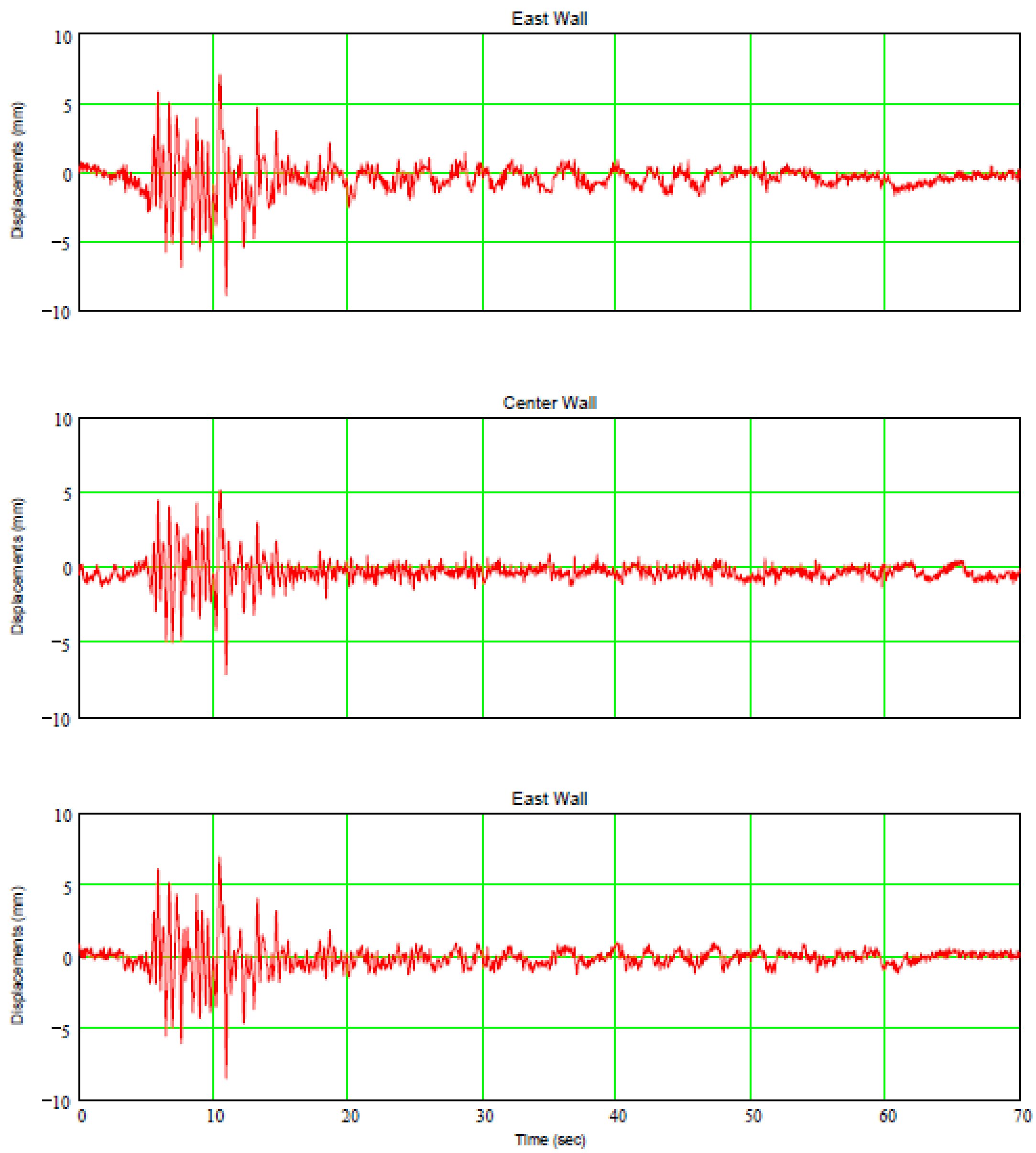
The Time histories of relative displacements and drift are presented in the following pages.

4. Relative Displacements & Drifts.

Title = "Test 09 - Sherman Oaks Record"

Description = "Two Storey OSB panel wall system w/o stucco"

Drift - Roof Level vs. 2nd Floor

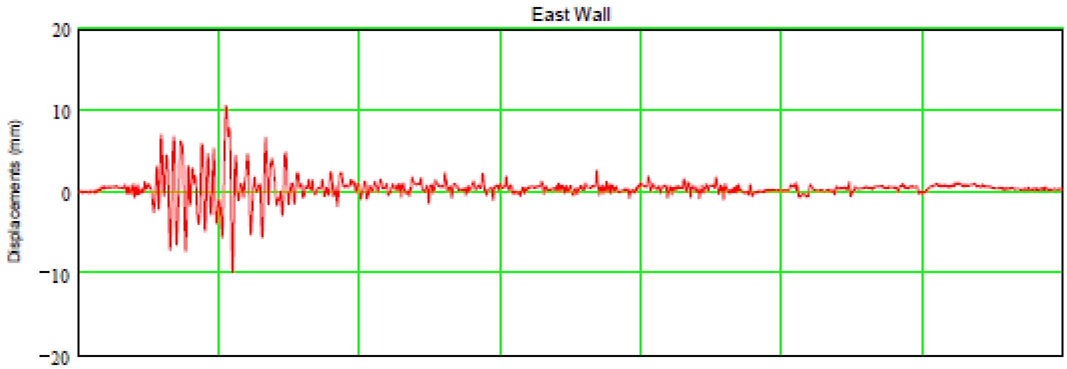


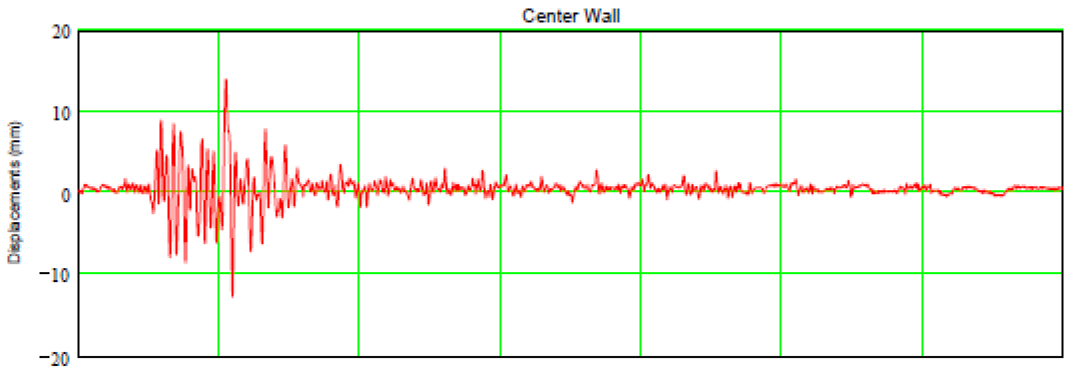
4. Relative Displacements & Drifts.

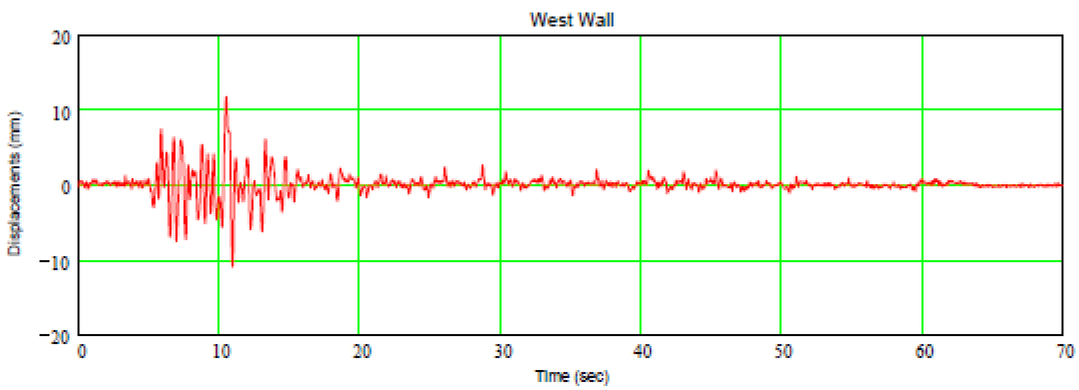
Title - "Test 09 - Sherman Oaks Record"

Description - "Two Storey OSB panel wall system w/o stucco"

Drift - 2nd Floor vs. Base Level







Title = "Test 09 - Sherman Oaks Record"

Description = "Two Storey OSB panel wall system w/o stucco"

Date_of_Test = "28 July 2000"

Section V

Load - Deformation Diagrams

The load used for the following diagrams was calculated as the base shear resulting from adding the story shears from each floor. Each story shear was computed as the product of the measured absolute acceleration and the calculated floor mass. The floor weight was calculated to be 9.0 and 8.8 metric tons for the roof and second floor respectively. Since the measured actuator load include the inertia load of the shake table and part of the weight of the first floor it was concluded that a better representation of the base shear could be obtained from the computed story shears rather than by a removing these inertia loads from the measured actuator loads.

Maximum Loads

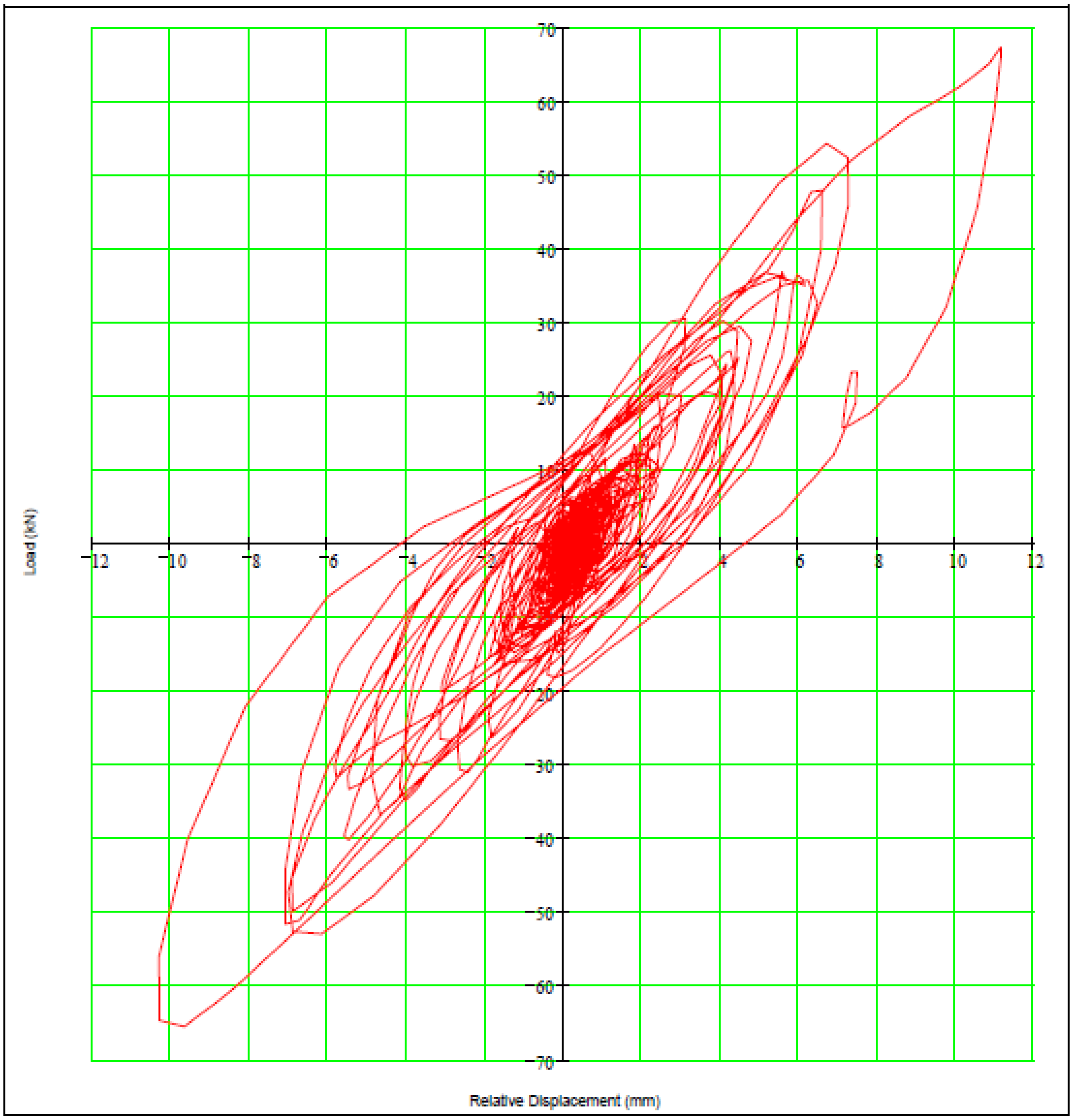
Maximum load for the First Storey (kN): $\max\left(\overrightarrow{|CLDF|}\right) = 67.413$

5. Load-Deformation Diagrams.

Title = "Test 09 - Sherman Oaks Record"

Description = "Two Storey OSB panel wall system w/o stucco"

Load-Deformation, First Storey



Title = "Test 09 - Sherman Oaks Record"

Description = "Two Storey OSB panel wall system w/o stucco"

Date_of_Test = "28 July 2000"

Section VI

Calculated Velocity

The velocity in each floor is calculated from the the differential of the displacement. A subroutine to differentiate displacement signals by finite differences was written for this purpose. The resulting the peak velocities for each floor are shown below.

	West		Center		East	
Roof Level	MV3 = 57.578	cm/s	MV2 = 57.269	cm/s	MV1 = 57.425	cm/s
2nd Floor	MV6 = 51.242	cm/s	MV5 = 52.633	cm/s	MV4 = 54.013	cm/s
Basement Level	MV9 = 47.83	cm/s	MV8 = 46.932	cm/s	MV7 = 45.937	cm/s

or

Roof Level	MIV3 = 22.669	in/s	MIV2 = 22.547	in/s	MIV1 = 22.608	in/s
2nd Floor	MIV6 = 20.174	in/s	MIV5 = 20.722	in/s	MIV4 = 21.265	in/s
Basement Level	MIV9 = 18.831	in/s	MIV8 = 18.477	in/s	MIV7 = 18.085	in/s

Reference:C:\Documents and Settings\Mehdi H. K. Kharrazi\Desktop\Earthquake 99 Final Edition\EQ 99 - TWO STOREY (Part I)\Test

Department of Civil Engineering
The University of British Columbia

Summary of Earthquake 99 Project
2-Storey House Shake Table Test

Data Analysis Project Information:

Title = "Test 10 - Nahanni"

Description = "Two Storey OSB panel wall system with stucco"

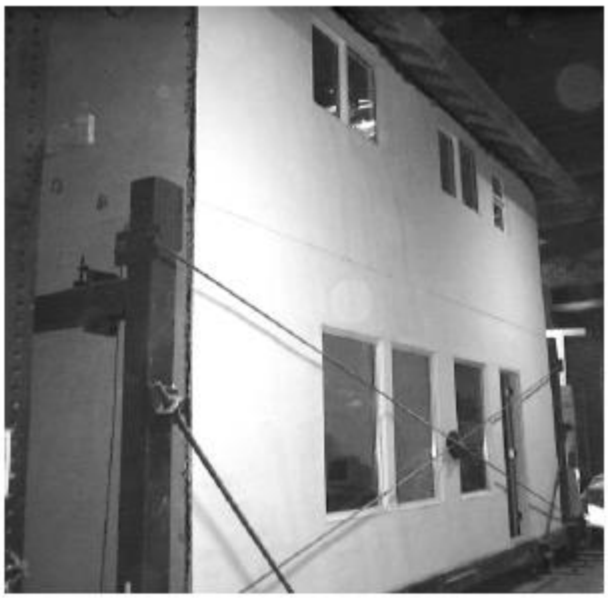
Date_of_Test = "13 June 2001"

Test_No = 10

Run_sequence = 1

Directed and Supervised by C.E.Ventura

Analysis done by M.H.K.Kharrazi / C.E.Ventura



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Section **Page**

Title = "Test 10 - Nahanni"

Description = "Two Storey OSB panel wall system with stucco"

Date_of_Test = "13 June 2001"

Section I

Shake Table Motions

This section presents the recorded shake table motions (base level motion) during this test.

The information presented here includes:

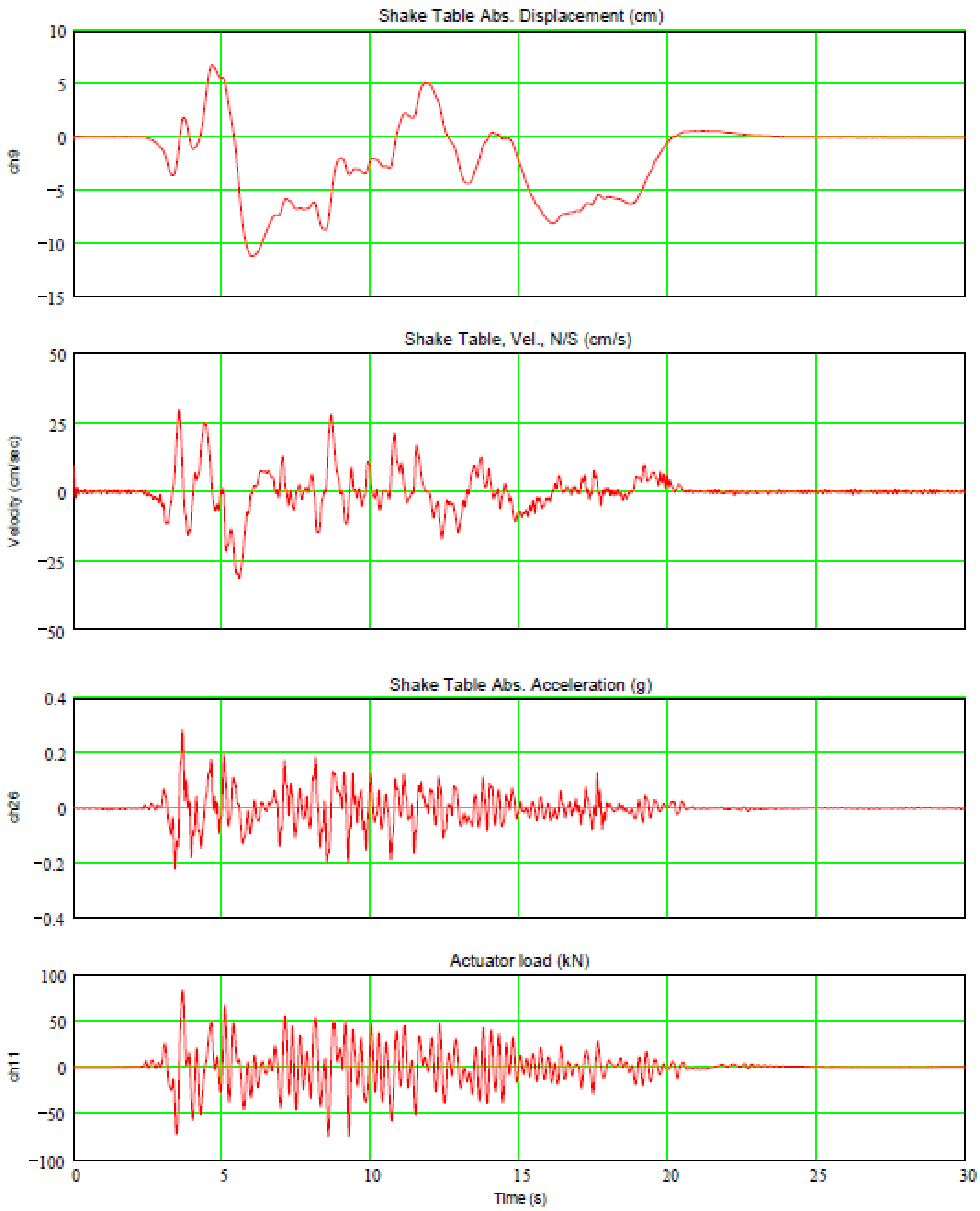
a) shake table actuator displacement (in cm)
b) shake table inferred velocity (in cm/sec)
c) shake table actuator acceleration (in g's)
d) shake table actuator load (in kN)

The recorded peak values (after signal conditioning) of these motions are:

a) Displacement:	Mdisp = 11.241	cm	=	Mdispl = 4.426	in
b) Velocity:	MVel = 31.329	cm/sec	=	MVell = 12.334	in/sec
c) Acceleration:	Maccl = 0.286	g			
d) Load:	MLoad = 84.128	kN	=	MLoadl = 18.914	kips

1. Shake Table Motions Title = "Test 10 - Nahanni" Date_of_Test = "13 June 2001"

Description = "Two Storey OSB panel wall system with stucco"



Title = "Test 10 - Nahanni"

Description = "Two Storey OSB panel wall system with stucco"

Date_of_Test = "13 June 2001"

Section II

Horizontal Absolute Accelerations

The peak values of absolute acceleration are:

a. Roof Level (N/S direction)

East Wall	MARLE = 0.355	g
Center Wall	MARLC = 0.409	g
West Wall	MARLW = 0.347	g

b. 2nd Floor (N/S Direction)

East Wall	MA2LE = 0.317	g
Center Wall	MA2LC = 0.317	g
West Wall	MA2LW = 0.311	g

c. Base Level (N/S Direction)

East Wall	MABLE = 0.293	g
Center Wall	MABLC = 0.286	g
West Wall	MABLW = 0.275	g

Title = "Test 10 - Nahanni"

Description = "Two Storey OSB panel wall system with stucco"

Date_of_Test = "13 June 2001"

Section III

Anchor Rod Loads

The peak values of the measured loads are as follow:

a. 2nd floor rods

NE North Rod	MNENR2 = 0	kN	or	MNENR2I = 0	kips
NE South Rod	MNESR2 = 0	kN	or	MNESR2I = 0	kips
SE North Rod	MSENR2 = 0	kN	or	MSENR2I = 0	kips
SE South Rod	MSESR2 = 0	kN	or	MSESR2I = 0	kips

b. Base level rods

NE North Rod	MNENRB = 1.144	kN	or	MNENRBI = 0.257	kips
NE South Rod	MNESRB = 0.297	kN	or	MNESRBI = 0.067	kips
SE North Rod	MSENRB = 0.44	kN	or	MSENRBI = 0.099	kips
SE South Rod	MSESRB = 2.063	kN	or	MSESRBI = 0.464	kips

Note: Any prestressed load in the anchors before the earthquake simulation test was removed by zeroing the beginning of each record. So the loads presented in the following plots should be interpreted as just the dynamic component of the rod loads, and not the total rod loads.

Title = "Test 10 - Nahanni"

Description = "Two Storey OSB panel wall system with stucco"

Date_of_Test = "13 June 2001"

Section IV

Relative Displacements and Drifts

The following relative displacements with respect to the Base Level are considered here:
a) displacement of the 2nd floor, and
b) displacement of the Roof Level.

The drifts considered here are the *drift of the 2nd Floor* with respect to the Base Level and the *drift of the Roof Level* with respect to the 2nd Floor.

The peak values are:

a. Relative Displacements

a.i) Roof vs. Base Level					
East Wall	MRBDE = 0.43	cm	or	MRBDEI = 0.169	in
Center Wall	MRBDC = 0.638	cm	or	MRBDCI = 0.251	in
West Wall	MRBDW = 0.447	cm	or	MRBDWI = 0.176	in
a.ii) 2nd Floor vs. Base Level					
East Wall	M2BDE = 0.36	cm	or	M2BDEI = 0.142	in
Center Wall	M2BDC = 0.525	cm	or	M2BDCI = 0.207	in
West Wall	M2BDW = 0.356	cm	or	M2BDWI = 0.14	in

b. Drifts

b.i) Roof vs. 2nd Floor					
East Wall	MRBDDE = 0.217	cm	or	MRBDDEI = 0.085	in
Center Wall	MRBDDC = 0.185	cm	or	MRBDDCI = 0.073	in
West Wall	MRBDDW = 0.18	cm	or	MRBDDWI = 0.071	in
b.ii) 2nd Floor vs. Base Level					
East Wall	M2BDDE = 0.36	cm	or	M2BDDEI = 0.142	in
Center Wall	M2BDDC = 0.525	cm	or	M2BDDCI = 0.207	in
West Wall	M2BDDW = 0.356	cm	or	M2BDDWI = 0.14	in

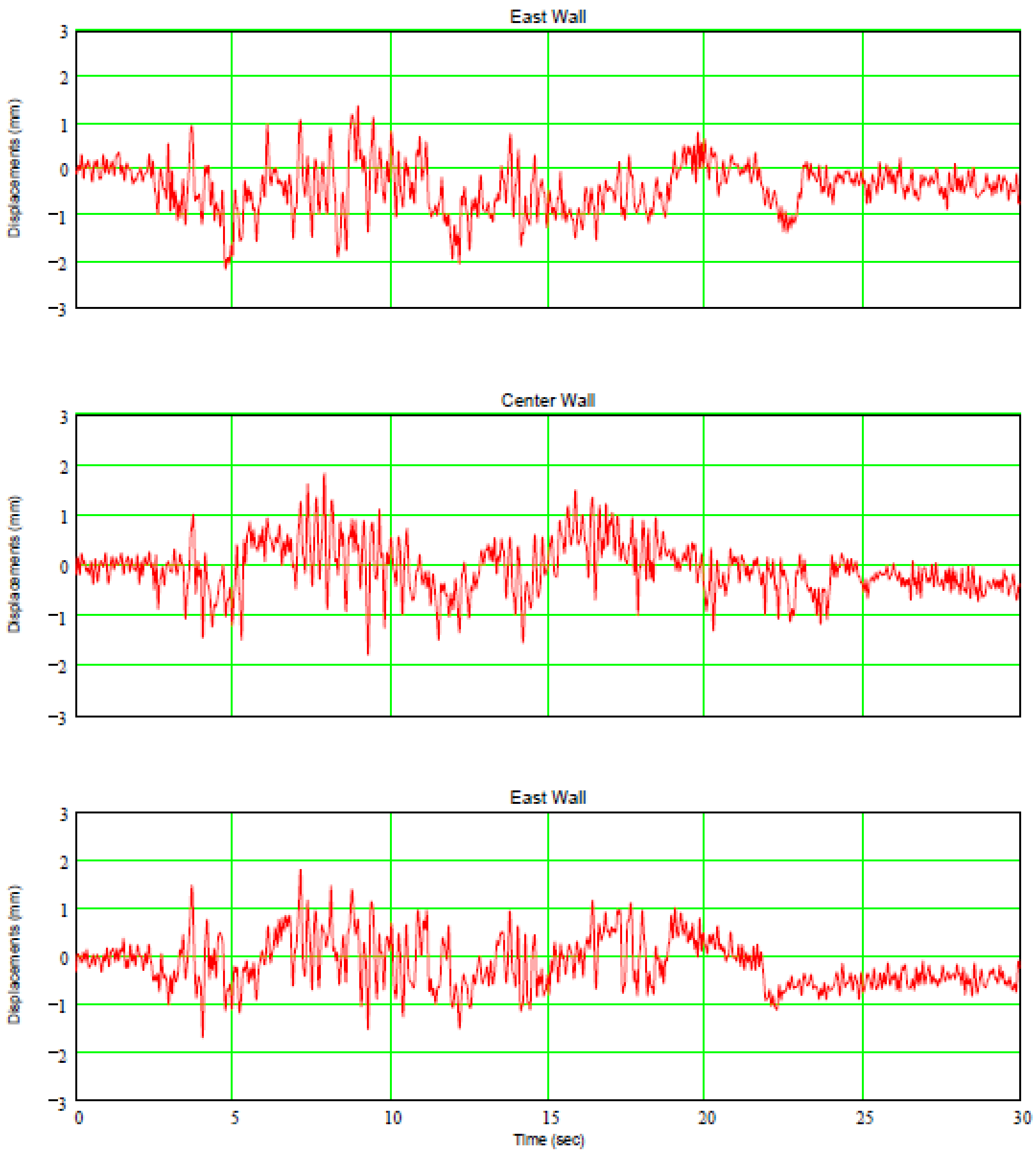
The Time histories of relative displacements and drift are presented in the following pages.

4. Relative Displacements & Drifts.

Title ▬ "Test 10 - Nahanni"

Description ▬ "Two Storey OSB panel wall system with stucco"

Drift - Roof Level vs. 2nd Floor

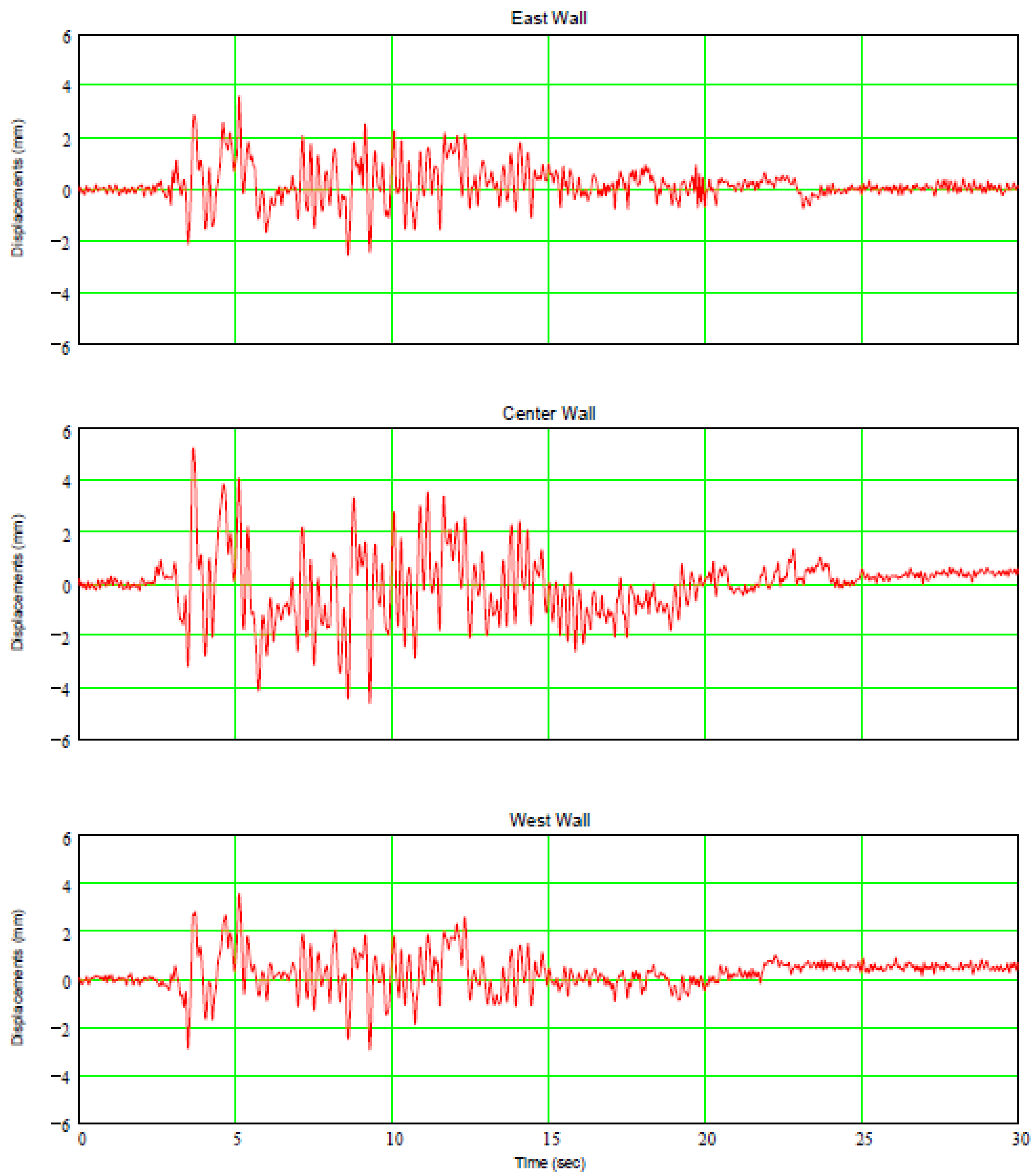


4. Relative Displacements & Drifts.

Title - "Test 10 - Nahanni"

Description - "Two Storey OSB panel wall system with stucco"

Drift - 2nd Floor vs. Base Level



file: Summary T 10 R 1 New.mcd | Page 8 of 11 | Date processed: 19/04/2002

Title = "Test 10 - Nahanni"

Description = "Two Storey OSB panel wall system with stucco"

Date_of_Test = "13 June 2001"

Section V

Load - Deformation Diagrams

The load used for the following diagrams was calculated as the base shear resulting from adding the story shears from each floor. Each story shear was computed as the product of the measured absolute acceleration and the calculated floor mass. The floor weight was calculated to be 10.1 and 9.7 metric tons for the roof and second floor respectively. Since the measured actuator load include the inertia load of the shake table and part of the weight of the first floor it was concluded that a better representation of the base shear could be obtained from the computed story shears rather than by a removing these inertia loads from the measured actuator loads.

Maximum Loads

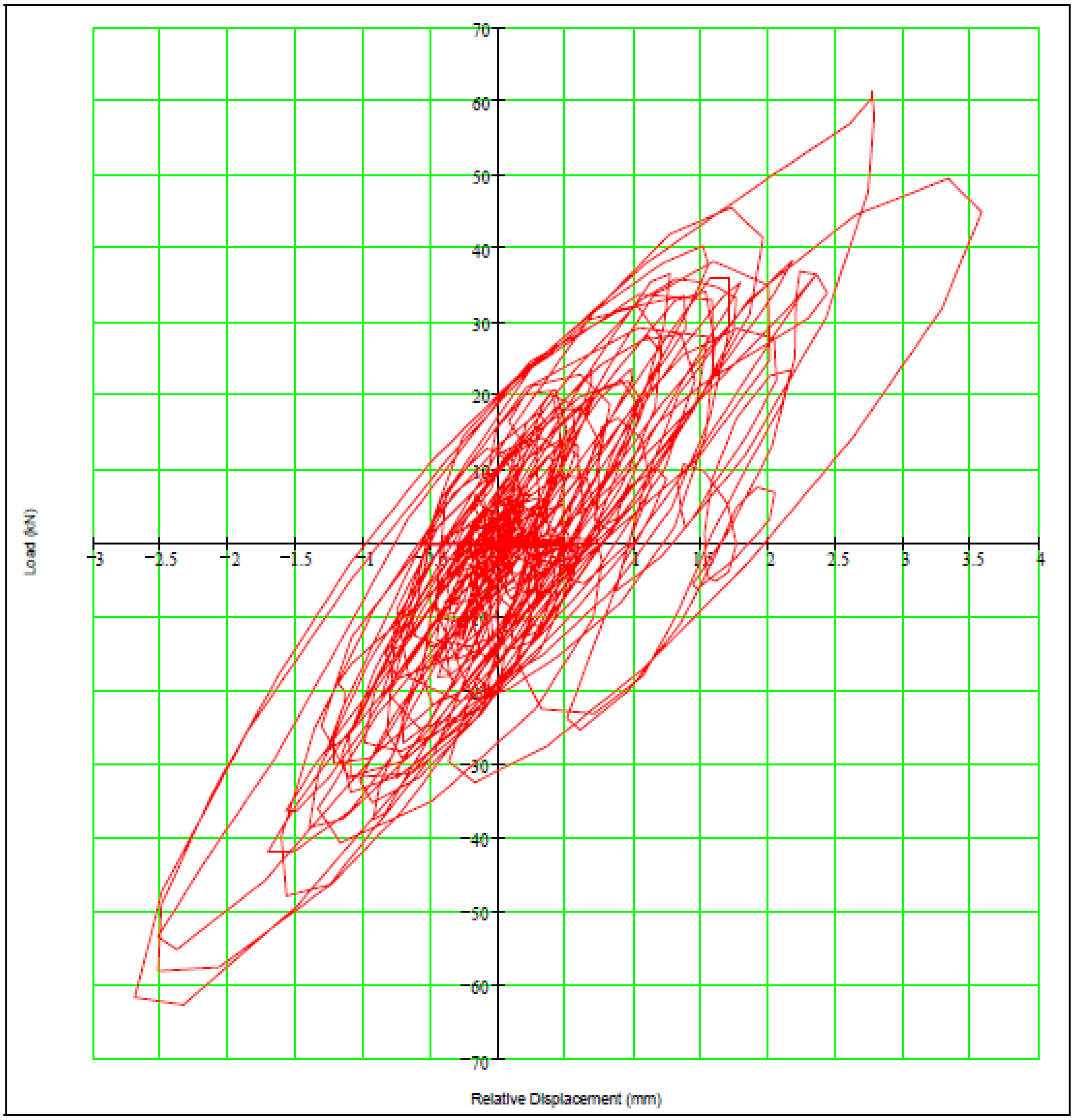
Maximum load for the First Storey (kN): $\max\left(\left|\overrightarrow{CLDF}\right|\right) = 62.57$

5. Load-Deformation Diagrams.

Title = "Test 10 - Nahanni"

Description = "Two Storey OSB panel wall system with stucco"

Load-Deformation, First Storey



Title = "Test 10 - Nahanni"

Description = "Two Storey OSB panel wall system with stucco"

Date_of_Test = "13 June 2001"

Section VI

Calculated Velocity

The velocity in each floor is calculated from the the differential of the displacement. A subroutine to differentiate displacement signals by finite differences was written for this purpose. The resulting the peak velocities for each floor are shown below.

	West		Center		East	
Roof Level	MV3 = 32.696	cm/s	MV2 = 34.037	cm/s	MV1 = 33.454	cm/s
2nd Floor	MV6 = 32.013	cm/s	MV5 = 33.936	cm/s	MV4 = 32.064	cm/s
Basement Level	MV9 = 31.429	cm/s	MV8 = 31.329	cm/s	MV7 = 31.13	cm/s

or

Roof Level	MIV3 = 12.872	in/s	MIV2 = 13.4	in/s	MIV1 = 13.171	in/s
2nd Floor	MIV6 = 12.604	in/s	MIV5 = 13.361	in/s	MIV4 = 12.624	in/s
Basement Level	MIV9 = 12.374	in/s	MIV8 = 12.334	in/s	MIV7 = 12.256	in/s

Reference:C:\Documents and Settings\Mehdi H. K. Kharrazi\Desktop\Earthquake 99 Final Edition\EQ 99 - TWO STOREY (Part I)\Test 1

Department of Civil Engineering
The University of British Columbia

Summary of Earthquake 99 Project
2-Storey House Shake Table Test

Data Analysis Project Information:

Title = "Test 11 - Nahanni Earthquake"

Description = "2 Storey OSB system with stucco & rain screen"

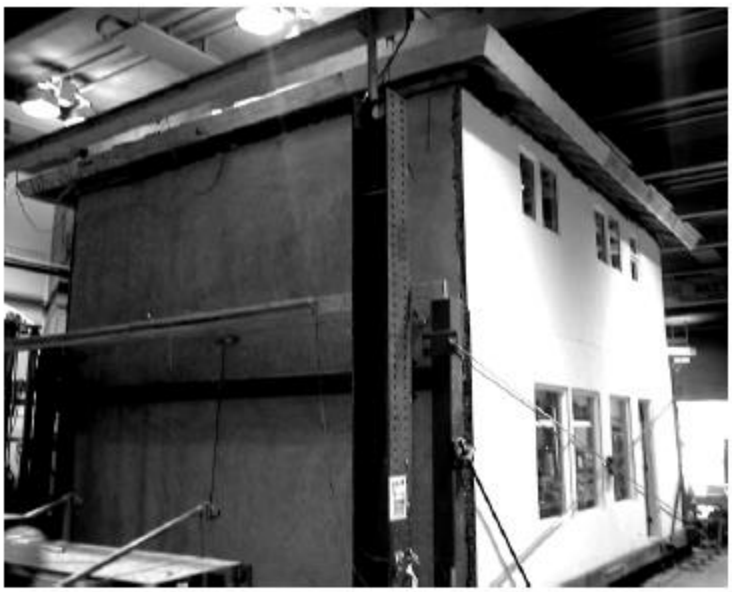
Date_of_Test = "26 June 2001"

Test_No = 11

Run_sequence = 1

Directed and Supervised by C.E.Ventura

Analysis done by M.H.K.Kharrazi / C.E.Ventura



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Section **Page**

Title = "Test 11 - Nahanni Earthquake"

Description = "2 Storey OSB system with stucco & rain screen"

Date_of_Test = "26 June 2001"

Section I

Shake Table Motions

This section presents the recorded shake table motions (base level motion) during this test.

The information presented here includes:

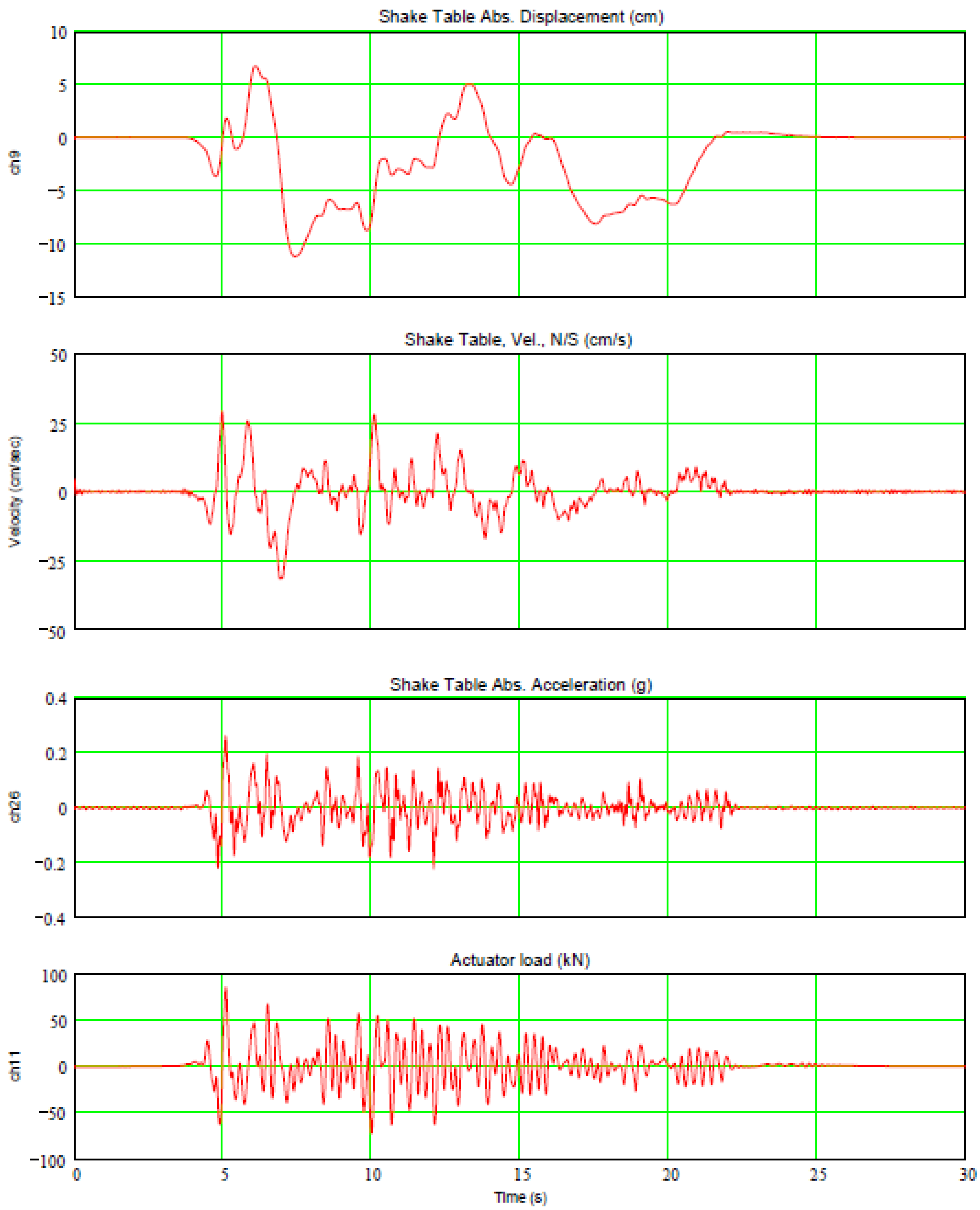
a) shake table actuator displacement (in cm)
b) shake table inferred velocity (in cm/sec)
c) shake table actuator acceleration (in g's)
d) shake table actuator load (in kN)

The recorded peak values (after signal conditioning) of these motions are:

a) Displacement:	Mdisp = 11.243	cm	=	Mdispl = 4.426	in
b) Velocity:	MVel = 31.568	cm/sec	=	MVell = 12.428	in/sec
c) Acceleration:	Maccl = 0.263	g			
d) Load:	MLoad = 86.17	kN	=	MLoadl = 19.373	kips

1. Shake Table Motions Title = "Test 11 - Nahanni Earthquake" Date_of_Test = "26 June 2001"

Description = "2 Storey OSB system with stucco & rain screen"



file: Summary T 11 R 1 New.mcd Page 3 of 11 Date processed: 19/04/2002

Title = "Test 11 - Nahanni Earthquake"

Description = "2 Storey OSB system with stucco & rain screen"

Date_of_Test = "26 June 2001"

Section II

Horizontal Absolute Accelerations

The peak values of absolute acceleration are:

a. Roof Level (N/S direction)

East Wall	MARLE = 0.358	g
Center Wall	MARLC = 0.38	g
West Wall	MARLW = 0.352	g

b. 2nd Floor (N/S Direction)

East Wall	MA2LE = 0.341	g
Center Wall	MA2LC = 0.325	g
West Wall	MA2LW = 0.321	g

c. Base Level (N/S Direction)

East Wall	MABLE = 0.294	g
Center Wall	MABLC = 0.263	g
West Wall	MABLW = 0.274	g

Title = "Test 11 - Nahanni Earthquake"

Description = "2 Storey OSB system with stucco & rain screen"

Date_of_Test = "26 June 2001"

Section III

Anchor Rod Loads

The peak values of the measured loads are as follow:

a. 2nd floor rods

NE North Rod	MNENR2 = 0	kN	or	MNENR2I = 0	kips
NE South Rod	MNESR2 = 0	kN	or	MNESR2I = 0	kips
SE North Rod	MSENR2 = 0	kN	or	MSENR2I = 0	kips
SE South Rod	MSESR2 = 0	kN	or	MSESR2I = 0	kips

b. Base level rods

NE North Rod	MNENRB = 0.346	kN	or	MNENRBI = 0.078	kips
NE South Rod	MNESRB = 0.235	kN	or	MNESRBI = 0.053	kips
SE North Rod	MSENRB = 0.419	kN	or	MSENRBI = 0.094	kips
SE South Rod	MSESRB = 1.428	kN	or	MSESRBI = 0.321	kips

Note: Any prestressed load in the anchors before the earthquake simulation test was removed by zeroing the beginning of each record. So the loads presented in the following plots should be interpreted as just the dynamic component of the rod loads, and not the total rod loads.

Section IV

Relative Displacements and Drifts

The following relative displacements with respect to the Base Level are considered here:

a) displacement of the 2nd floor, and
b) displacement of the Roof Level.

The drifts considered here are the *drift of the 2nd Floor* with respect to the Base Level and the *drift of the Roof Level* with respect to the 2nd Floor.

The peak values are:

a. Relative Displacements

a.i) Roof vs. Base Level

East Wall	MRBDE = 0.529	cm	or	MRBDEI = 0.208	in
Center Wall	MRBDC = 0.788	cm	or	MRBDCI = 0.31	in
West Wall	MRBDW = 0.618	cm	or	MRBDWI = 0.243	in

a.ii) 2nd Floor vs. Base Level

East Wall	M2BDE = 0.454	cm	or	M2BDEI = 0.179	in
Center Wall	M2BDC = 0.681	cm	or	M2BDCI = 0.268	in
West Wall	M2BDW = 0.475	cm	or	M2BDWI = 0.187	in

b. Drifts

b.i) Roof vs. 2nd Floor

East Wall	MRBDDE = 0.287	cm	or	MRBDDEI = 0.113	in
Center Wall	MRBDDC = 0.201	cm	or	MRBDDCI = 0.079	in
West Wall	MRBDDW = 0.236	cm	or	MRBDDWI = 0.093	in

b.ii) 2nd Floor vs. Base Level

East Wall	M2BDDE = 0.454	cm	or	M2BDDEI = 0.179	in
Center Wall	M2BDDC = 0.681	cm	or	M2BDDCI = 0.268	in
West Wall	M2BDDW = 0.475	cm	or	M2BDDWI = 0.187	in

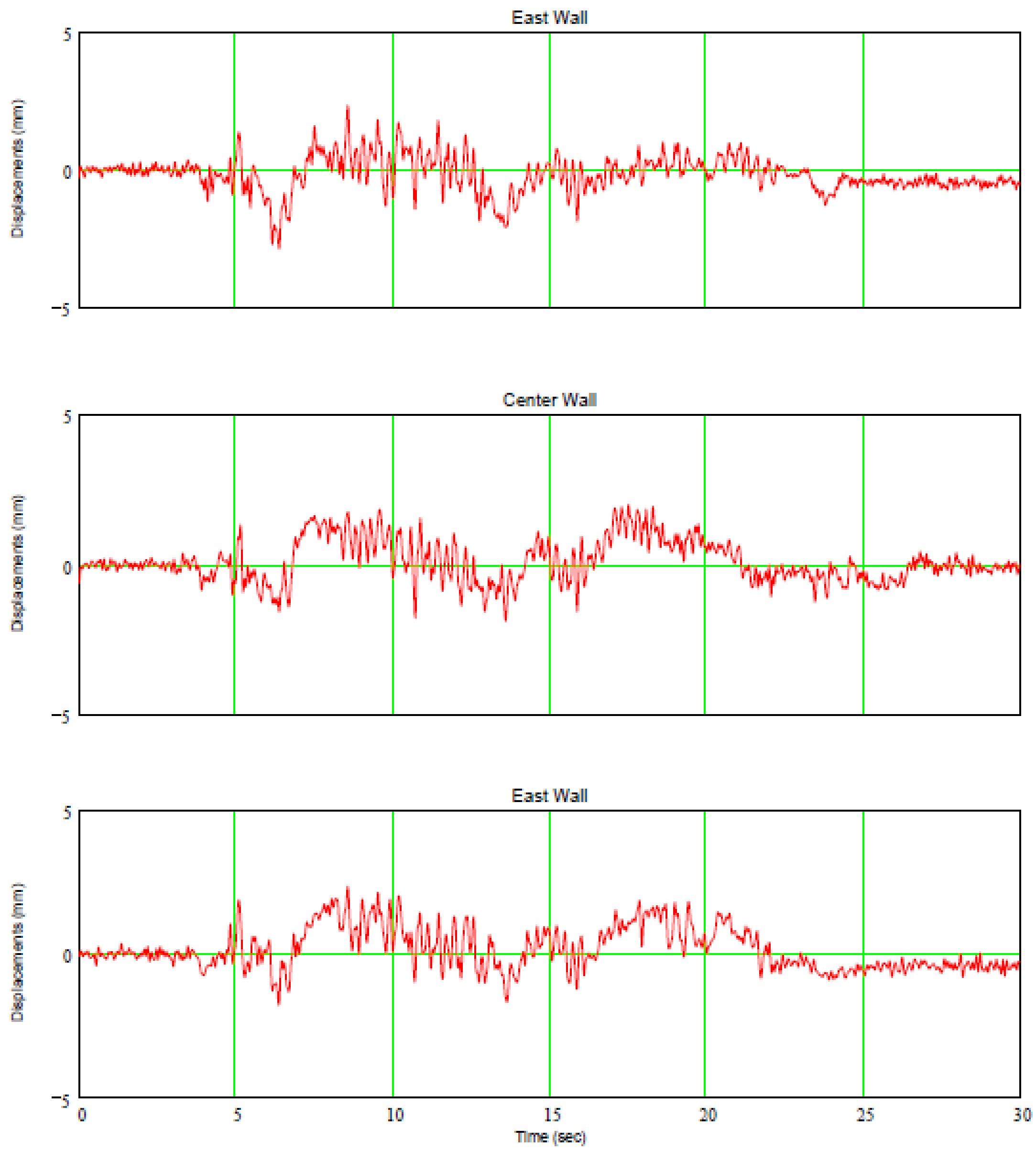
The Time histories of relative displacements and drift are presented in the following pages.

4. Relative Displacements & Drifts.

Title - "Test 11 - Nahanni Earthquake"

Description - "2 Storey OSB system with stucco & rain screen"

Drift - Roof Level vs. 2nd Floor

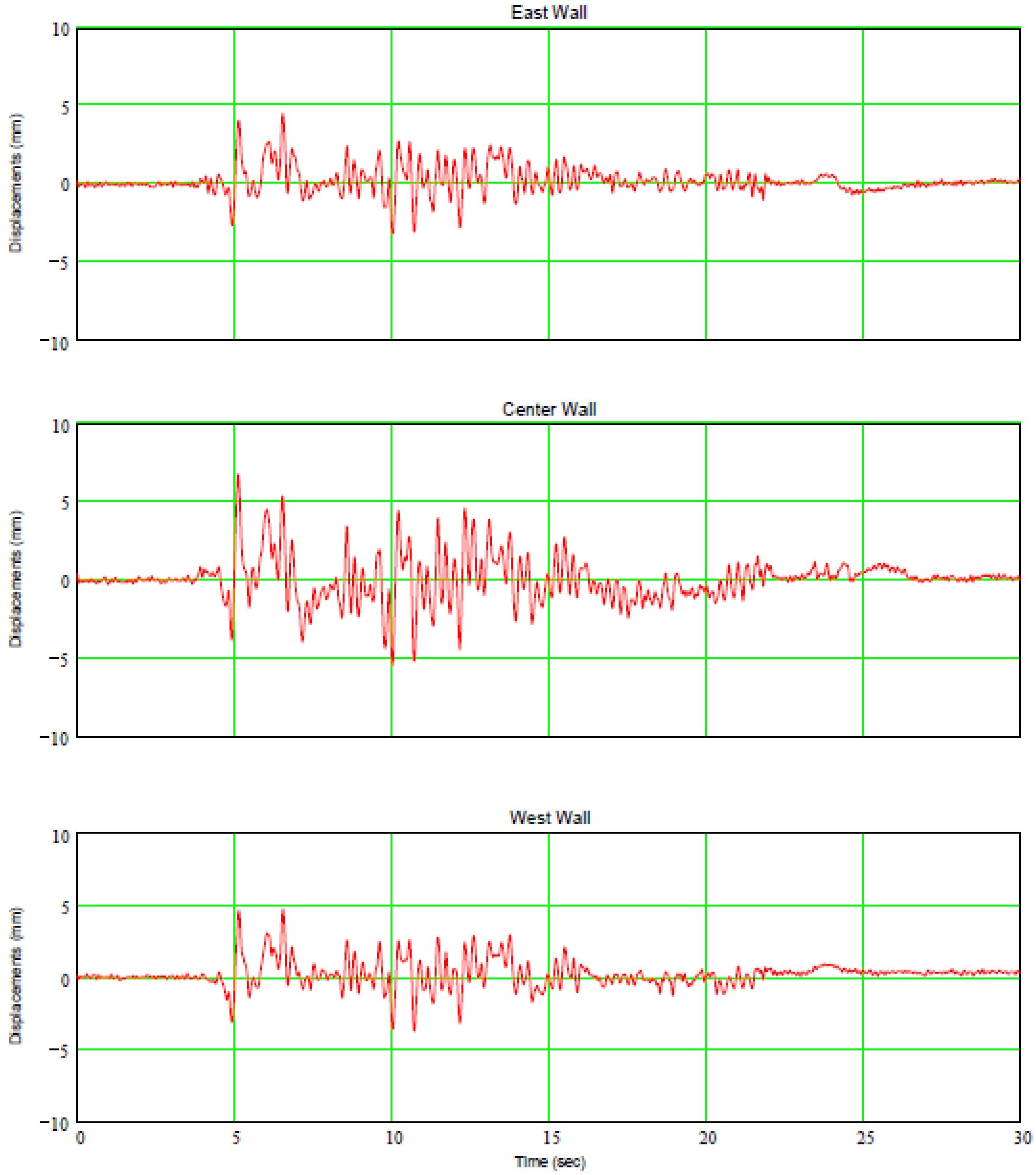


4. Relative Displacements & Drifts.

Title = "Test 11 - Nahanni Earthquake"

Description = "2 Storey OSB system with stucco & rain screen"

Drift - 2nd Floor vs. Base Level



Title = "Test 11 - Nahanni Earthquake"

Description = "2 Storey OSB system with stucco & rain screen"

Date_of_Test = "26 June 2001"

Section V

Load - Deformation Diagrams

The load used for the following diagrams was calculated as the base shear resulting from adding the story shears from each floor. Each story shear was computed as the product of the measured absolute acceleration and the calculated floor mass. Each floor weight was calculated to be 10 metric tons. Since the measured actuator load include the inertia load of the shake table and part of the weight of the first floor it was concluded that a better representation of the base shear could be obtained from the computed story shears rather than by a removing these inertia loads from the measured actuator loads."

Maximum Loads

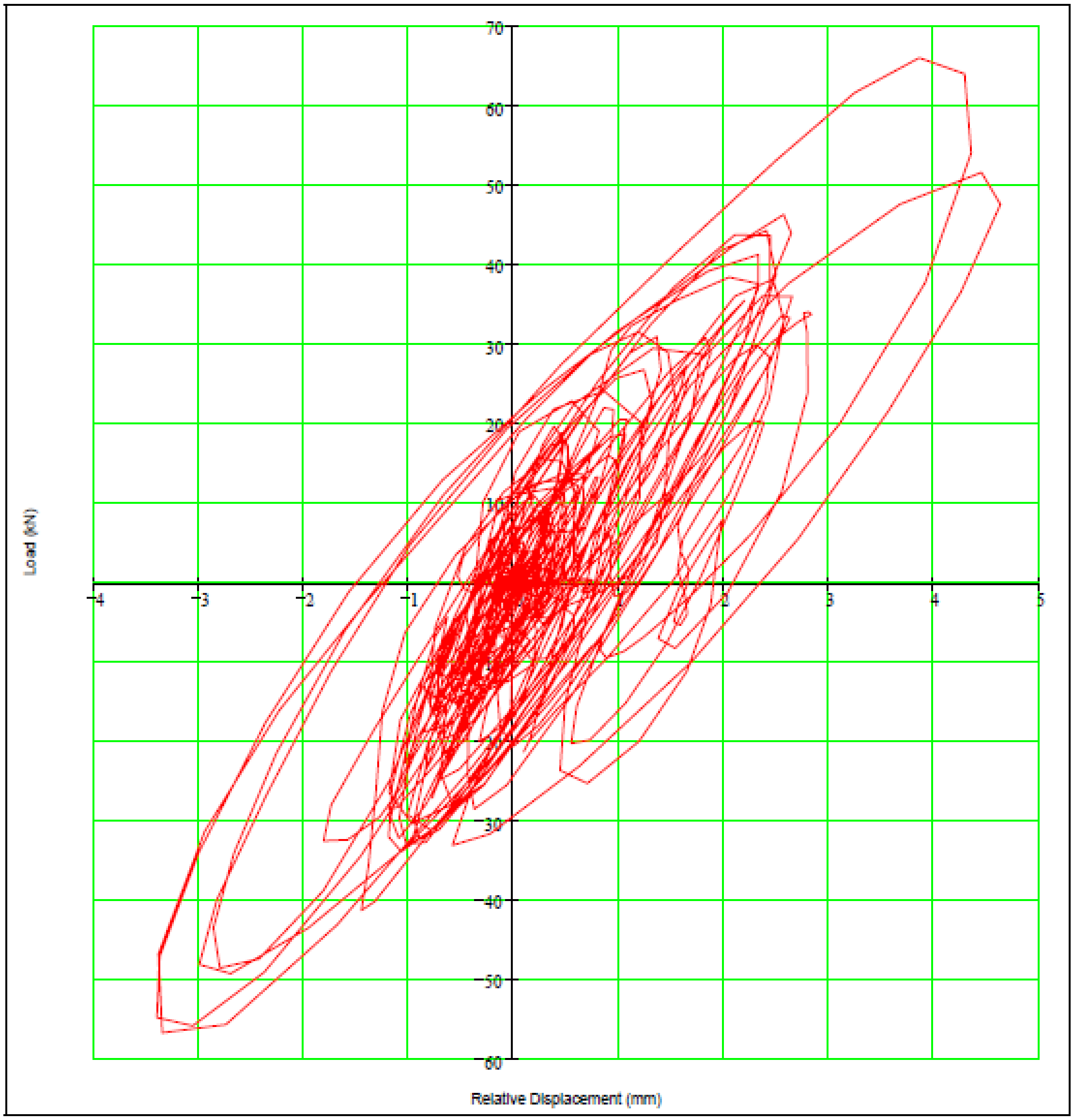
Maximum load for the First Storey (kN): $\max\left(\left|\overrightarrow{CLDF}\right|\right) = 66.184$

5. Load-Deformation Diagrams.

Title = "Test 11 - Nahanni Earthquake"

Description = "2 Storey OSB system with stucco & rain screen"

Load-Deformation, First Storey



Title = "Test 11 - Nahanni Earthquake"

Description = "2 Storey OSB system with stucco & rain screen"

Date_of_Test = "26 June 2001"

Section VI

Calculated Velocity

The velocity in each floor is calculated from the the differential of the displacement. A subroutine to differentiate displacement signals by finite differences was written for this purpose. The resulting the peak velocities for each floor are shown below.

	West		Center		East	
Roof Level	MV3 = 33.826	cm/s	MV2 = 35.672	cm/s	MV1 = 33.794	cm/s
2nd Floor	MV6 = 32.98	cm/s	MV5 = 34.067	cm/s	MV4 = 32.871	cm/s
Basement Level	MV9 = 31.913	cm/s	MV8 = 31.568	cm/s	MV7 = 31.199	cm/s

or

Roof Level	MIV3 = 13.317	in/s	MIV2 = 14.044	in/s	MIV1 = 13.305	in/s
2nd Floor	MIV6 = 12.984	in/s	MIV5 = 13.412	in/s	MIV4 = 12.941	in/s
Basement Level	MIV9 = 12.564	in/s	MIV8 = 12.428	in/s	MIV7 = 12.283	in/s

Department of Civil Engineering
The University of British Columbia

Summary of Earthquake 99 Project
2-Storey House Shake Table Test

Data Analysis Project Information:

Title = "Test 12 - Landers 1992"

Description = "2 Storey OSB system w/o stucco and hold- down "

Date_of_Test = "17 July 2001"

Test_No = 12

Run_sequence = 1

Directed and Supervised by C.E.Ventura

Analysis done by M.H.K.Kharrazi / C.E.Ventura

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Section **Page**

Title = "Test 12 - Landers 1992"

Description = "2 Storey OSB system w/o stucco and hold- down "

Date_of_Test = "17 July 2001"

Section I

Shake Table Motions

This section presents the recorded shake table motions (base level motion) during this test.

The information presented here includes:

a) shake table actuator displacement (in cm)
b) shake table inferred velocity (in cm/sec)
c) shake table actuator acceleration (in g's)
d) shake table actuator load (in kN)

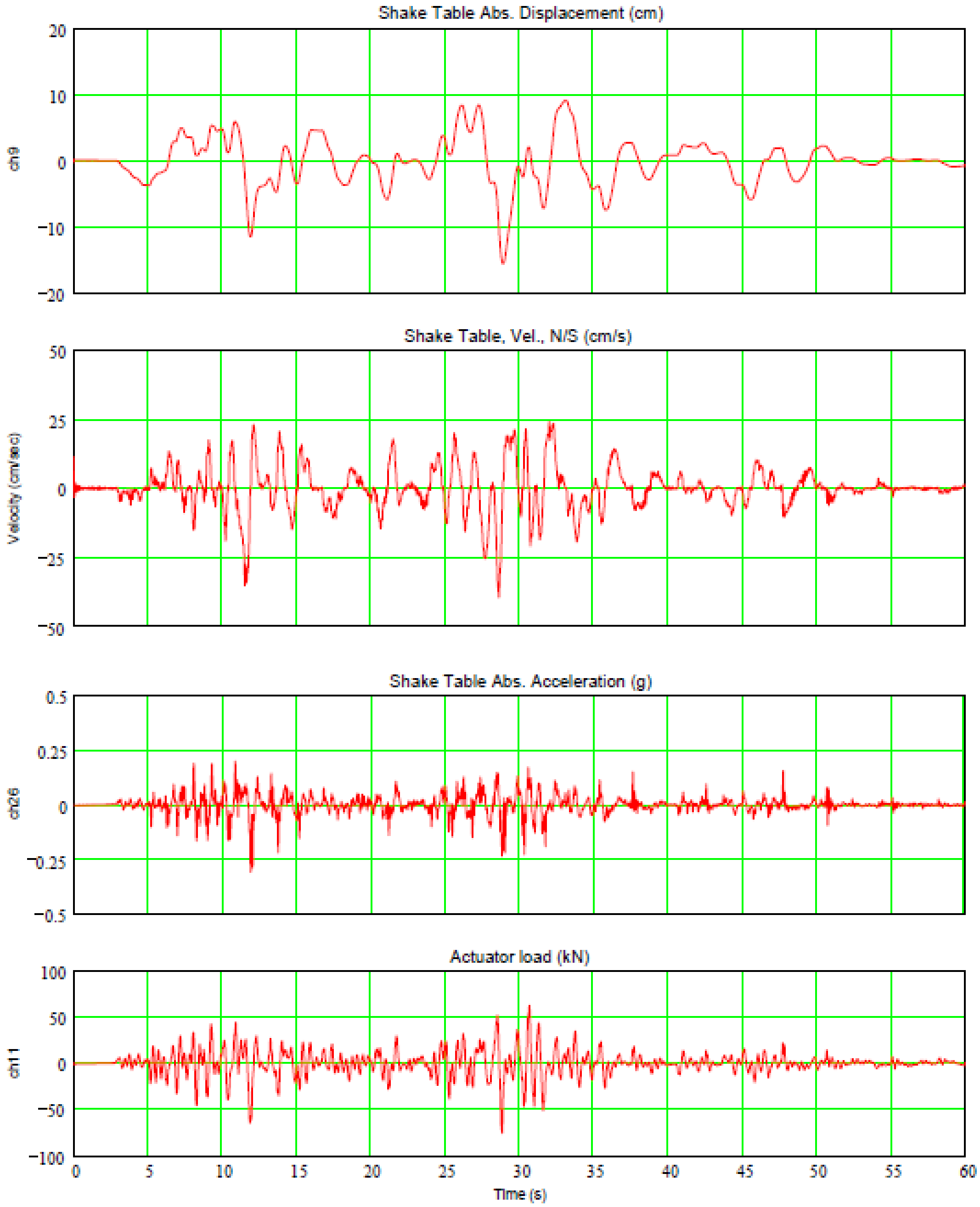
The recorded peak values (after signal conditioning) of these motions are:

a) Displacement:	Mdisp = 15.574	cm	=	Mdispl = 6.132	in
b) Velocity:	MVel = 39.481	cm/sec	=	MVell = 15.544	in/sec
c) Acceleration:	Maccl = 0.308	g			
d) Load:	MLoad = 75.482	kN	=	MLoadl = 16.97	kips

1. Shake Table Motions

Title = "Test 12 - Landers 1992" Date_of_Test = "17 July 2001"

Description = "2 Storey OSB system w/o stucco and hold- down "



Title = "Test 12 - Landers 1992"

Description = "2 Storey OSB system w/o stucco and hold- down "

Date_of_Test = "17 July 2001"

Section II

Horizontal Absolute Accelerations

The peak values of absolute acceleration are:

a. Roof Level (N/S direction)

East Wall	MARLE = 0.383	g
Center Wall	MARLC = 0.412	g
West Wall	MARLW = 0.371	g

b. 2nd Floor (N/S Direction)

East Wall	MA2LE = 0.437	g
Center Wall	MA2LC = 0.413	g
West Wall	MA2LW = 0.356	g

c. Base Level (N/S Direction)

East Wall	MABLE = 0.283	g
Center Wall	MABLC = 0.308	g
West Wall	MABLW = 0.3	g

Title = "Test 12 - Landers 1992"

Description = "2 Storey OSB system w/o stucco and hold- down "

Date_of_Test = "17 July 2001"

Section III

Anchor Rod Loads

The peak values of the measured loads are as follow:

a. 2nd floor rods

NE North Rod	MNENR2 = 0	kN	or	MNENR2I = 0	kips
NE South Rod	MNESR2 = 0	kN	or	MNESR2I = 0	kips
SE North Rod	MSENR2 = 0	kN	or	MSENR2I = 0	kips
SE South Rod	MSESR2 = 0	kN	or	MSESR2I = 0	kips

b. Base level rods

NE North Rod	MNENRB = 0	kN	or	MNENRBI = 0	kips
NE South Rod	MNESRB = 0	kN	or	MNESRBI = 0	kips
SE North Rod	MSENRB = 0	kN	or	MSENRBI = 0	kips
SE South Rod	MSESRB = 0	kN	or	MSESRBI = 0	kips

Note: Any prestressed load in the anchors before the earthquake simulation test was removed by zeroing the beginning of each record. So the loads presented in the following plots should be interpreted as just the dynamic component of the rod loads, and not the total rod loads.

Title = "Test 12 - Landers 1992"

Description = "2 Storey OSB system w/o stucco and hold- down "

Date_of_Test = "17 July 2001"

Section IV

Relative Displacements and Drifts

The following relative displacements with respect to the Base Level are considered here:

a) displacement of the 2nd floor, and

b) displacement of the Roof Level.

The drifts considered here are the *drift of the 2nd Floor* with respect to the Base Level and the *drift of the Roof Level* with respect to the 2nd Floor.

The peak values are:

a. Relative Displacements

a.i) Roof vs. Base Level						
East Wall	MRBDE = 1.713	cm	or	MRBDEI = 0.674	in
Center Wall	MRBDC = 2.131	cm	or	MRBDCI = 0.839	in
West Wall	MRBDW = 1.782	cm	or	MRBDWI = 0.702	in
a.ii) 2nd Floor vs. Base Level					
East Wall	M2BDE = 1.313	cm	or	M2BDEI = 0.517	in
Center Wall	M2BDC = 1.66	cm	or	M2BDCI = 0.654	in
West Wall	M2BDW = 1.455	cm	or	M2BDWI = 0.573	in

b. Drifts

b.i) Roof vs. 2nd Floor					
East Wall	MRBDDE = 0.471	cm	or	MRBDDEI = 0.186	in
Center Wall	MRBDDC = 0.475	cm	or	MRBDDCI = 0.187	in
West Wall	MRBDDW = 0.501	cm	or	MRBDDWI = 0.197	in
b.ii) 2nd Floor vs. Base Level					
East Wall	M2BDDE = 1.313	cm	or	M2BDDEI = 0.517	in
Center Wall	M2BDDC = 1.66	cm	or	M2BDDCI = 0.654	in
West Wall	M2BDDW = 1.455	cm	or	M2BDDWI = 0.573	in

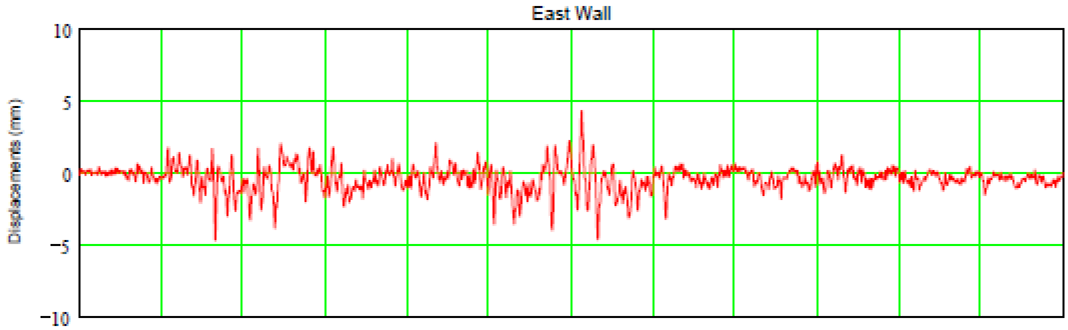
The Time histories of relative displacements and drift are presented in the following pages.

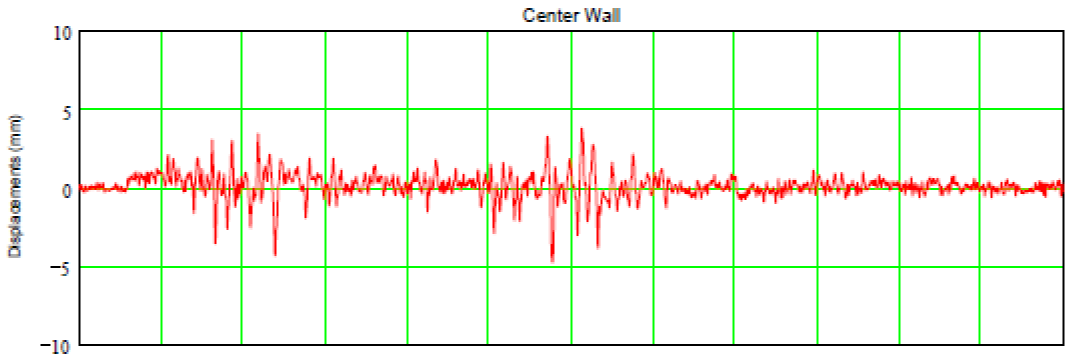
4. Relative Displacements & Drifts.

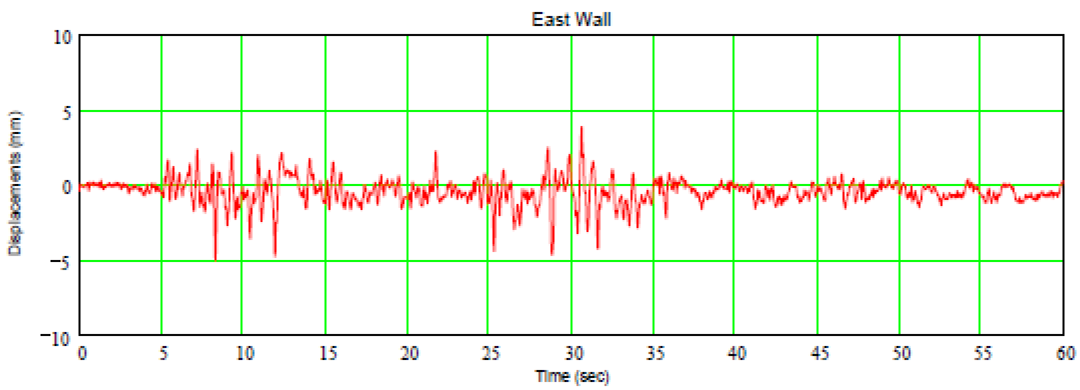
Title = "Test 12 - Landers 1992"

Description = "2 Storey OSB system w/o stucco and hold- down "

Drift - Roof Level vs. 2nd Floor





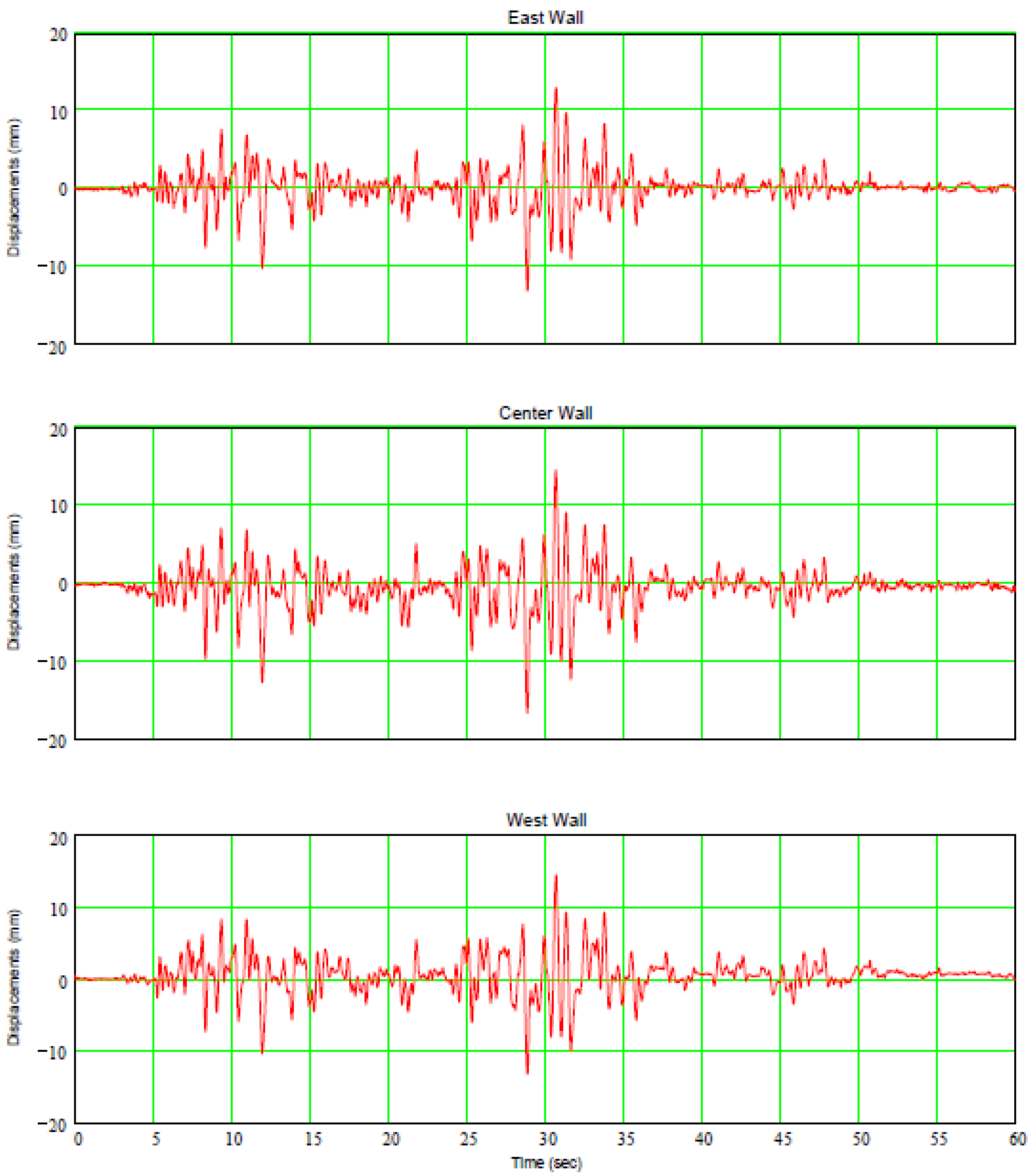


4. Relative Displacements & Drifts.

Title = "Test 12 - Landers 1992"

Description = "2 Storey OSB system w/o stucco and hold- down "

Drift - 2nd Floor vs. Base Level



Title = "Test 12 - Landers 1992"

Description = "2 Storey OSB system w/o stucco and hold- down "

Date_of_Test = "17 July 2001"

Section V

Load - Deformation Diagrams

The load used for the following diagrams was calculated as the base shear resulting from adding the story shears from each floor. Each story shear was computed as the product of the measured absolute acceleration and the calculated floor mass. The floor weight was calculated to be 9.0 and 8.8 metric tons for the roof and second floor respectively. Since the measured actuator load include the inertia load of the shake table and part of the weight of the first floor it was concluded that a better representation of the base shear could be obtained from the computed story shears rather than by a removing these inertia loads from the measured actuator loads.

Maximum Loads

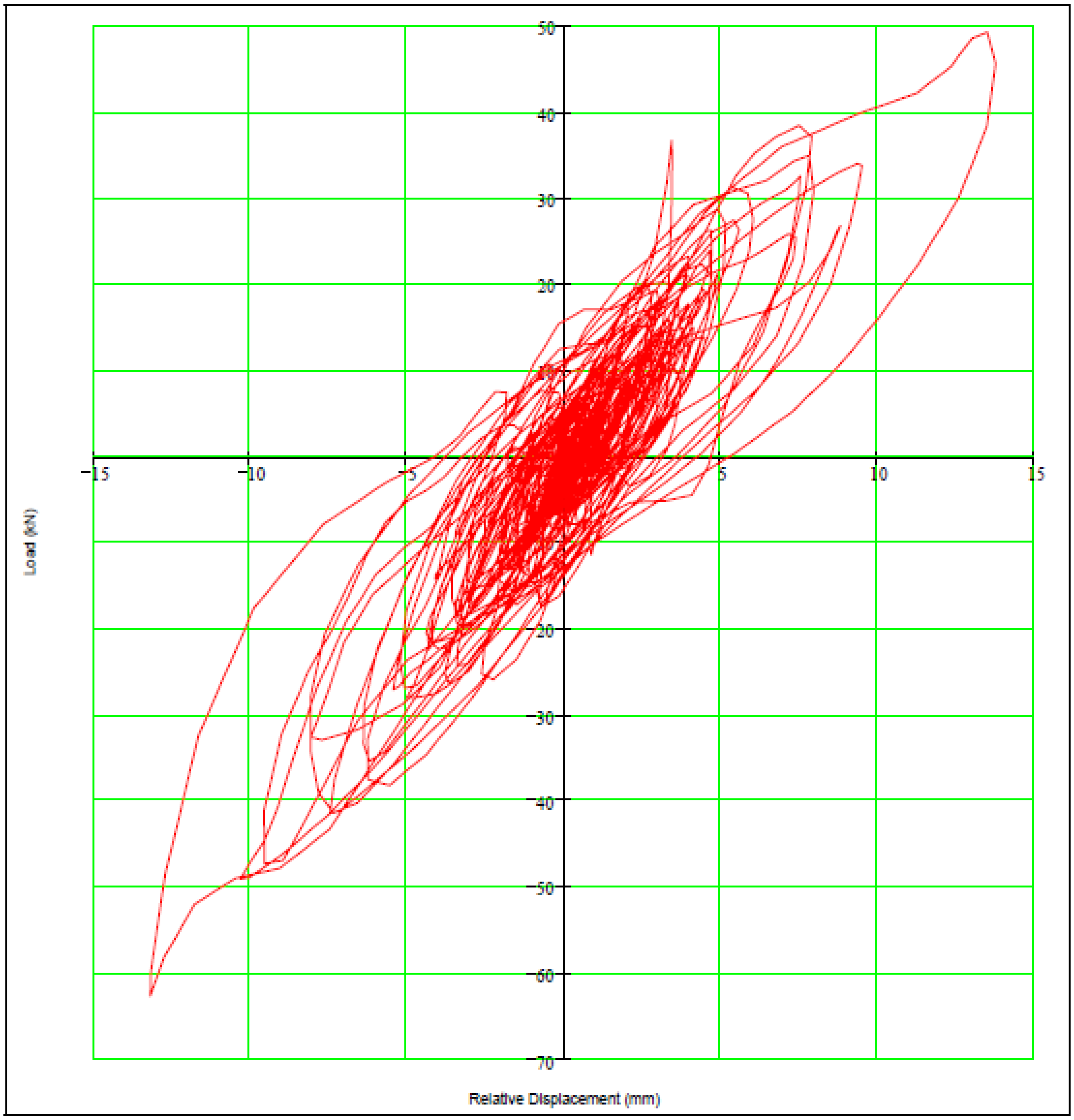
Maximum load for the First Storey (kN): $\max\left(\overrightarrow{|CLDF|}\right) = 62.613$

5. Load-Deformation Diagrams.

Title = "Test 12 - Landers 1992"

Description = "2 Storey OSB system w/o stucco and hold- down "

Load-Deformation, First Storey



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Title = "Test 12 - Landers 1992"

Description = "2 Storey OSB system w/o stucco and hold- down "

Date_of_Test = "17 July 2001"

Section VI

Calculated Velocity

The velocity in each floor is calculated from the the differential of the displacement. A subroutine to differentiate displacement signals by finite differences was written for this purpose. The resulting the peak velocities for each floor are shown below.

	West		Center		East	
Roof Level	MV3 = 50.332	cm/s	MV2 = 48.556	cm/s	MV1 = 48.964	cm/s
2nd Floor	MV6 = 47.247	cm/s	MV5 = 46.244	cm/s	MV4 = 47.416	cm/s
Basement Level	MV9 = 38.562	cm/s	MV8 = 39.481	cm/s	MV7 = 37	cm/s

or

Roof Level	MIV3 = 19.816	in/s	MIV2 = 19.116	in/s	MIV1 = 19.277	in/s
2nd Floor	MIV6 = 18.601	in/s	MIV5 = 18.206	in/s	MIV4 = 18.668	in/s
Basement Level	MIV9 = 15.182	in/s	MIV8 = 15.544	in/s	MIV7 = 14.567	in/s

Reference:C:\Documents and Settings\Mehdi H. K. Kharrazi\Desktop\Earthquake 99 Final Edition\EQ 99 - TWO STOREY (Part I)\Test

Department of Civil Engineering
The University of British Columbia

Summary of Earthquake 99 Project
2-Storey House Shake Table Test

Data Analysis Project Information:

Title = "Test 13 - Kobe 1995"

Description = "2 Storey board sheathing w/o stucco and hold-down"

Date_of_Test = "27 July 2001"

Test_No = 13

Run_sequence = 1

Directed and Supervised by C.E.Ventura

Analysis done by M.H.K.Kharrazi / C.E.Ventura

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Title = "Test 13 - Kobe 1995"

Description = "2 Storey board sheathing w/o stucco and hold-down"

Date_of_Test = "27 July 2001"

Section I

Shake Table Motions

This section presents the recorded shake table motions (base level motion) during this test.

The information presented here includes:

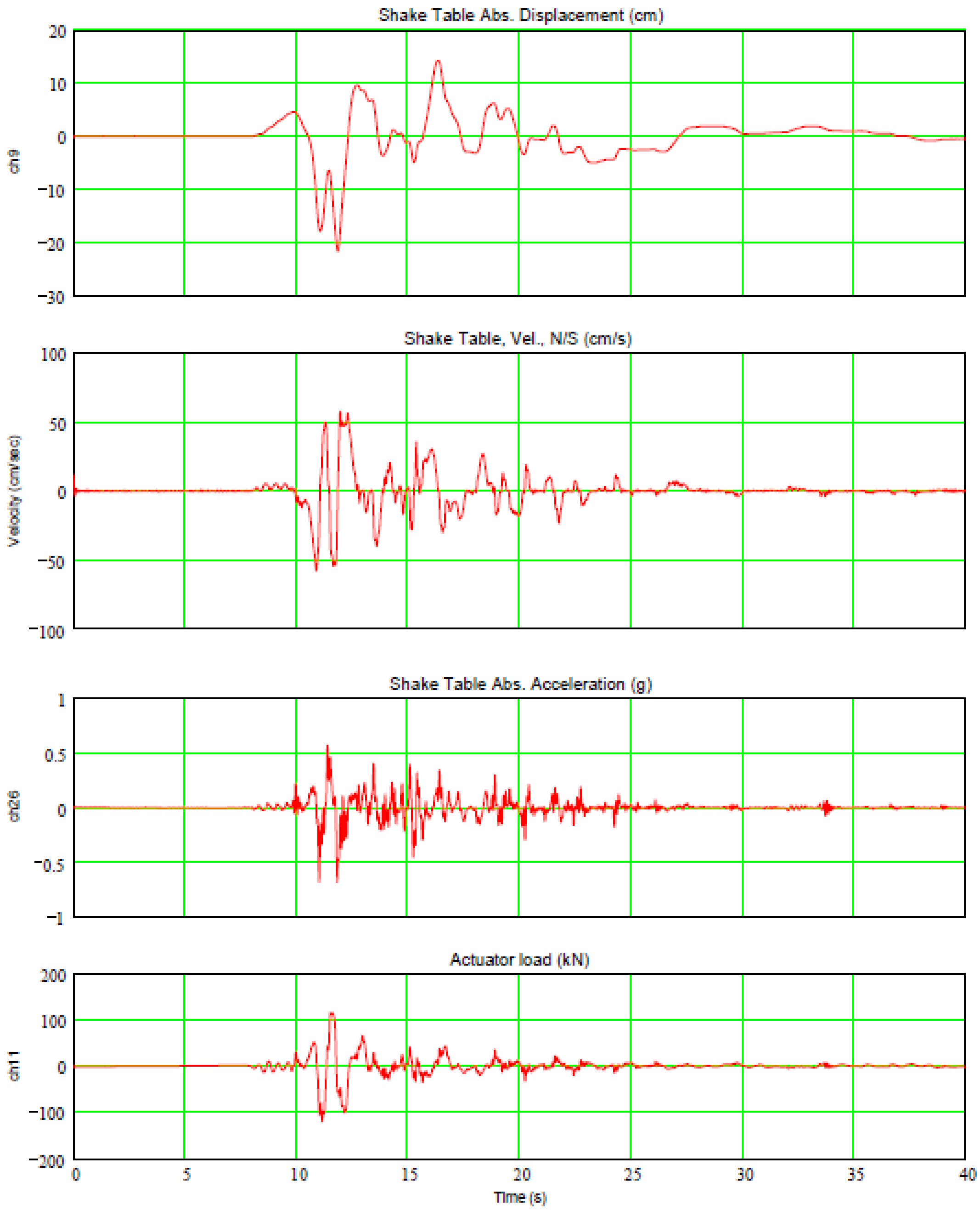
a) shake table actuator displacement (in cm)
b) shake table inferred velocity (in cm/sec)
c) shake table actuator acceleration (in g's)
d) shake table actuator load (in kN)

The recorded peak values (after signal conditioning) of these motions are:

a) Displacement:	Mdisp = 21.713	cm	=	Mdispl = 8.548	in
b) Velocity:	MVel = 57.882	cm/sec	=	MVell = 22.788	in/sec
c) Acceleration:	Maccl = 0.683	g			
d) Load:	MLoad = 119.474	kN	=	MLoadl = 26.86	kips

1. Shake Table Motions Title = "Test 13 - Kobe 1995" Date_of_Test = "27 July 2001"

Description = "2 Storey board sheathing w/o stucco and hold-down"



file: Summary T 13 R 1 New.mcd Page 3 of 11 Date processed: 19/04/2002

Title = "Test 13 - Kobe 1995"

Description = "2 Storey board sheathing w/o stucco and hold-down"

Date_of_Test = "27 July 2001"

Section II

Horizontal Absolute Accelerations

The peak values of absolute acceleration are:

a. Roof Level (N/S direction)

East Wall	MARLE = 0.598	g
Center Wall	MARLC = 0.726	g
West Wall	MARLW = 0.657	g

b. 2nd Floor (N/S Direction)

East Wall	MA2LE = 0.86	g
Center Wall	MA2LC = 0.805	g
West Wall	MA2LW = 0.893	g

c. Base Level (N/S Direction)

East Wall	MABLE = 0.789	g
Center Wall	MABLC = 0.683	g
West Wall	MABLW = 0.791	g

Title = "Test 13 - Kobe 1995"

Description = "2 Storey board sheathing w/o stucco and hold-down"

Date_of_Test = "27 July 2001"

Section III

Anchor Rod Loads

The peak values of the measured loads are as follow:

a. 2nd floor rods

NE North Rod	MNENR2 = 0	kN	or	MNENR2I = 0	kips
NE South Rod	MNESR2 = 0	kN	or	MNESR2I = 0	kips
SE North Rod	MSENR2 = 0	kN	or	MSENR2I = 0	kips
SE South Rod	MSESR2 = 0	kN	or	MSESR2I = 0	kips

b. Base level rods

NE North Rod	MNENRB = 0	kN	or	MNENRBI = 0	kips
NE South Rod	MNESRB = 0	kN	or	MNESRBI = 0	kips
SE North Rod	MSENRB = 0	kN	or	MSENRBI = 0	kips
SE South Rod	MSESRB = 0	kN	or	MSESRBI = 0	kips

Note: Any prestressed load in the anchors before the earthquake simulation test was removed by zeroing the beginning of each record. So the loads presented in the following plots should be interpreted as just the dynamic component of the rod loads, and not the total rod loads.

Title = "Test 13 - Kobe 1995"

Description = "2 Storey board sheathing w/o stucco and hold-down"

Date_of_Test = "27 July 2001"

Section IV

Relative Displacements and Drifts

The following relative displacements with respect to the Base Level are considered here:

a) displacement of the 2nd floor, and
b) displacement of the Roof Level.

The drifts considered here are the *drift of the 2nd Floor* with respect to the Base Level and the *drift of the Roof Level* with respect to the 2nd Floor.

The peak values are:

a. Relative Displacements

a.i) Roof vs. Base Level

East Wall	MRBDE = 22.672	cm	or	MRBDEI = 8.926	in
Center Wall	MRBDC = 23.922	cm	or	MRBDCI = 9.418	in
West Wall	MRBDW = 23.166	cm	or	MRBDWI = 9.121	in

a.ii) 2nd Floor vs. Base Level

East Wall	M2BDE = 21.045	cm	or	M2BDEI = 8.286	in
Center Wall	M2BDC = 22.454	cm	or	M2BDCI = 8.84	in
West Wall	M2BDW = 21.638	cm	or	M2BDWI = 8.519	in

b. Drifts

b.i) Roof vs. 2nd Floor

East Wall	MRBDDE = 1.701	cm	or	MRBDDEI = 0.67	in
Center Wall	MRBDDC = 1.543	cm	or	MRBDDCI = 0.607	in
West Wall	MRBDDW = 1.658	cm	or	MRBDDWI = 0.653	in

b.ii) 2nd Floor vs. Base Level

East Wall	M2BDDE = 21.045	cm	or	M2BDDEI = 8.286	in
Center Wall	M2BDDC = 22.454	cm	or	M2BDDCI = 8.84	in
West Wall	M2BDDW = 21.638	cm	or	M2BDDWI = 8.519	in

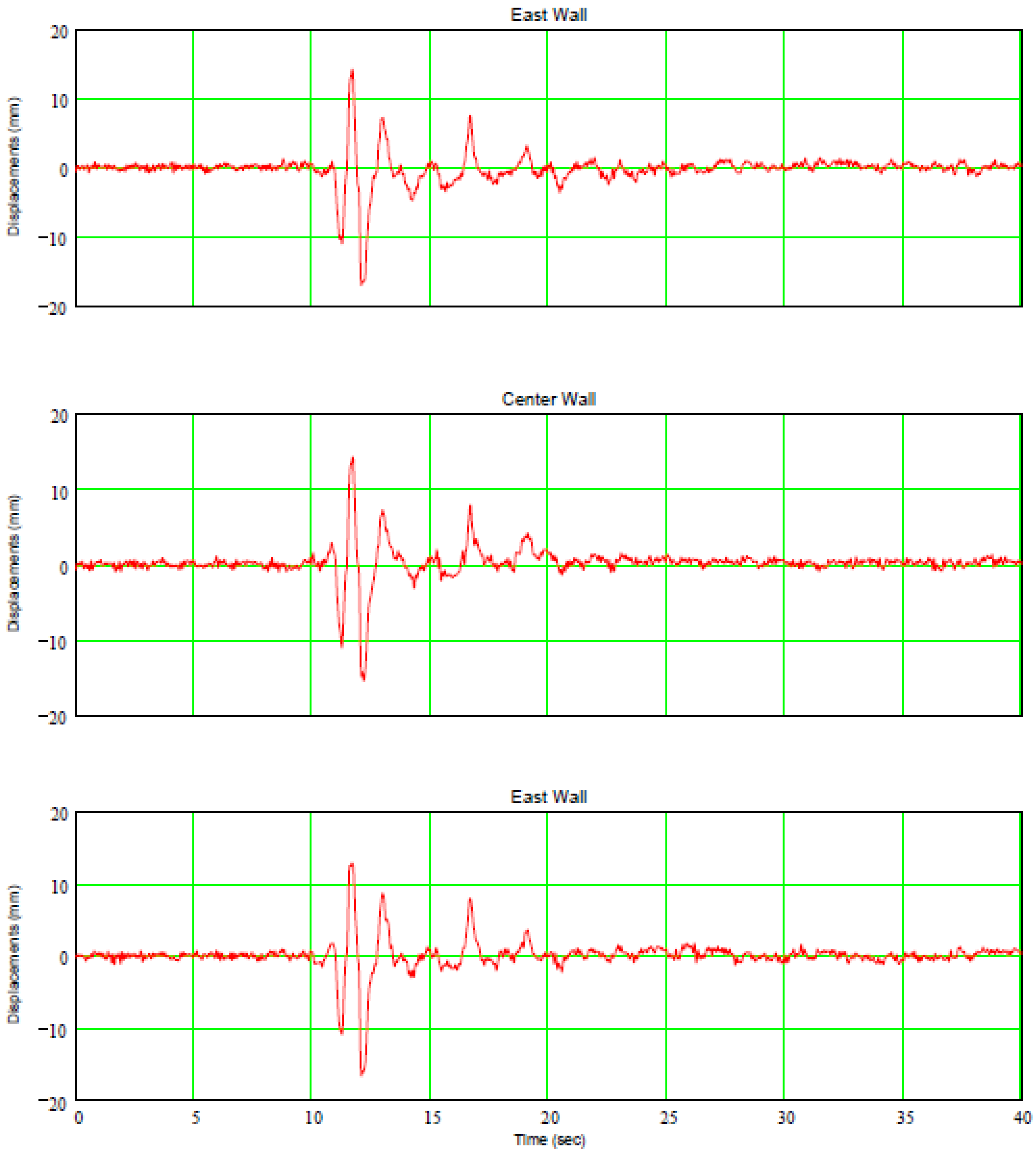
The Time histories of relative displacements and drift are presented in the following pages.

4. Relative Displacements & Drifts.

Title = "Test 13 - Kobe 1995"

Description = "2 Storey board sheathing w/o stucco and hold-down"

Drift - Roof Level vs. 2nd Floor

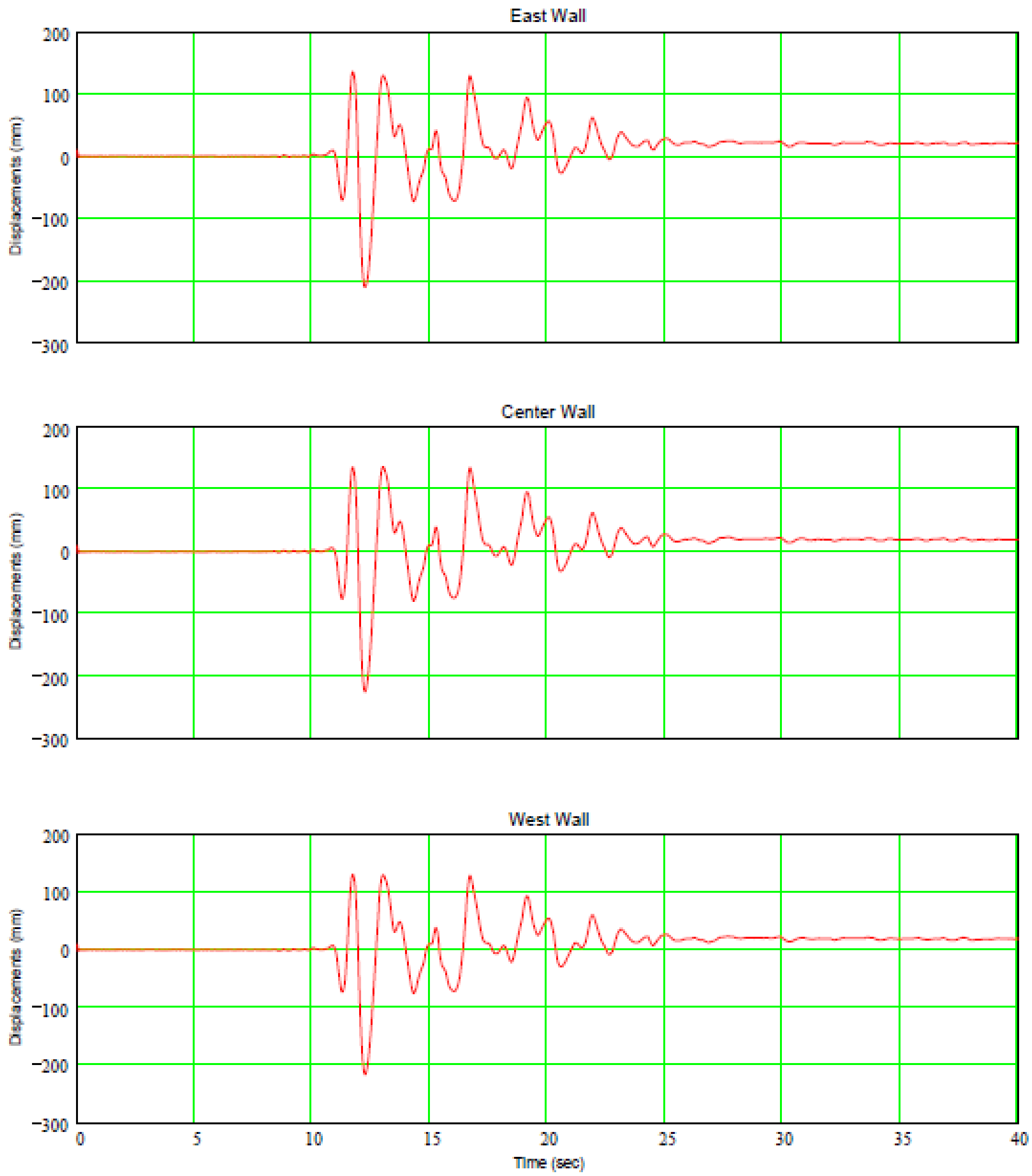


4. Relative Displacements & Drifts.

Title = "Test 13 - Kobe 1995"

Description = "2 Storey board sheathing w/o stucco and hold-down"

Drift - 2nd Floor vs. Base Level



Title = "Test 13 - Kobe 1995"

Description = "2 Storey board sheathing w/o stucco and hold-down"

Date_of_Test = "27 July 2001"

Section V

Load - Deformation Diagrams

The load used for the following diagrams was calculated as the base shear resulting from adding the story shears from each floor. Each story shear was computed as the product of the measured absolute acceleration and the calculated floor mass. The floor weight was calculated to be 9.0 and 8.8 metric tons for the roof and second floor respectively. Since the measured actuator load include the inertia load of the shake table and part of the weight of the first floor it was concluded that a better representation of the base shear could be obtained from the computed story shears rather than by a removing these inertia loads from the measured actuator loads.

Maximum Loads

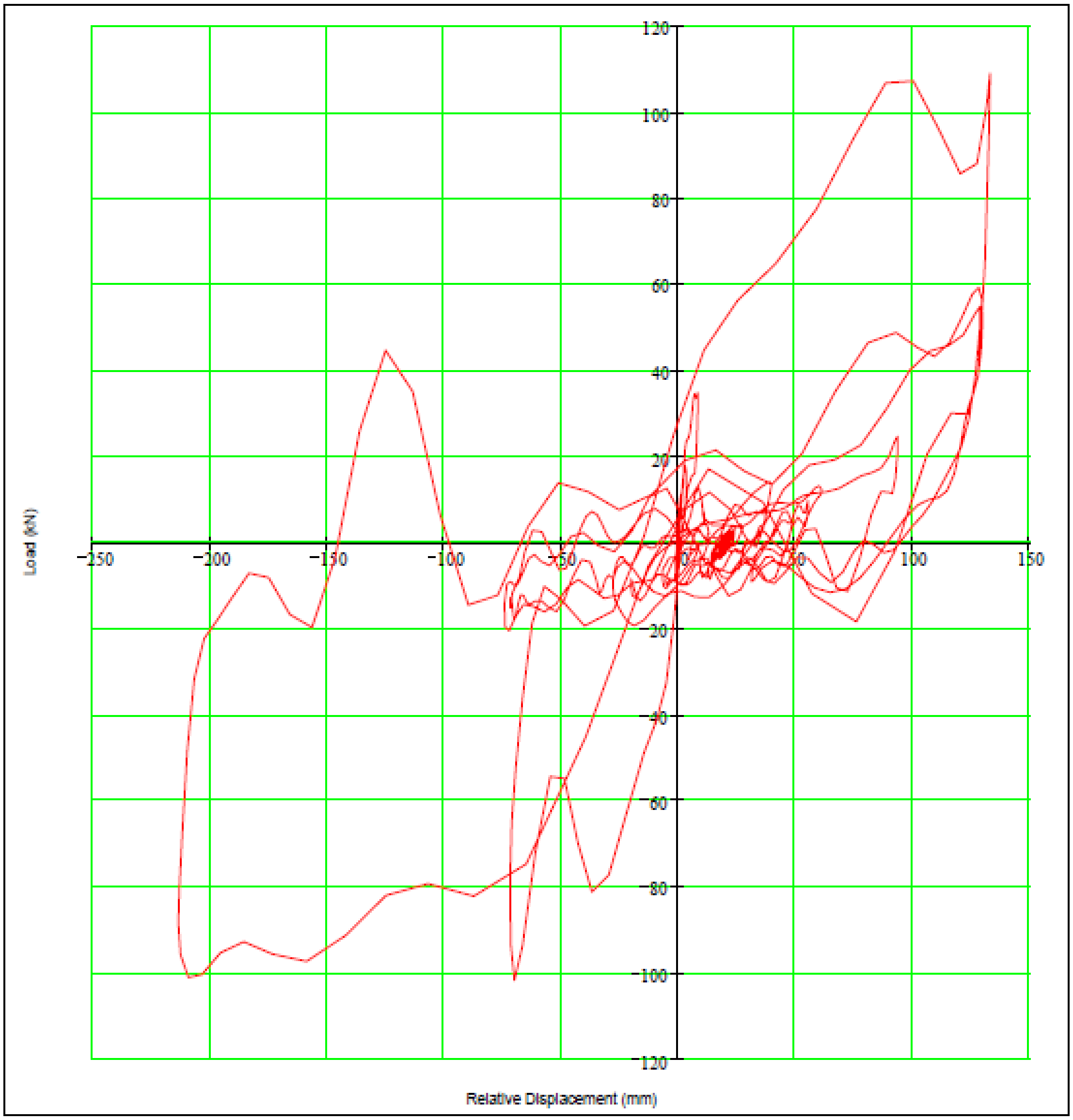
Maximum load for the First Storey (kN): $\max\left(\left|\overrightarrow{CLDF}\right|\right) = 109.353$

5. Load-Deformation Diagrams.

Title = "Test 13 - Kobe 1995"

Description = "2 Storey board sheathing w/o stucco and hold-down"

Load-Deformation, First Storey



Title = "Test 13 - Kobe 1995"

Description = "2 Storey board sheathing w/o stucco and hold-down"

Date_of_Test = "27 July 2001"

Section VI

Calculated Velocity

The velocity in each floor is calculated from the the differential of the displacement. A subroutine to differentiate displacement signals by finite differences was written for this purpose. The resulting the peak velocities for each floor are shown below.

	West		Center		East	
Roof Level	MV3 = 90.531	cm/s	MV2 = 91.534	cm/s	MV1 = 96.385	cm/s
2nd Floor	MV6 = 85.923	cm/s	MV5 = 85.301	cm/s	MV4 = 87.685	cm/s
Basement Level	MV9 = 57.799	cm/s	MV8 = 57.882	cm/s	MV7 = 61.367	cm/s

or

Roof Level	MIV3 = 35.642	in/s	MIV2 = 36.037	in/s	MIV1 = 37.947	in/s
2nd Floor	MIV6 = 33.828	in/s	MIV5 = 33.583	in/s	MIV4 = 34.522	in/s
Basement Level	MIV9 = 22.756	in/s	MIV8 = 22.788	in/s	MIV7 = 24.16	in/s

Reference:C:\Documents and Settings\Mehdi H. K. Kharrazi\Desktop\Earthquake 99 Final Edition\EQ 99 - TWO STOREY (Part I)\Test

Department of Civil Engineering
The University of British Columbia

Summary of Earthquake 99 Project 2-Storey House Shake Table Test

Data Analysis Project Information:

Title = "Test 14 - Landers 1992"

Description = "2 Storey OSB w/o stucco and w/ hold-downs"

Date_of_Test = "3 Aug. 2001"

Test_No = 14

Run_sequence = 1

Directed and Supervised by C.E.Ventura

Analysis done by M.H.K.Kharrazi / C.E.Ventura

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Section **Page**

Title = "Test 14 - Landers 1992"

Description = "2 Storey OSB w/o stucco and w/ hold-downs"

Date_of_Test = "3 Aug. 2001"

Section I

Shake Table Motions

This section presents the recorded shake table motions (base level motion) during this test.

The information presented here includes:

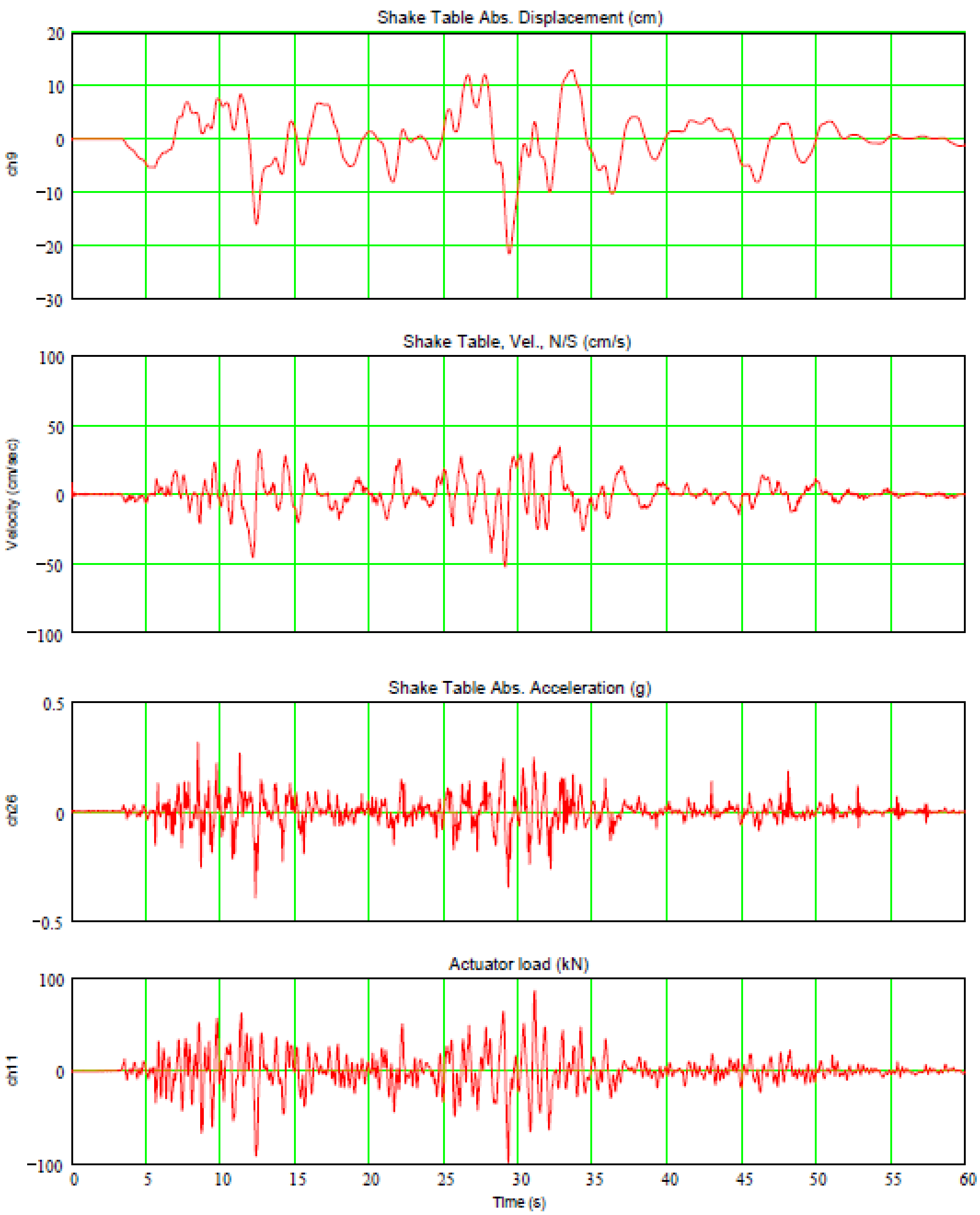
a) shake table actuator displacement (in cm)
b) shake table inferred velocity (in cm/sec)
c) shake table actuator acceleration (in g's)
d) shake table actuator load (in kN)

The recorded peak values (after signal conditioning) of these motions are:

a) Displacement:	Mdisp = 21.6	cm	=	Mdispl = 8.504	in
b) Velocity:	MVel = 52.455	cm/sec	=	MVelI = 20.651	in/sec
c) Acceleration:	Maccl = 0.394	g			
d) Load:	MLoad = 98.892	kN	=	MLoadI = 22.233	kips

1. Shake Table Motions Title = "Test 14 - Landers 1992" Date_of_Test = "3 Aug. 2001"

Description = "2 Storey OSB w/o stucco and w/ hold-downs"



file: Summary T 14 R 1 New.mcd Page 3 of 11 Date processed: 19/04/2002

Title = "Test 14 - Landers 1992"

Description = "2 Storey OSB w/o stucco and w/ hold-downs"

Date_of_Test = "3 Aug. 2001"

Section II

Horizontal Absolute Accelerations

The peak values of absolute acceleration are:

a. Roof Level (N/S direction)

East Wall	MARLE = 0.508	g
Center Wall	MARLC = 0.564	g
West Wall	MARLW = 0.496	g

b. 2nd Floor (N/S Direction)

East Wall	MA2LE = 0.415	g
Center Wall	MA2LC = 0.421	g
West Wall	MA2LW = 0.423	g

c. Base Level (N/S Direction)

East Wall	MABLE = 0.407	g
Center Wall	MABLC = 0.394	g
West Wall	MABLW = 0.415	g

Title = "Test 14 - Landers 1992"

Description = "2 Storey OSB w/o stucco and w/ hold-downs"

Date_of_Test = "3 Aug. 2001"

Section III

Anchor Rod Loads

The peak values of the measured loads are as follow:

a. 2nd floor rods

NE North Rod	MNENR2 = 0	kN	or	MNENR2I = 0	kips
NE South Rod	MNESR2 = 0	kN	or	MNESR2I = 0	kips
SE North Rod	MSENR2 = 0	kN	or	MSENR2I = 0	kips
SE South Rod	MSESR2 = 0	kN	or	MSESR2I = 0	kips

b. Base level rods

NE North Rod	MNENRB = 3.885	kN	or	MNENRBI = 0.874	kips
NE South Rod	MNESRB = 5.24	kN	or	MNESRBI = 1.178	kips
SE North Rod	MSENRB = 7.919	kN	or	MSENRBI = 1.78	kips
SE South Rod	MSESRB = 7.277	kN	or	MSESRBI = 1.636	kips

Note: Any prestressed load in the anchors before the earthquake simulation test was removed by zeroing the beginning of each record. So the loads presented in the following plots should be interpreted as just the dynamic component of the rod loads, and not the total rod loads.

Section IV

Relative Displacements and Drifts

The following relative displacements with respect to the Base Level are considered here:

a) displacement of the 2nd floor, and
b) displacement of the Roof Level.

The drifts considered here are the *drift of the 2nd Floor* with respect to the Base Level and the *drift of the Roof Level* with respect to the 2nd Floor.

The peak values are:

a. Relative Displacements

a.i) Roof vs. Base Level

East Wall	MRBDE = 1.869	cm	or	MRBDEI = 0.736	in
Center Wall	MRBDC = 2.269	cm	or	MRBDCI = 0.893	in
West Wall	MRBDW = 1.976	cm	or	MRBDWI = 0.778	in

a.ii) 2nd Floor vs. Base Level

East Wall	M2BDE = 0.922	cm	or	M2BDEI = 0.363	in
Center Wall	M2BDC = 1.381	cm	or	M2BDCI = 0.544	in
West Wall	M2BDW = 1.089	cm	or	M2BDWI = 0.429	in

b. Drifts

b.i) Roof vs. 2nd Floor

East Wall	MRBDDE = 1.04	cm	or	MRBDDEI = 0.409	in
Center Wall	MRBDDC = 1.108	cm	or	MRBDDCI = 0.436	in
West Wall	MRBDDW = 1.078	cm	or	MRBDDWI = 0.424	in

b.ii) 2nd Floor vs. Base Level

East Wall	M2BDDE = 0.922	cm	or	M2BDDEI = 0.363	in
Center Wall	M2BDDC = 1.381	cm	or	M2BDDCI = 0.544	in
West Wall	M2BDDW = 1.089	cm	or	M2BDDWI = 0.429	in

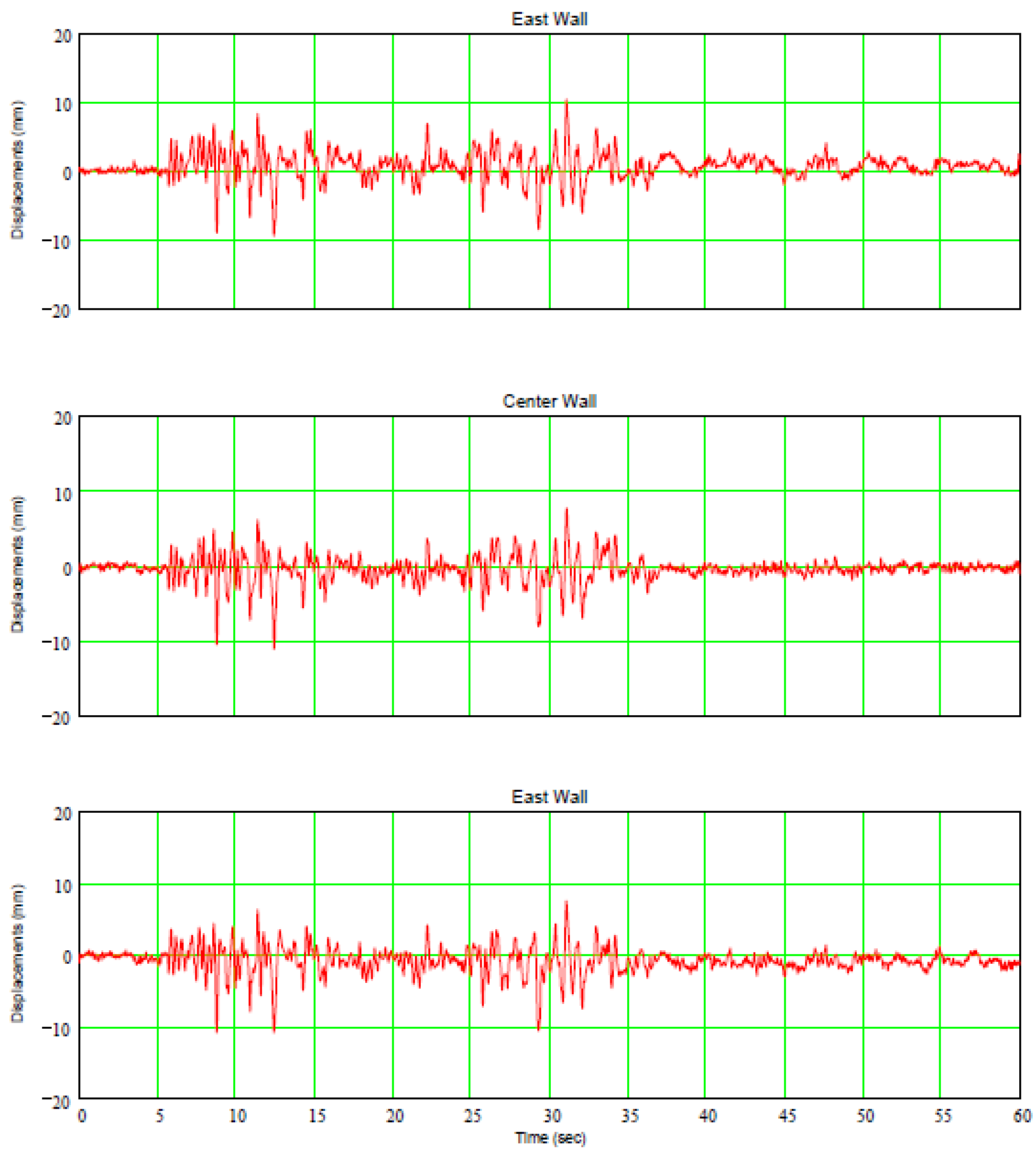
The Time histories of relative displacements and drift are presented in the following pages.

4. Relative Displacements & Drifts.

Title = "Test 14 - Landers 1992"

Description = "2 Storey OSB w/o stucco and w/ hold-downs"

Drift - Roof Level vs. 2nd Floor

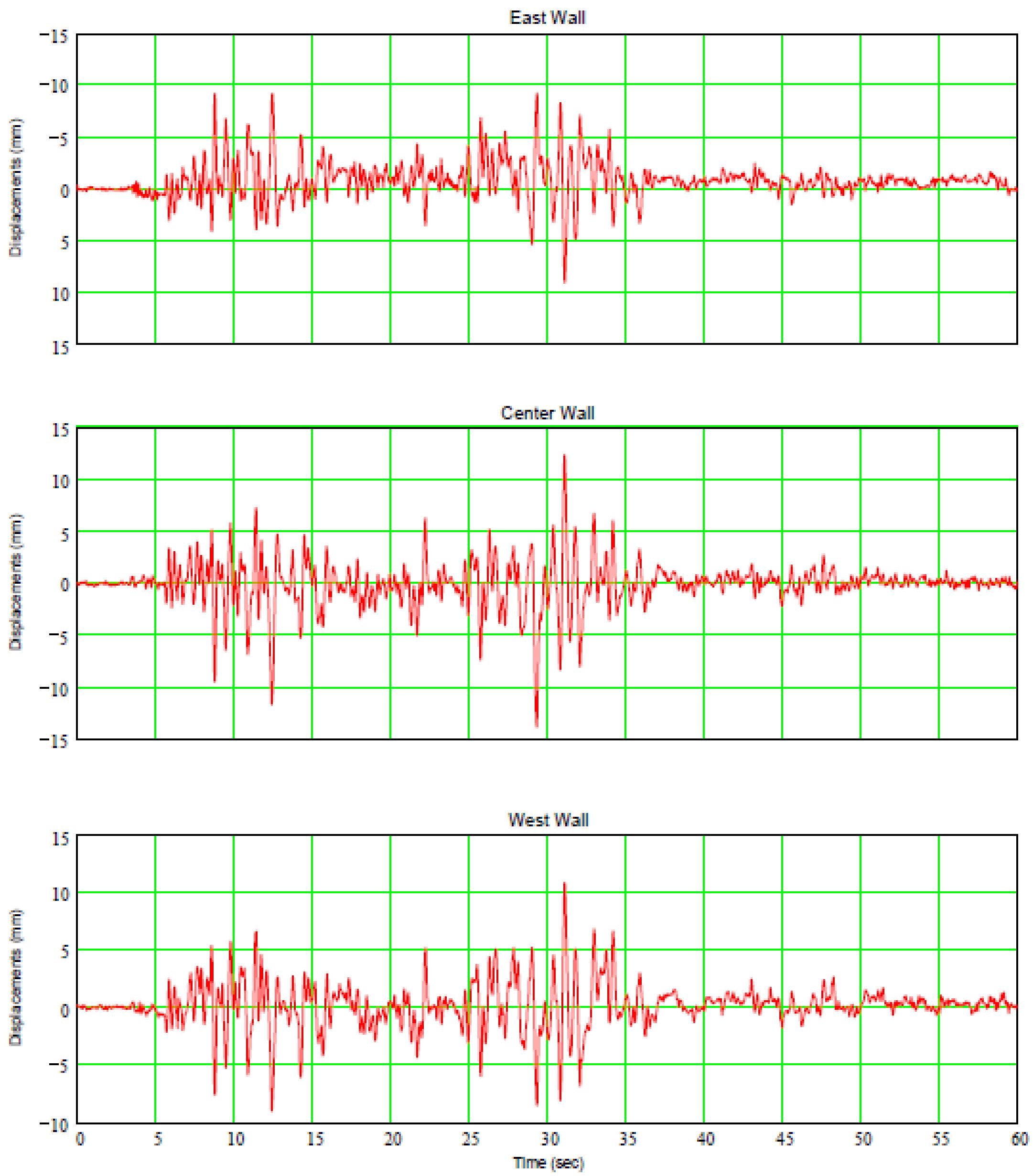


4. Relative Displacements & Drifts.

Title = "Test 14 - Landers 1992"

Description = "2 Storey OSB w/o stucco and w/ hold-downs"

Drift - 2nd Floor vs. Base Level



Title = "Test 14 - Landers 1992"

Description = "2 Storey OSB w/o stucco and w/ hold-downs"

Date_of_Test = "3 Aug. 2001"

Section V

Load - Deformation Diagrams

The load used for the following diagrams was calculated as the base shear resulting from adding the story shears from each floor. Each story shear was computed as the product of the measured absolute acceleration and the calculated floor mass. The floor weight was calculated to be 9.0 and 8.8 metric tons for the roof and second floor respectively. Since the measured actuator load include the inertia load of the shake table and part of the weight of the first floor it was concluded that a better representation of the base shear could be obtained from the computed story shears rather than by a removing these inertia loads from the measured actuator loads.

Maximum Loads

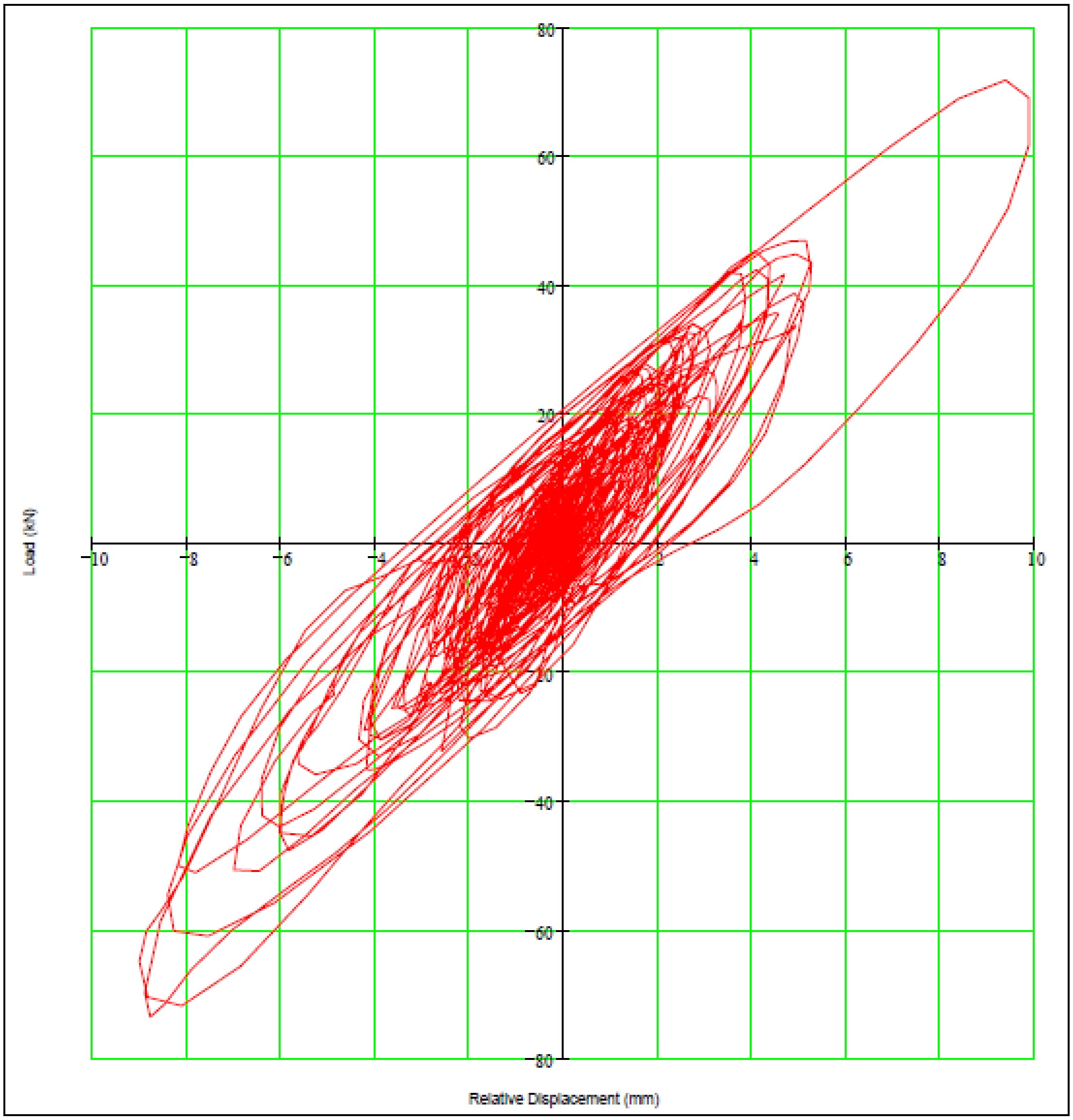
Maximum load for the First Storey (kN): $\max\left(\left|\overrightarrow{CLDF}\right|\right) = 73.501$

5. Load-Deformation Diagrams.

Title = "Test 14 - Landers 1992"

Description = "2 Storey OSB w/o stucco and w/ hold-downs"

Load-Deformation, First Storey



Title = "Test 14 - Landers 1992"

Description = "2 Storey OSB w/o stucco and w/ hold-downs"

Date_of_Test = "3 Aug. 2001"

Section VI

Calculated Velocity

The velocity in each floor is calculated from the the differential of the displacement. A subroutine to differentiate displacement signals by finite differences was written for this purpose. The resulting the peak velocities for each floor are shown below.

	West		Center		East	
Roof Level	MV3 = 65.308	cm/s	MV2 = 63.33	cm/s	MV1 = 63.414	cm/s
2nd Floor	MV6 = 57.945	cm/s	MV5 = 59.16	cm/s	MV4 = 58.245	cm/s
Basement Level	MV9 = 53.286	cm/s	MV8 = 52.455	cm/s	MV7 = 51.001	cm/s

or

Roof Level	MIV3 = 25.712	in/s	MIV2 = 24.933	in/s	MIV1 = 24.966	in/s
2nd Floor	MIV6 = 22.813	in/s	MIV5 = 23.291	in/s	MIV4 = 22.931	in/s
Basement Level	MIV9 = 20.979	in/s	MIV8 = 20.651	in/s	MIV7 = 20.079	in/s

Reference:C:\Documents and Settings\Mehdi H. K. Kharrazi\Desktop\Earthquake 99 Final Edition\EQ 99 - TWO STOREY (Part II)\Tes

Department of Civil Engineering
The University of British Columbia

Summary of Earthquake 99 Project
2-Storey House Shake Table Test

Data Analysis Project Information:

Title = "Test 15 - Llayllay - Chilie 1985"

Description = "2 Storey OSB panel w/o stucco and w/ Hold-down"

Date_of_Test = "29 Aug. 2001"

Test_No = 15

Run_sequence = 1

Directed and Supervised by C.E.Ventura

Analysis done by M.H.K.Kharrazi / C.E.Ventura

Table of Contents

Title = "Test 15 - Llayllay - Chilie 1985"

Description = "2 Storey OSB panel w/o stucco and w/ Hold-down"

Date_of_Test = "29 Aug. 2001"

Section I

Shake Table Motions

This section presents the recorded shake table motions (base level motion) during this test.

The information presented here includes:

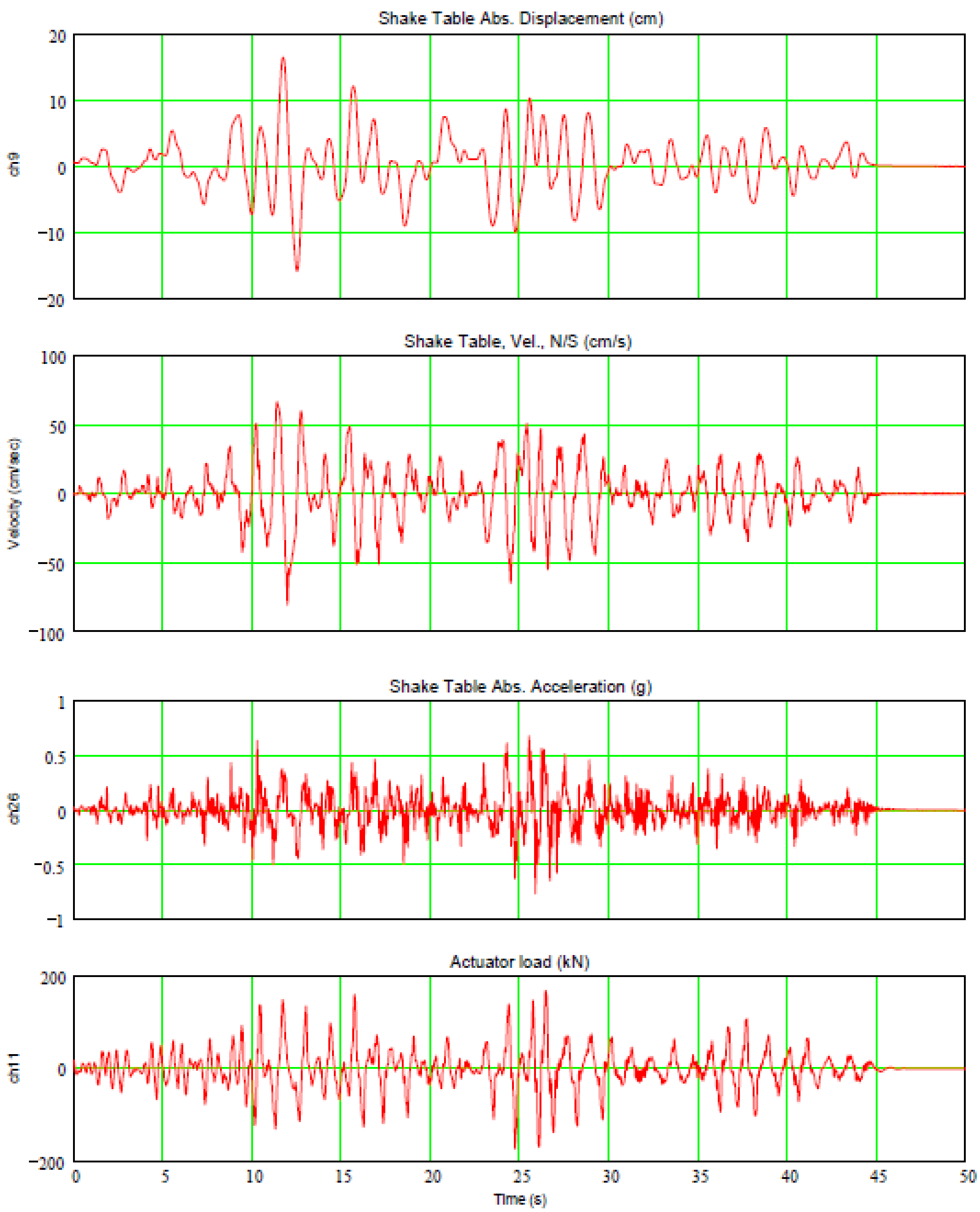
a) shake table actuator displacement (in cm)
b) shake table inferred velocity (in cm/sec)
c) shake table actuator acceleration (in g's)
d) shake table actuator load (in kN)

The recorded peak values (after signal conditioning) of these motions are:

a) Displacement:	Mdisp = 16.388	cm	=	Mdispl = 6.452	in
b) Velocity:	MVel = 80.496	cm/sec	=	MVell = 31.691	in/sec
c) Acceleration:	Maccl = 0.764	g			
d) Load:	MLoad = 174.51	kN	=	MLoadl = 39.233	kips

1. Shake Table Motions Title = "Test 15 - Llayllay - Chilie 1985" Date_of_Test = "29 Aug. 2001"

Description = "2 Storey OSB panel w/o stucco and w/ Hold-down"



Title = "Test 15 - Llayllay - Chilie 1985"

Description = "2 Storey OSB panel w/o stucco and w/ Hold-down"

Date_of_Test = "29 Aug. 2001"

Section II

Horizontal Absolute Accelerations

The peak values of absolute acceleration are:

a. Roof Level (N/S direction)

East Wall	MARLE = 1.129	g
Center Wall	MARLC = 1.254	g
West Wall	MARLW = 1.148	g

b. 2nd Floor (N/S Direction)

East Wall	MA2LE = 0.94	g
Center Wall	MA2LC = 0.982	g
West Wall	MA2LW = 1.166	g

c. Base Level (N/S Direction)

East Wall	MABLE = 0.708	g
Center Wall	MABLC = 0.764	g
West Wall	MABLW = 0.763	g

Title = "Test 15 - Llayllay - Chilie 1985"

Description = "2 Storey OSB panel w/o stucco and w/ Hold-down"

Date_of_Test = "29 Aug. 2001"

Section III

Anchor Rod Loads

The peak values of the measured loads are as follow:

a. 2nd floor rods

NE North Rod	MNENR2 = 0	kN	or	MNENR2I = 0	kips
NE South Rod	MNESR2 = 0	kN	or	MNESR2I = 0	kips
SE North Rod	MSENR2 = 0	kN	or	MSENR2I = 0	kips
SE South Rod	MSESR2 = 0	kN	or	MSESR2I = 0	kips

b. Base level rods

NE North Rod	MNENRB = 34.175	kN	or	MNENRBI = 7.683	kips
NE South Rod	MNESRB = 31.389	kN	or	MNESRBI = 7.057	kips
SE North Rod	MSENRB = 23.576	kN	or	MSENRBI = 5.3	kips
SE South Rod	MSESRB = 38.983	kN	or	MSESRBI = 8.764	kips

Note: Any prestressed load in the anchors before the earthquake simulation test was removed by zeroing the beginning of each record. So the loads presented in the following plots should be interpreted as just the dynamic component of the rod loads, and not the total rod loads.

Title = "Test 15 - Llayllay - Chilie 1985"

Description = "2 Storey OSB panel w/o stucco and w/ Hold-down"

Date_of_Test = "29 Aug. 2001"

Section IV

Relative Displacements and Drifts

The following relative displacements with respect to the Base Level are considered here:

a) displacement of the 2nd floor, and

b) displacement of the Roof Level.

The drifts considered here are the *drift of the 2nd Floor* with respect to the Base Level and the *drift of the Roof Level* with respect to the 2nd Floor.

The peak values are:

a. Relative Displacements

a.i) Roof vs. Base Level					
East Wall	MRBDE = 11.377	cm	or	MRBDEI = 4.479	in
Center Wall	MRBDC = 12.024	cm	or	MRBDCI = 4.734	in
West Wall	MRBDW = 11.381	cm	or	MRBDWI = 4.481	in
a.ii) 2nd Floor vs. Base Level					
East Wall	M2BDE = 8.632	cm	or	M2BDEI = 3.398	in
Center Wall	M2BDC = 9.201	cm	or	M2BDCI = 3.623	in
West Wall	M2BDW = 8.711	cm	or	M2BDWI = 3.429	in

b. Drifts

b.i) Roof vs. 2nd Floor					
East Wall	MRBDDE = 3.16	cm	or	MRBDDEI = 1.244	in
Center Wall	MRBDDC = 3.049	cm	or	MRBDDCI = 1.2	in
West Wall	MRBDDW = 2.952	cm	or	MRBDDWI = 1.162	in
b.ii) 2nd Floor vs. Base Level					
East Wall	M2BDDE = 8.632	cm	or	M2BDDEI = 3.398	in
Center Wall	M2BDDC = 9.201	cm	or	M2BDDCI = 3.623	in
West Wall	M2BDDW = 8.711	cm	or	M2BDDWI = 3.429	in

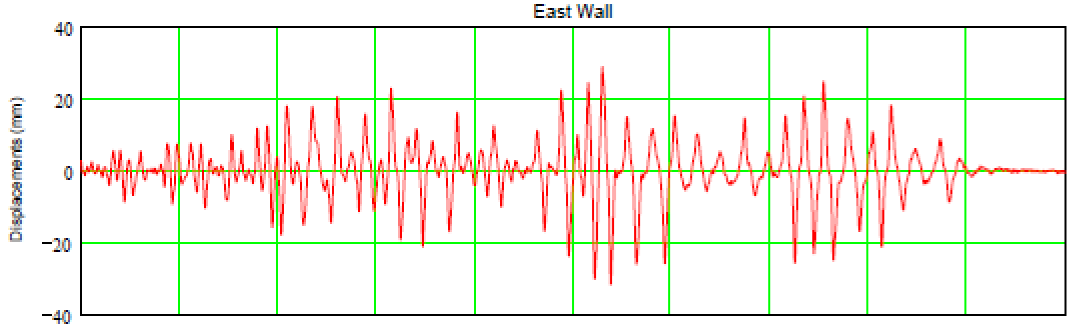
The Time histories of relative displacements and drift are presented in the following pages.

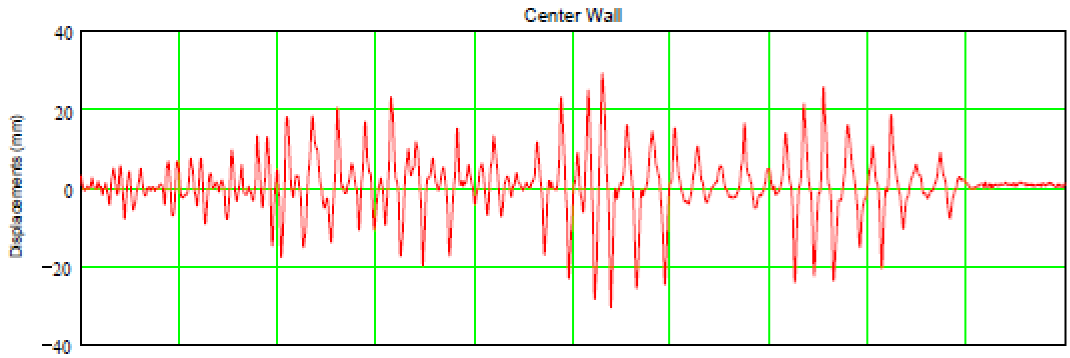
4. Relative Displacements & Drifts.

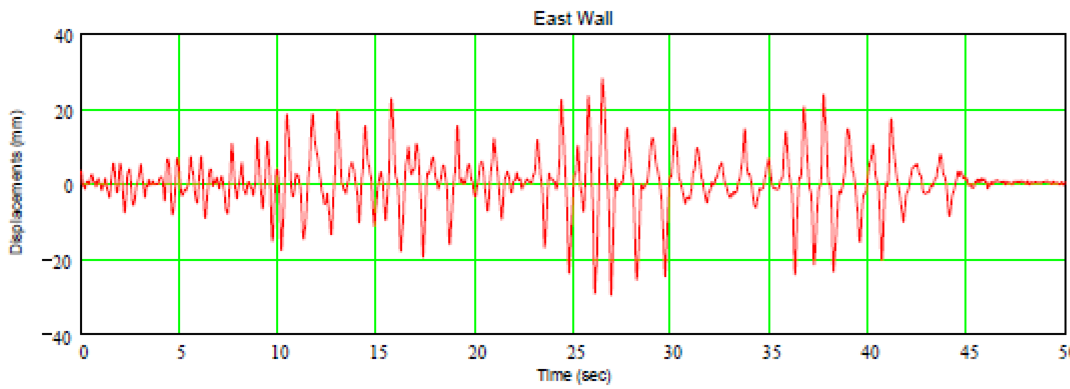
Title – "Test 15 - Llayllay - Chilie 1985"

Description – "2 Storey OSB panel w/o stucco and w/ Hold-down"

Drift - Roof Level vs. 2nd Floor





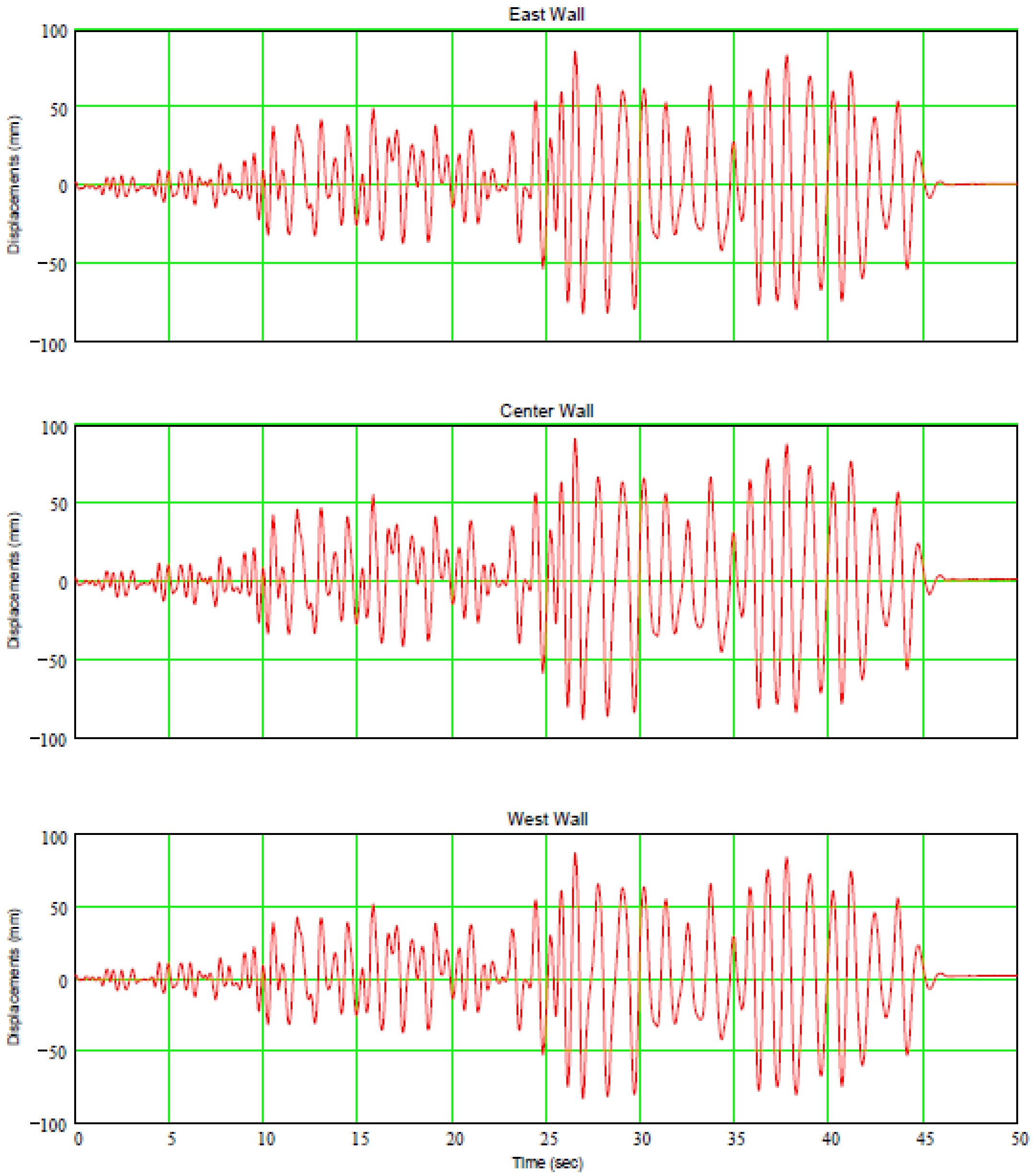


4. Relative Displacements & Drifts.

Title = "Test 15 - Llayllay - Chilie 1985"

Description = "2 Storey OSB panel w/o stucco and w/ Hold-down"

Drift - 2nd Floor vs. Base Level



Title = "Test 15 - Llayllay - Chilie 1985"

Description = "2 Storey OSB panel w/o stucco and w/ Hold-down"

Date_of_Test = "29 Aug. 2001"

Section V

Load - Deformation Diagrams

The load used for the following diagrams was calculated as the base shear resulting from adding the story shears from each floor. Each story shear was computed as the product of the measured absolute acceleration and the calculated floor mass. The floor weight was calculated to be 9.0 and 8.8 metric tons for the roof and second floor respectively. Since the measured actuator load include the inertia load of the shake table and part of the weight of the first floor it was concluded that a better representation of the base shear could be obtained from the computed story shears rather than by a removing these inertia loads from the measured actuator loads.

Maximum Loads

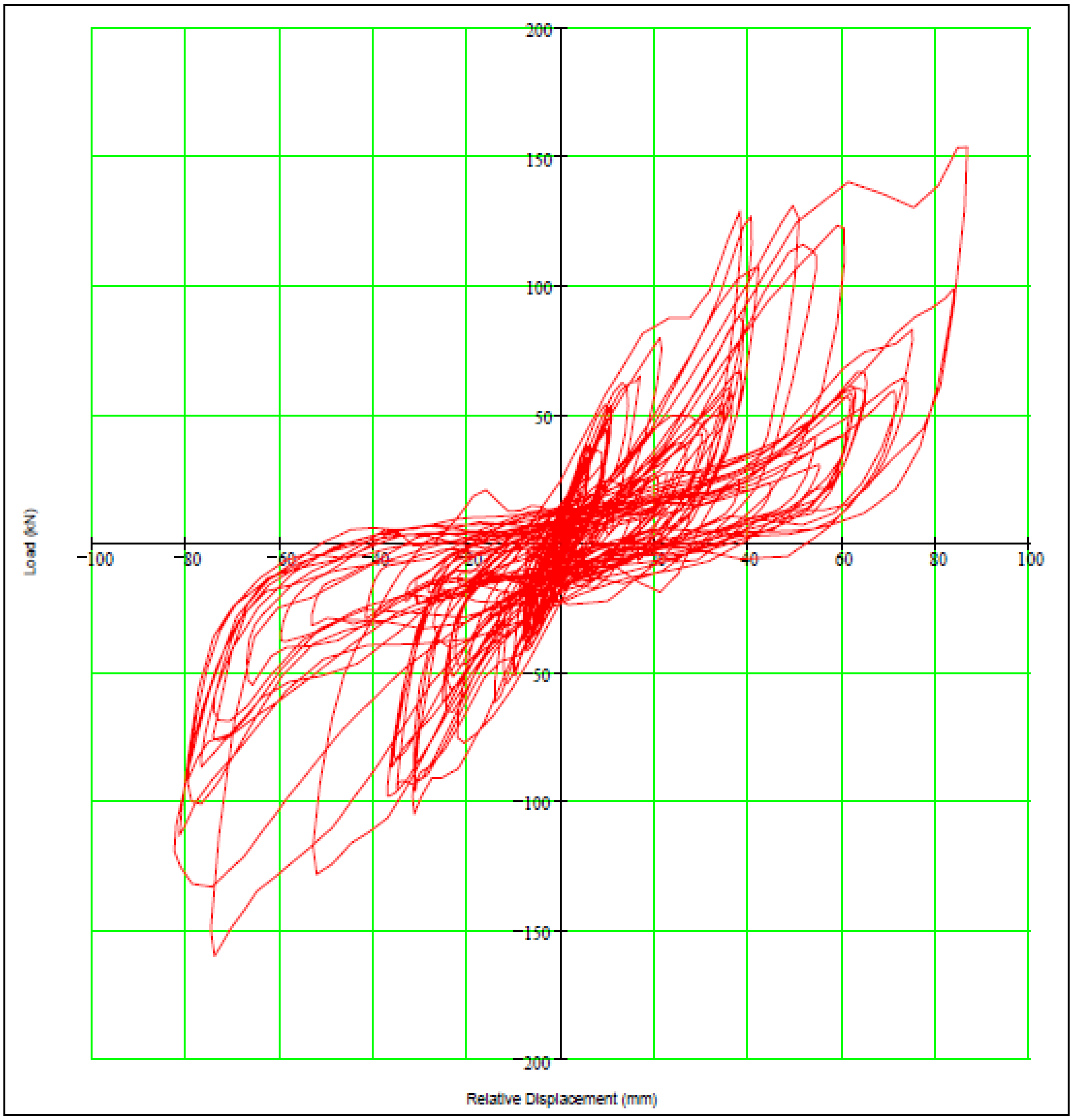
Maximum load for the First Storey (kN): $\max\left(\overrightarrow{|CLDF|}\right) = 159.993$

5. Load-Deformation Diagrams.

Title - "Test 15 - Llayllay - Chilie 1985"

Description - "2 Storey OSB panel w/o stucco and w/ Hold-down"

Load-Deformation, First Storey



Title = "Test 15 - Llayllay - Chilie 1985"

Description = "2 Storey OSB panel w/o stucco and w/ Hold-down"

Date_of_Test = "29 Aug. 2001"

Section VI

Calculated Velocity

The velocity in each floor is calculated from the the differential of the displacement. A subroutine to differentiate displacement signals by finite differences was written for this purpose. The resulting the peak velocities for each floor are shown below.

	West		Center		East	
Roof Level	MV3 = 98.686	cm/s	MV2 = 99.692	cm/s	MV1 = 99.81	cm/s
2nd Floor	MV6 = 87.642	cm/s	MV5 = 88.7	cm/s	MV4 = 87.77	cm/s
Basement Level	MV9 = 77.486	cm/s	MV8 = 80.496	cm/s	MV7 = 77.552	cm/s

or

Roof Level	MIV3 = 38.853	in/s	MIV2 = 39.249	in/s	MIV1 = 39.295	in/s
2nd Floor	MIV6 = 34.505	in/s	MIV5 = 34.921	in/s	MIV4 = 34.555	in/s
Basement Level	MIV9 = 30.506	in/s	MIV8 = 31.691	in/s	MIV7 = 30.532	in/s

Appendix D Combined Sheathing

The following report is a literature review on non-structural walls in experimental testing and numerical modeling. It outlines the superposition method, the FEMA-P807 guidelines recommendations and the 'Toolbox Method' as part of the School Retrofit Project Guidelines.

Non-structural Walls

Interior and exterior non-structural finishes, such as gypsum wall board, plaster on lathe and stucco, have been have been found to substantially contribute to the strength and stiffness in wood-frame buildings and alter the lateral behavior of the structure (Filiatrault, Christovasilis, Wanitkorkul, & van de Lindt, 2010; Filiatrault, Fischer, Folz, & Uang, 2002). Shear wall assemblies may consist of multiple layered materials with significant differences in hysteretic behaviour and ductility. The additional non-structural sheathing has been observed to alter the failure mechanisms of the wood shear walls. The monotonic and cyclic backbone curves of wood shear walls tests with and without non-structural sheathing indicate the behavior cannot be captured by simply taking the sum of the two material backbone curves (Ceccotti & Karacabeyli, 2000; Gatto & Uang, 2002; Pardoen, Walman, Kazanjy, Freund, & Hamilton, 2003). Rose and Keith (1997) found that gypsum contributed to the shear stiffness and strength at small displacements prior to the maximum shear strength of the assembly. The additional non-structural sheathing decreased the yield and ultimate drift when compared to the walls with only OSB sheathing in cyclic wall test by Toothman et al. (2003). The ductility for wood shear walls with non-structural sheathing and without were observed to remain constant (Toothman, 2003; Chen, Nott, Chui, Doudak, & Ni, 2014). The ultimate strength and initial stiffness for the combined material was consistently less than the direct sum of the two separate material properties (Toothman, 2003).

There does not seem to be much agreement in the academic community regarding how to simulate the composite effects of non-structural finishing materials that are prevalent in light-frame wood construction.

Engineers traditionally ignore the contribution of the non-structural sheathing and interior gypsum walls with the assumption that it is conservative. For first-story retrofits, as described in the FEMA P-807 guidelines, ignoring the contribution of the non-structural materials this assumption is not necessarily true; the base floor may be over-strengthened and drive damage to the upper floors (FEMA, 2012). The superposition technique, where the hysteretic spring for the wall assembly are taken as the additive of the ultimate strength and stiffness of the materials, gives acceptable performance predictions when compared to the full scale experimental tests. For instance, Kim and Rosowsky (2005) modeled wood-frame shear walls with gypsum wall board in SAWS. The two-storey building tested by Filiatrault et al. (2010) was modeled by van de Lindt et al. (2010) for the structure at three different building phases: structural wood walls installed; GWB interior sheathing installed; and finally following the installation of the stucco exterior finish. The numerical model using the superposition method gives acceptable predictions when compared to the experimental results; there were however more discrepancies for the models with stucco finish (Bahmani P. , 2015).

The FEMA P-807 document proposes a methodology to superimpose the backbone curves for the various sheathing materials. The document categorizes the sheathing into high and low displacement categories and proposed that the sum of the maximum of 100% of the wood sheathing backbone (high ductility material) and 50% of the other sheathing material(s) backbone (low ductility material i.e. gypsum, stucco) *or* 100% of the other sheathing material(s) backbone and 50% of the structural wood sheathing backbone is assumed. This rule was determined by compiling a number of cyclic and monotonic shear wall tests with different sheathing configurations. The plot, as shown in Figure D.1, shows the backbone curves of separate stucco, gypsum board, and OSB shear wall tests, as well as the combined wall test compared to the proposed combination rules. The strength and energy dissipation is overestimated by simply adding the strength of the three materials; the 100%/50% rules give a reasonable prediction of the backbone response when compared to the combined wall test. The FEMA P-807 documents limit the backbone drift to 5%, where the materials are assumed to have zero residual strength at higher drift levels. Recent studies suggest that light-frame wood construction have significant residual strength and collapse occurs close to drift ratios between 7-11% (Pei S. , van de Lindt, Wehbe, & Liu, 2013) for single shear walls and up to 11-16% drift for full scale structures (Pang, Ziaei, & Filiatrault, 2012). Thus, the FEMA P-807 guidelines may be overly conservative.

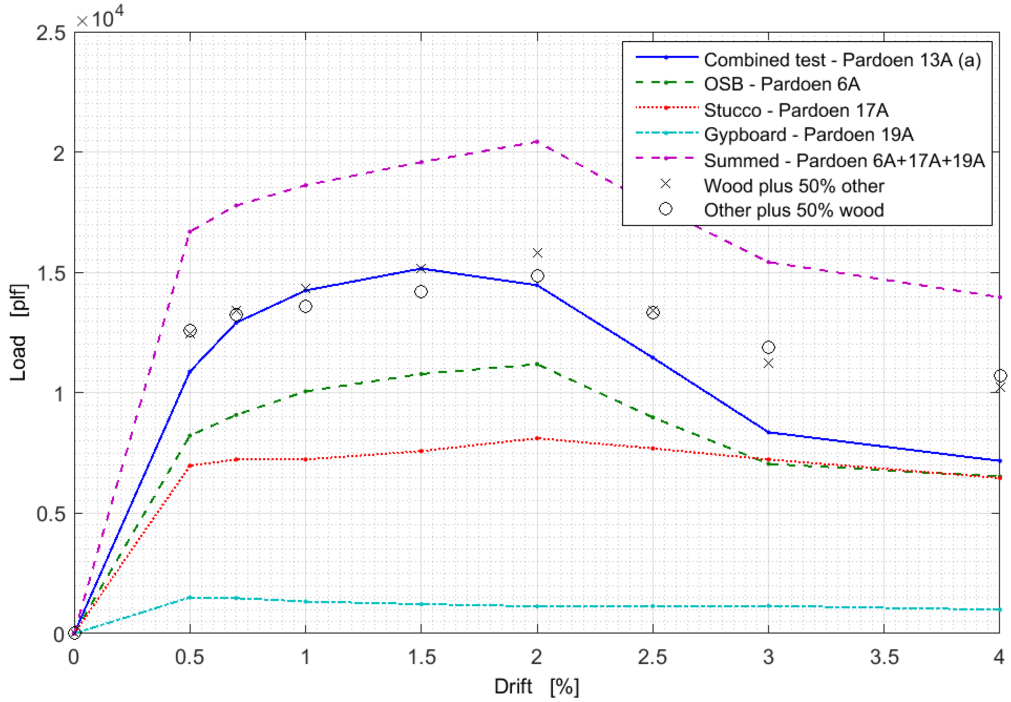


Figure D.1: Separate and combined wall tests for OSB, Stucco and Gypsum Boards compared to 100%/50% rule proposed in the FEMA P-807 Guidelines (FEMA, 2012)

Bahmani (2015) investigated the numerical combination of the sheathing materials with an experimental study of 18 wood-frame shear walls with one, two, or three conventional finishes. The shear walls were tested with the CUREE-Caltech cyclic protocol. Anchor bolts and standard hold down devices were used to transfer the shear to the steel base, to ensure that the walls performed in racking, as well as eliminate the risk of end-post or sill-plate splitting failure modes.

The single wall backbone curves in the tests were compared to the material backbone curves that were recommended in the FEMA P-807 documents. It was observed that the backbones from the experimental data were similar for the stucco and wood structural panel (8d @102mm (4") o.c.). There were significant discrepancies between the FEMA P-807 suggested backbone curves and the experimental backbone curve for horizontal board wall systems. The horizontal board walls were capable of significant deformation before collapse. This behavior was also observed in the laboratory tests as part of the Innovative Retrofit Testing Program at UBC (EERF, 2009) where the walls deformed over 8% drift without collapse. The gypsum wall board backbone properties in the FEMA P-807 documents are based on gypsum walls with a 178mm (7") fastener spacing, the experimental testing by Bahmani used 406mm (16") fastener spacing and the EERF documents tested gypsum wall specimens with 203mm (8") fastening spacing. As shown in Figure D.2 the FEMA P-807 gypsum wall backbone curves have significantly more strength than the experimental tests by Bahmani (2015) and EERF (2009). For purposes of assessment of existing buildings, the FEMA P-807 guidelines may overestimate the strength of the material in place.

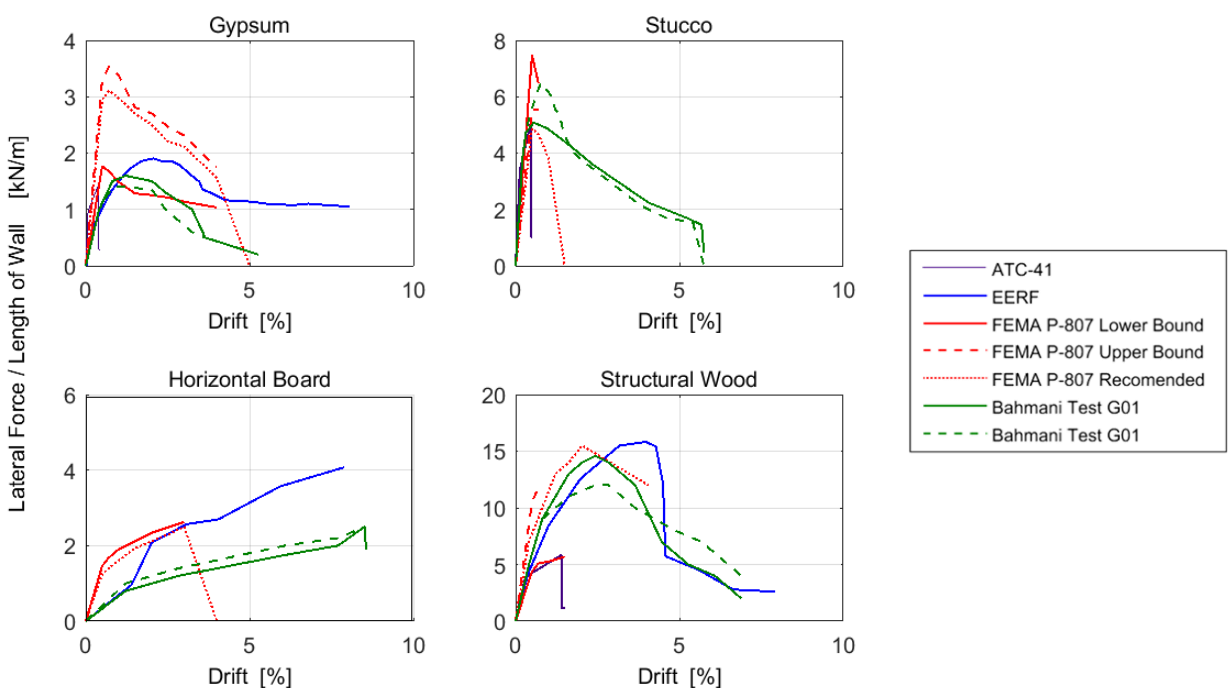


Figure D.2: Comparison of backbones from the experimental tests by Bahmani and EERF and FEMA P-807 and ATC-41 data for (i) gypsum, (ii) stucco, (iii) horizontal board and (iv) structural wood

Bahmani (2015) compared the multiple sheathing tests with the numerical combinations methods. The backbone curves of the individual sheathing test were superimposed with 100% of the strength values and were compared to the combined wall tests: horizontal board & gypsum; stucco & structural wood; horizontal board, structural wood & gypsum; and stucco, structural wood & gypsum wall systems. The peak capacity was observed in general to be higher for the combined wall test than the superimposed individual tests and occurred at the same lateral displacement. The initial (elastic) stiffness of the combined test was lower than the superimposed individual tests. The decay rate post peak was observed to be higher for the combined test; this indicates the superimposed system overestimated the restoring forces post-peak comparted to the combined wall system.

Non-linear time history (NLTH) analysis (FEMA, 2009) was conducted to further investigate the dynamic properties of the wall systems. Each wall system was modeled with a single-degree-of-freedom (SDOF) spring using the EPHM material model that were matched to the wall test hysteretic responses. The superimposed individual tests experienced slightly lower drift ratios than the combined test specimens (with the expectation of the HWS, WSP and GWB wall combination). This suggests that the superposition method may be slightly un-conservative.

The backbone curves of the combined wall tests and the superimposed walls using the combination rule by FEMA P-807 was also compared. The superimposed backbones following the FEMA P-807 rule consistently underestimated the ultimate strength; the difference in the peak forces were 31%, 23% and 20% for the stucco/structural wood test, the horizontal board/structural wood/gypsum test and stucco/structural wood/gypsum test, respectively. A NLTH analysis was conducted and the FEMA P-807 combinations resulted in larger lateral displacement than the combined wall test models. It was concluded that the proposed rule in FEMA-P807 leads to a conservative design that is within an acceptable range.

The Seismic Retrofit Guidelines applies the 'Toolbox Method' to combine the contribution of different systems for either risk assessment of a building or refining the retrofit design. It is important to note that the guidelines are applied to many different types of construction including concrete, steel, masonry and wood, and therefore are very general in nature. Many of the schools have multiple building blocks built at different time periods with varying construction materials, and practices.

Blocked OSB, Unblocked OSB, Gypsum and Shiplap are the four main timber prototypes in the guidelines. Single-degree of freedom pinching models for the prototypes were defined with backbones and hysteretic rules based on the results from the Innovative Retrofit Testing Program and the Earthquake 99 (EQ-99) Project at UBC (EERF, 2009), as well as the CUREE-Caltech Wood frame projects. The prototypes were analyzed separately with incremental non-linear dynamic analyses performed in CANNY for crustal, subcrustal and subduction hazards in Victoria and Vancouver, British Columbia with a range of resistances as a percentage of the seismic weight. The analytical results were post-processed to show the relationship between the probability of drift exceedance (PDE) for a given design drift limit of the structural material and the required factored resistance.

A simplified approach to determine the contribution of each component within the structure is then applied. The governing design drift is defined as the minimum of the design drifts for the components in the system, as shown in the schematic in Figure D.3. The contribution of resistance for each component as a percentage of the seismic weight is determined from the PDE vs. R_m relationship for the prototype at the governing drift level. The component can generate resistance up to the governing drift limit; the drift of the entire system limited by its most brittle component). The engineer can choose to ignore the strength contribution of certain brittle components to allow the structure to experience higher drift levels. The structure is deemed to be deficient if:

$$\sum \left(\frac{Capacity}{Demand}\right) \geq 1.0$$

where the capacity of each component is calculated using unfactored code equations and engineering judgment and the demand is calculated as the product of the required resistance (R_m) for the component and the seismic weight of the entire system. This method is believed to be conservative, however has not been extensively investigated for light-frame wood structures where sheathing layers and non-structural walls can significantly alter the dynamic behavior of the structure.

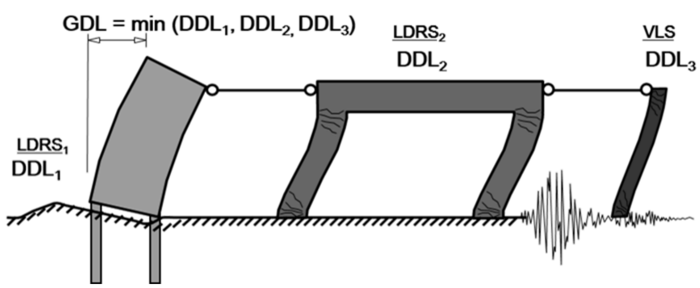


Figure D.3: Governing drift limit for system for the 'Toolbox Method'

Appendix E Drawing of Full-scale Classroom

The following are drawings of the full-scale classroom provided by TBG Seismic Consultant Ltd. These drawings include elevation view and wall framing of the test specimen.

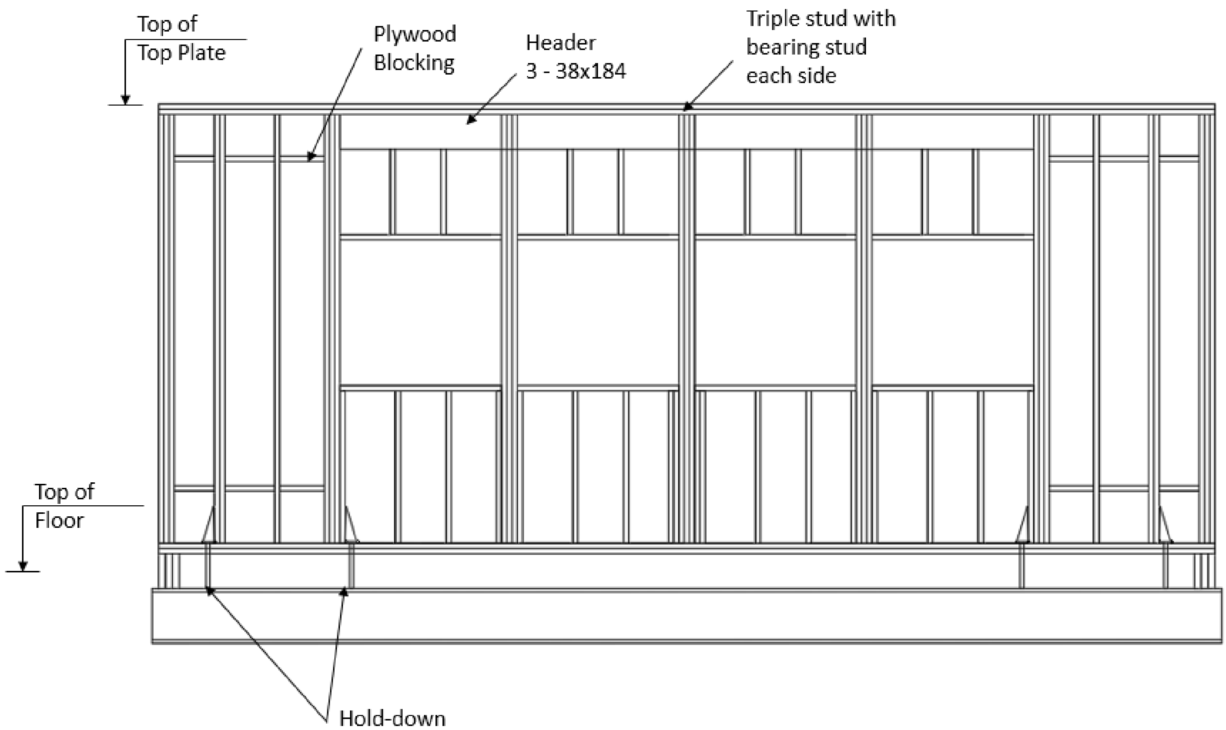


Figure E.1: Elevation – Exterior Wall Framing

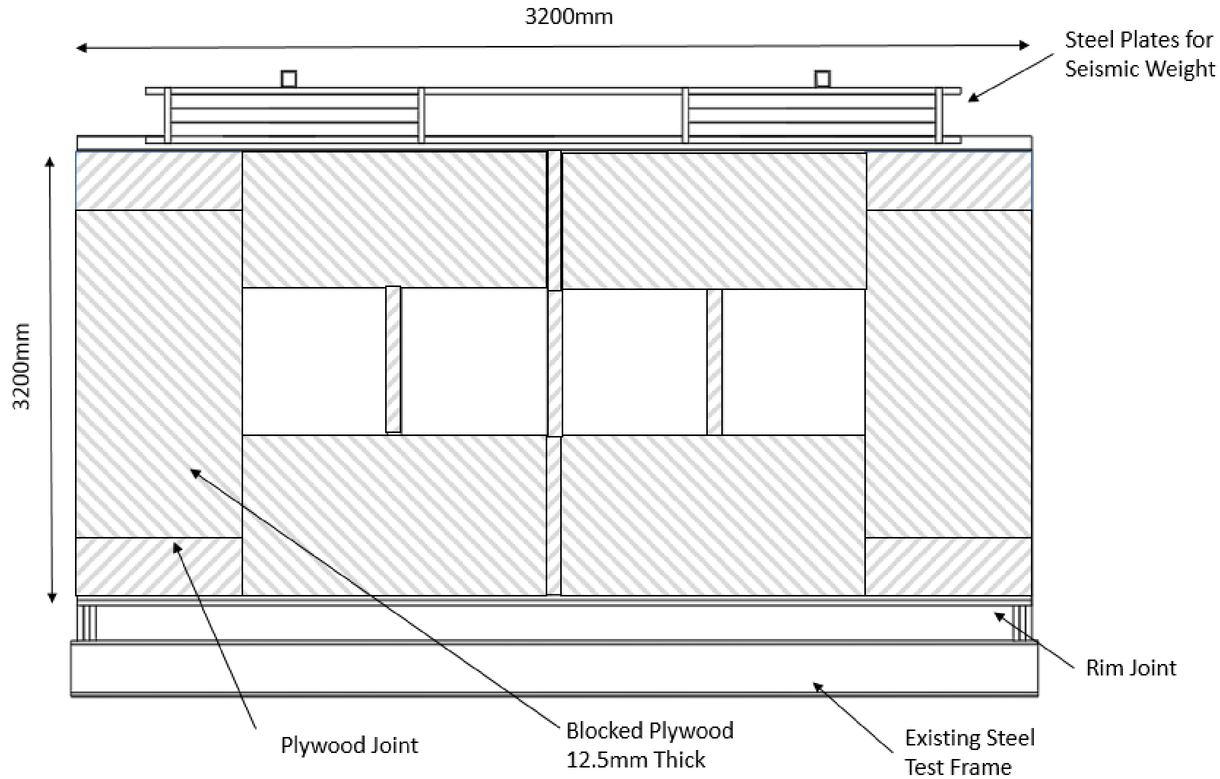
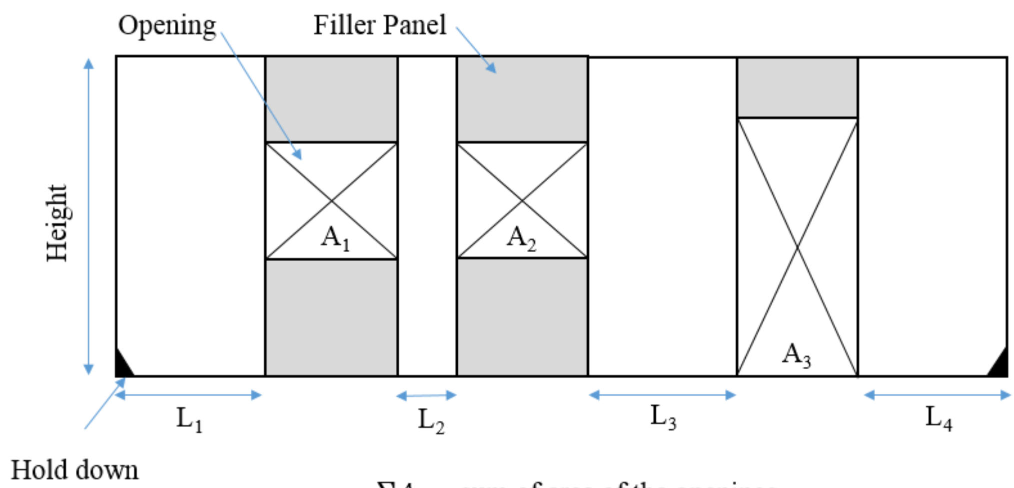


Figure E.2: Elevation – Test Structure Exterior Wall

Appendix F Opening Factor

The FEMA P-807 document recommends using the opening summarized in Figure E.1. This is based on work recommendations from the SDPWS (Seismic Design Provisions for Wind and Seismic) that is confirmed with experimental test results by Dolan and Heine (1997).



$\sum A_i$ = sum of area of the openings
$\sum L_i$ = sum of length of full-height wall segments
H = floor to ceiling wall height

$$\text{Opening Factor} = 0.92\alpha - 0.72\alpha^2 + 0.80\alpha^3$$

$$\alpha = \frac{1}{1 + \frac{\sum A_i}{H \sum L_i}}$$

Figure F.1: Schematic of Opening Factor

A typical wood frame garage wall with an equivalent seismic weight to a second story was tested and modeled numerically with M-CASHEW2 by van de Lindt et al. (2012b). The objective of the work was to study the dynamic behavior of a light-frame wood garage wall at collapse drift levels and to simulate the wall behavior in a numerical model up to full collapse.

The test specimen was 4.52m in length and 2.45m in height with a vehicles opening of 3.3m by 2.064m. The framing members were 2 x 6 Hem-Fir with 16d sinker nails. The sheathing was 12mm (15/32in.) thick

OSB with 8d common gundriven nails spaced at 152mm (6 in.) and 304mm (12 in.) along the panel edges and on the interior, respectively. The wall had a tributary seismic weight of 18.2kN (41 kips).

The ultimate resistance was calculated i) in accordance to the Canadian Wood Design Code (factored and unfactored), ii) using the recommended factors for openings as in accordance to the FEMA P-807 guidelines and iii) from the detailed M-CASHEW2 Model is summarized in Table---.

Without Openings		**With Openings**	
Factored Code Resistance	**Unfactored Code Resistance**	**FEMA P-807 Opening Factor**	**M-CASHEW2 Model**
5.0kN	8.5kN	11.5kN	18.1kN
28%W	47%W	63%W	99%W

Appendix G Additional Analysis for Full-scale Classroom Testing Program

The following report is for the second configuration for the full-scale classroom test as part of the Seismic Retrofit project.

Test Specimen

As part of the Seismic Retrofit project, a full-scale one-storey wood frame classroom was tested on the linear shake table at UBC EERF facility. The classroom had a plan dimension of 7.62m x 6.096m (300”x200”). The sheathing nails on the blocked shear wall segment were 8d common nails spaced at 100mm (4”) on the sheathing panel edges and 150mm (6”) on the interior studs. The unblocked wall sheathing nails were 8d common nails spaced at 12in. on the sheathing panel edges and 24in. on the interior studs. The studs were 2x4 Douglas Fir Lumber and the sheathing was 11mm (7/16 in.) plywood panels. Six (6) steel inertia plates (3600 kg each plate) and HSS sections were loaded on the specimen to simulate a second school storey. The total seismic weight was 250kN (56kips). A schematic of the north and south elevation is shown in Figure 20. An image of the structure is shown in Figure 21.

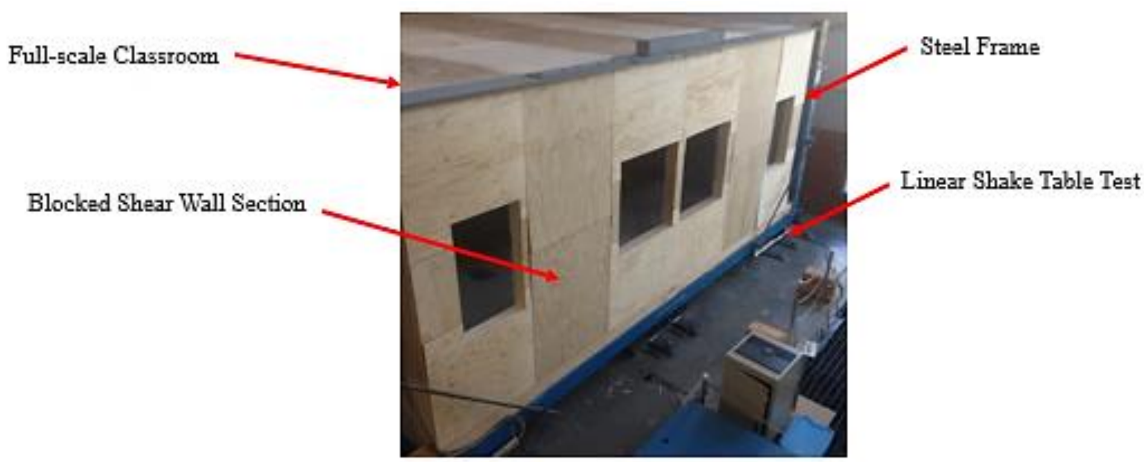


Figure G.1: Test Structure for Second Testing Configuration

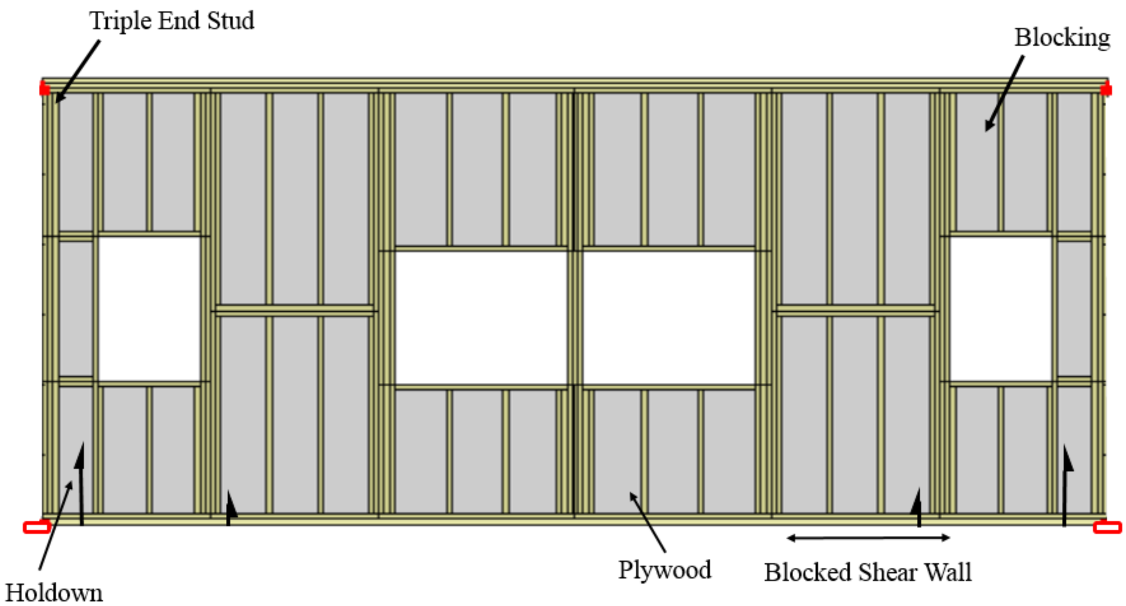


Figure G.2: M-CASHEW2 Model of Classroom North and South Elevation for Second Testing Configuration

Numerical Model

The prediction for the wall behavior was completed in two parts: (1) a detailed M-CASHEW2 model, (2) a global Timber 3D model.

Detailed M-CASHEW2 Model

The M-CASHEW2 model, developed by Pang and Hassenzadeh (2010), is a 2D shear wall and diaphragm modeling program. The frame elements have four translational and two rotational degrees of freedom (DOF). The sheathing panels are modeled with one rotational DOF, two translational DOFs and two shear DOFs. The bending and axial elongation of the framing members, separation and bearing contacts between framing members, uplift and anchorage of the hold down devices, shear deformation of the sheathing panels, nonlinear shear slip response of the sheathing nails, and second order effect of gravity loads (P-delta) can be captured.

Several connection types are defined in a database available in the M-CASHEW2 program and have been used for the classroom wall model. The sheathing nails between the framing and the plywood were modelled with the EPHM material model fitted to the connection test data by Ekiert and Hong (2006) for nominal 51mm (2 in.) thick Hem-Fir attached to 11.1 (7/16 in.) thick OSB using 8d common nails. This data was available and the difference in the sheathing type was felt to not significantly effect the response. The EPHM model was developed to capture the behaviour of light-frame wood shear walls at high drift levels where stiffness and strength degradation is significant. In-cyclic and cyclic deterioration of strength and stiffness is included in the model, which according to Ibarra et al. (2005) and Chandramohan et al. (Chandramohan, Baker, & Deierlein, in press) makes the model suitable for studying the influence of duration of ground motion on collapse.

The gypsum sheathing and framing connections are modeled with the MSTEW material model based on cyclic tests by Dinehart et al. (2008) of No. 6 gypsum screws and 12mm (1/2 in.) thick gypsum wall board. The frame-to-frame shear slip for the double stud nails are modeled elastically. The end nail connections between the end posts and sill plates were modelled with a non-linear hold-down spring to describe the uplift response and nail withdrawal, a well as a M-STEW model to described the shear-slip response of two 10d sinker nails. A non-linear contact element is used to describe the bearing deformation between the framing elements. The hold-down elements were modelled with non-linear hold-down springs based on the component testing by United Steel Products (UPS) hold-downs and matched by van de Lindt et al. (2012b). The details of the components of the M-CASHEW2 model and the hysteretic models used are shown in the following figures.

It should be noted that the elements were tested using the CUREE protocol (Hassanzadehshiraz, 2012). This protocol has been recognised to be realistic for simulating earthquake loading effects for light-frame wood construction. This protocol better captures the effect of crustal ground motions, further investigation of the effect on behaviour of the elements with longer protocols with multiple pulses should be completed to have a better representation of the element behavior in a long duration seismic event.

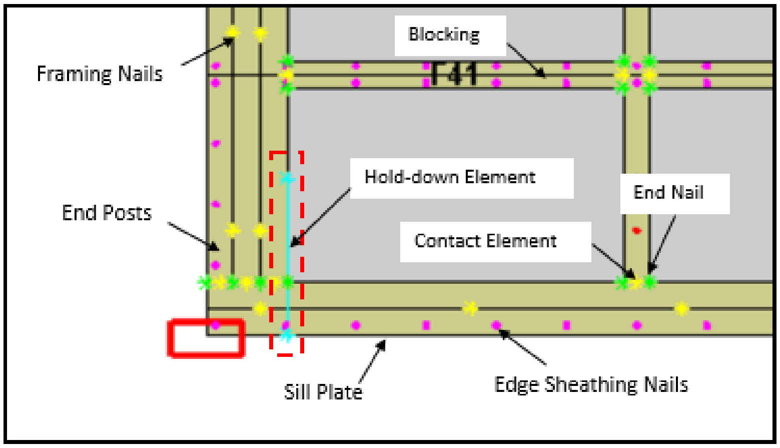


Figure G.3: Details of M-CASHEW2 model

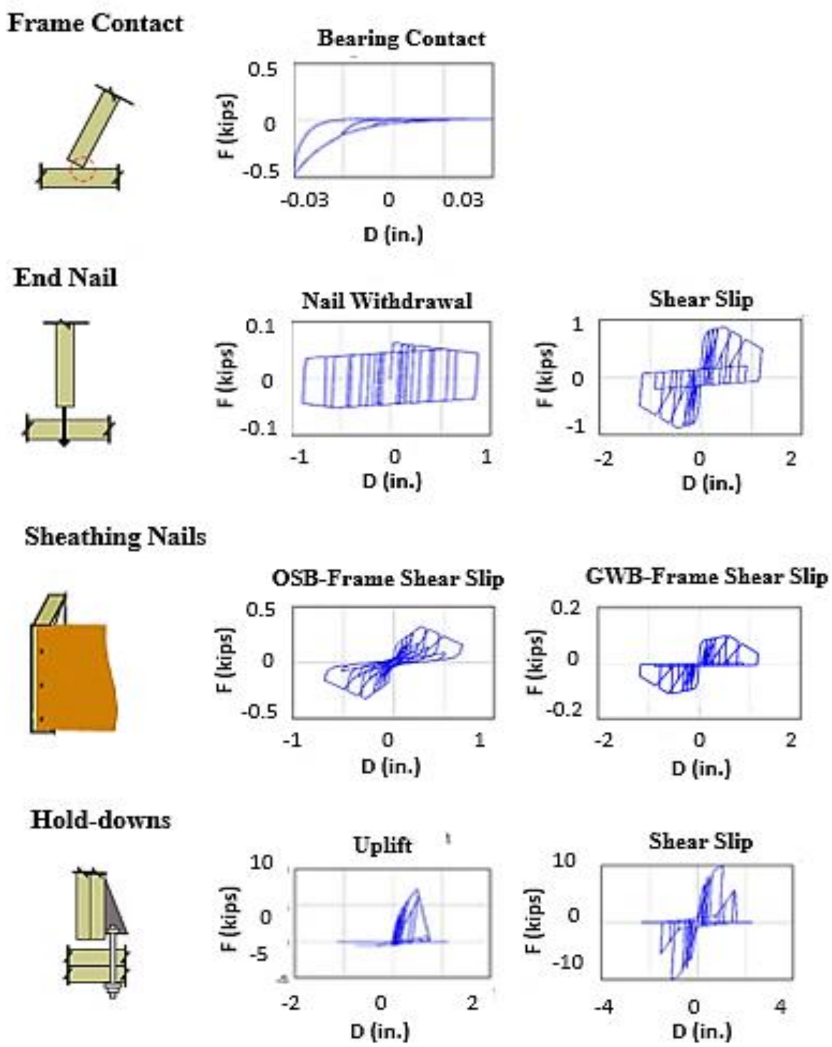


Figure G.4: Hysteretic models for (a) frame contact, (b) end nails, (c) sheathing nails, and (d) PHD5 Hold-downs, (van de Lindt J. W., Pei, C., & Hassansadeh, 2012b)

The monotonic and cyclic response of the shear wall model was determined, as shown in Figure 24 and Figure 25, respectively. The standard cyclic protocol in MCASHEW was used. The ultimate force and initial stiffness was estimated as 87.2kN (19.6 kips) and 2.62kN/mm (15.0 kips/in.) The displacement at ultimate is approximately 125mm (4.9 in).

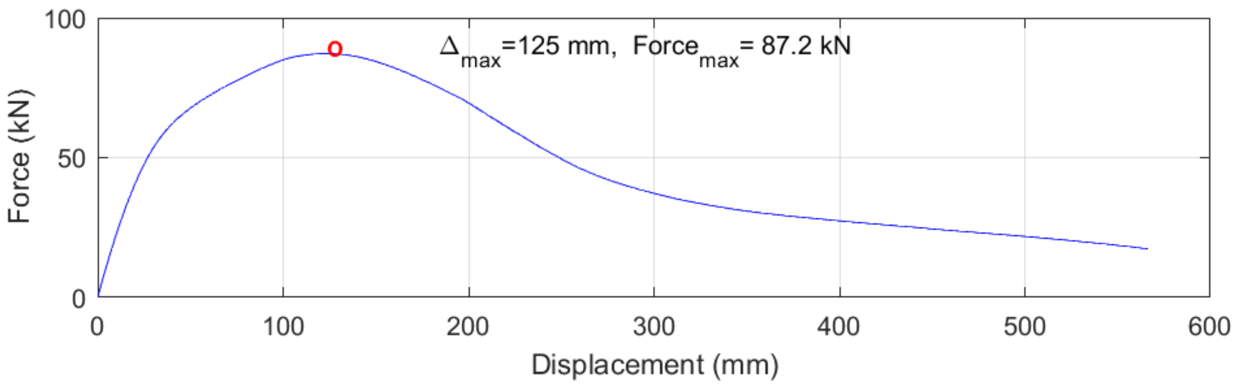
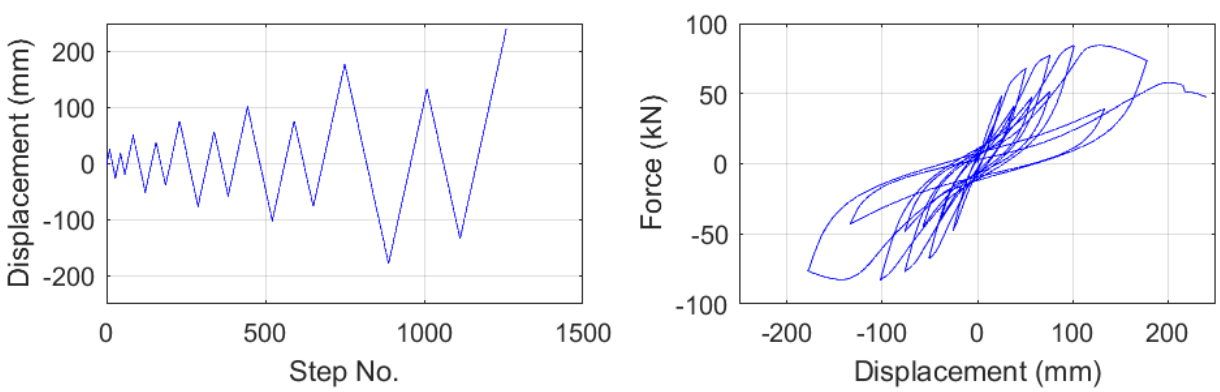


Figure G.5: Monotonic response of classroom shear wall numerical model



(a) (b)

Figure G.6: (a) Standard M-CASHEW2 Protocol, (b) Cyclic Response

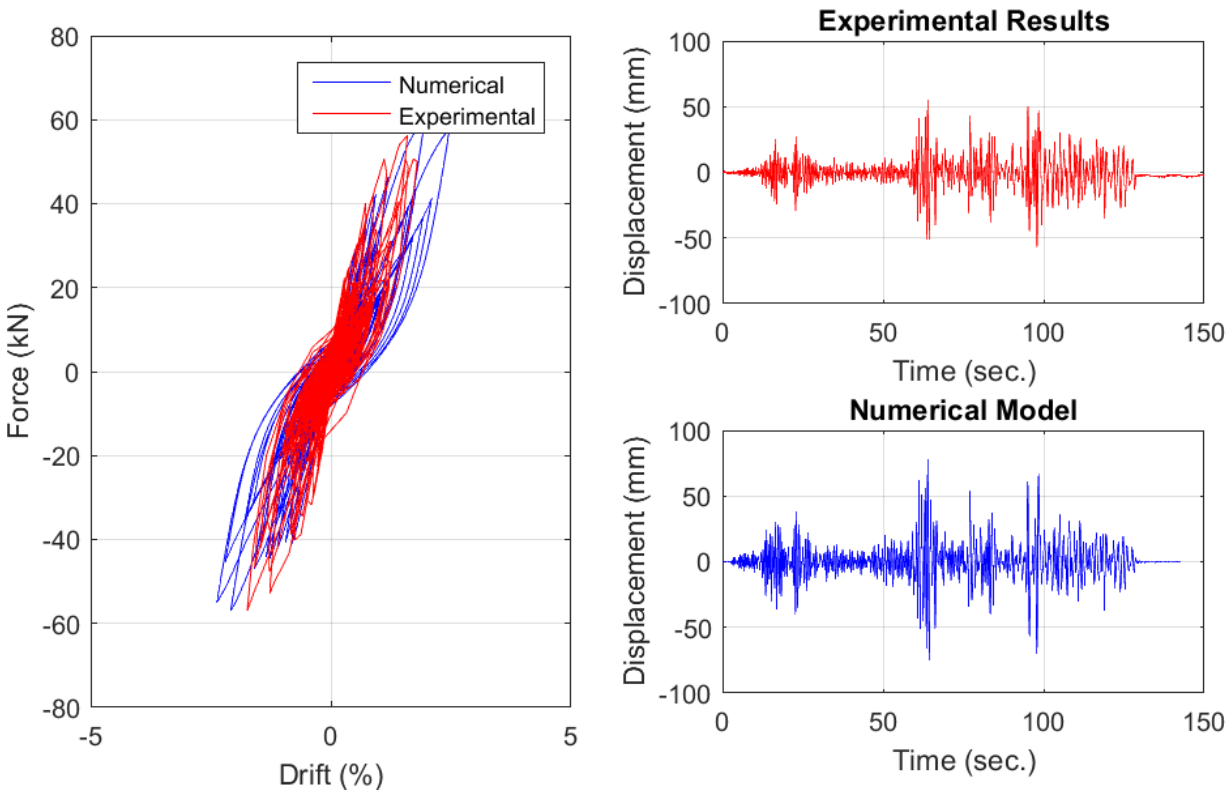


Figure G.7: Run 1 comparison of numerical and experimental hysteretic and time-history response

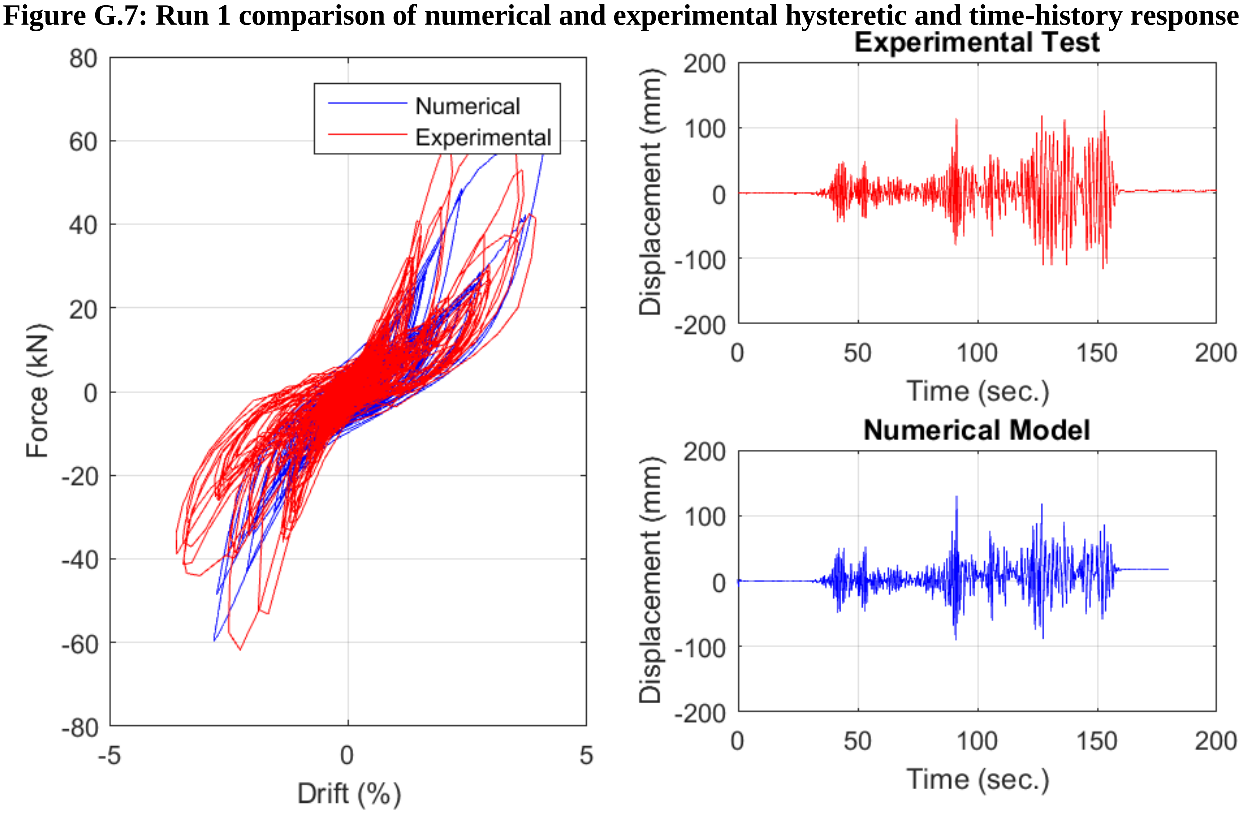


Figure G.8: Run 2 comparison of numerical and experimental hysteretic and time-history response

Global Timber3D Model

The RESST hysteretic model was matched to the monotonic and cyclic response of the classroom wall, as shown in the figure below. The additional cyclic and degradation parameters in the material model are based on data obtained from the tests conducted as part of the CUREE project, the cyclic wall tests from the University of British Columbia as part of the testing program for the School Seismic Retrofit Guidelines (EERF, 2009), tests performed by Bahmani and van de Lindt (2015), as well as the recommendations from the FEMA P-807 (2012) and the technical committee review for the on-going ATC-116 project.

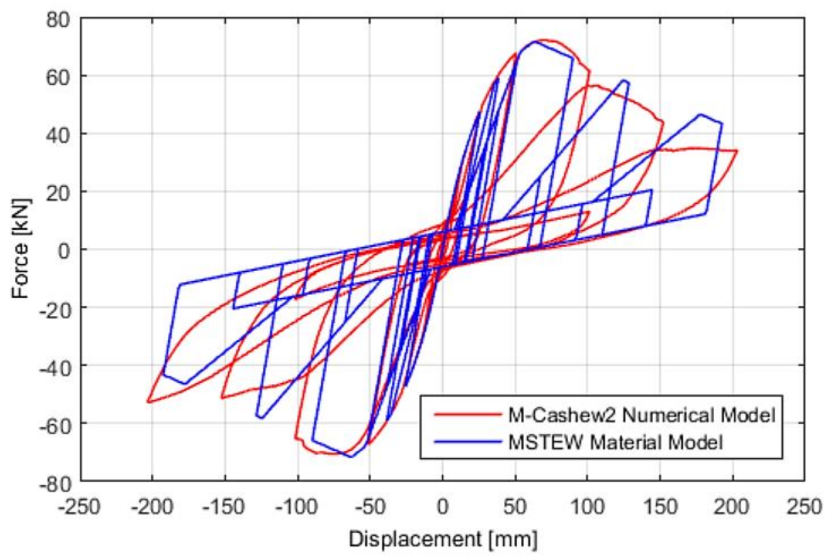


Figure G.9: MSTEW model fit to hysteretic loops for pretest Classroom wall model

It should be noted that around the openings four rectangular sheathing panels were used. The actual configuration involves two C-shaped panels and two rectangular panels. Therefore, moment resistance can develop at the corner of the openings. In a study of a garage light-frame wood wall van de Lindt et al. (2012b) recommended using bilinear springs to connect the rectangular panels and model the nonrectangular sheathing. Where the stiffness of the bilinear springs was calculated as:

$$k_e = \frac{K_{FE}\frac{EA_{par}+EA_{per}}{2}W_{panel}}{n_s L_{strand}} \qquad \text{Eq.F1}$$

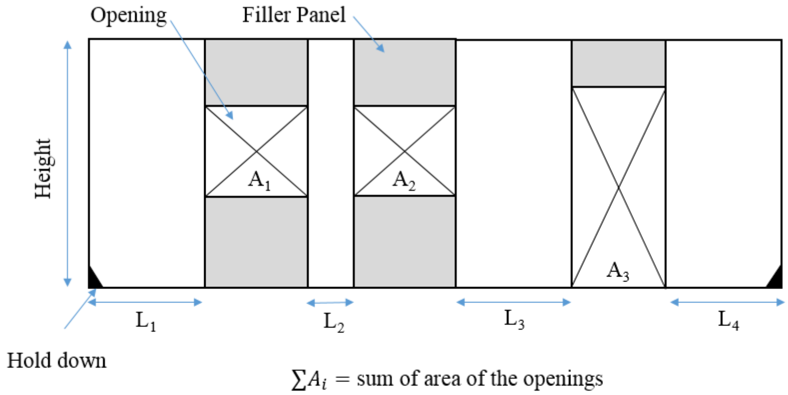
Where:

EA_{par}	Parallel design axial stiffness
EA_{per}	Perpendicular design axial stiffness
K_{FE}	allowable stress design to the nominal design conversion factor for the modulus of elasticity
W_{panel}	Width of the panel
L_{strand}	Average length of the wood strands
n_s	Number of bilinear springs

The contact between the sheathing panels was also not modelled. Bearing and friction may alter the lateral behavior.

The numerical model estimations for the ultimate capacity were compared to the calculated resistance from the Canadian Wood Design code (CWC, 2010). The code capacity was compared to cyclic experimental wall tests performed by UBC as part of the EERF and UBC98 projects. The ultimate strength of the experimental results was scaled linearly to the wall length of the system. The over-strength factor used in the code for wood shear walls is 1.7; an over-strength factor of 2 is recommended.

The FEMA-P807 guidelines recommend the use of an opening factor multiplied by the ultimate strength to account for the strength and stiffness contributions from the coupling beam behavior of the wall pier headers and sills around the openings. The schematic in Figure 27 shows how the opening factor is calculated; this factor is then multiplied by the ultimate strength of a wall of the same length without openings.



$\sum A_i$ = sum of area of the openings
$\sum L_i$ = sum of length of full-height wall segments
H = floor to ceiling wall height

$$\text{Opening Factor} = 0.92\alpha - 0.72\alpha^2 + 0.80\alpha^3$$

$$\alpha = \frac{1}{1 + \frac{\sum A_i}{H \sum L_i}}$$

Figure G.10: FEMA P-807 Opening Factor

Due to the different nailing schedules of the full height sheathing and the sheathing above and below the openings the FEMA P-807 opening factor cannot be simply applied. If the wall was entirely Blocked or unblocked OSB the structure would have a resistance of 151kN and 61kN, respectively. The recommended ultimate resistance was calculated:

$$R_{Model} = R_{LowerBound} - R_{SegmentedUnblocked\ wall} + R_{SegmentedBlocked\ wall}$$

Were $R_{LowerBound}$ was calculated based on the ultimate capacity for unblocked wood based on experimental testing of walls and the FEMA P-807 opening factor guidelines, $R_{SegmentedUnblockedwall}$ and $R_{SegmentedBlockedwall}$ is the resistance scaled to the 2.4m length per side for the unblocked wall prototype and blocked wall prototype, respectively. A schematic used to describe the recommended ultimate resistance is shown in the following figure:

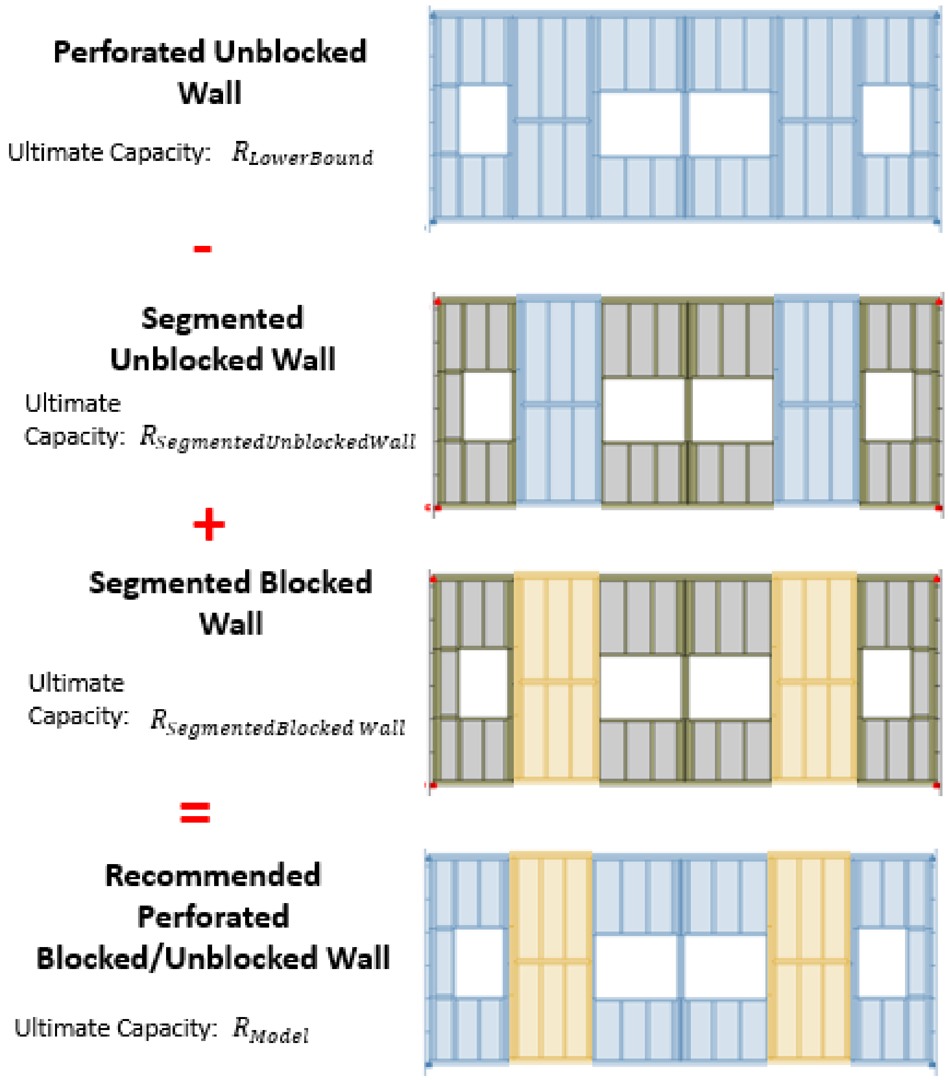


Figure G.11: Recommended Perforated Wall Ultimate Capacity

The recommended modeling resistance to account for the openings based on empirical data is between the upper and lower bound solutions.

Perforated Wall System (FEMA P-807 Opening Factor)		
Upper Bound Blocked Wall	**Lower Bound Unblocked Wall**	**Modeling Recommendation**
155kN	61kN	107kN
76%W	30%W	52%W

If it was assumed that the sheathing above and below the openings do not provide any additional strength or stiffness only 2.4m of solid wall segments for each side of the structure would be considered. This would represent a lower bound solution. GWB was installed on the interior walls of the test specimen and were accounted for in the numerical model using the superposition method. The stiffness and strength hysteretic parameters were linearly scaled to the length of the solid wall segments; the inner segment with the openings were not included. The gypsum wall parameters were based on data obtained from the tests conducted as part of the CUREE project, the cyclic wall tests from the University of British Columbia as part of the testing program for the School Seismic Retrofit Guidelines (EERF, 2009), tests performed by Bahmani and van de Lindt (2016), as well as the recommendations from the FEMA P-807 and the technical committee review for the on-going ATC-116 project.

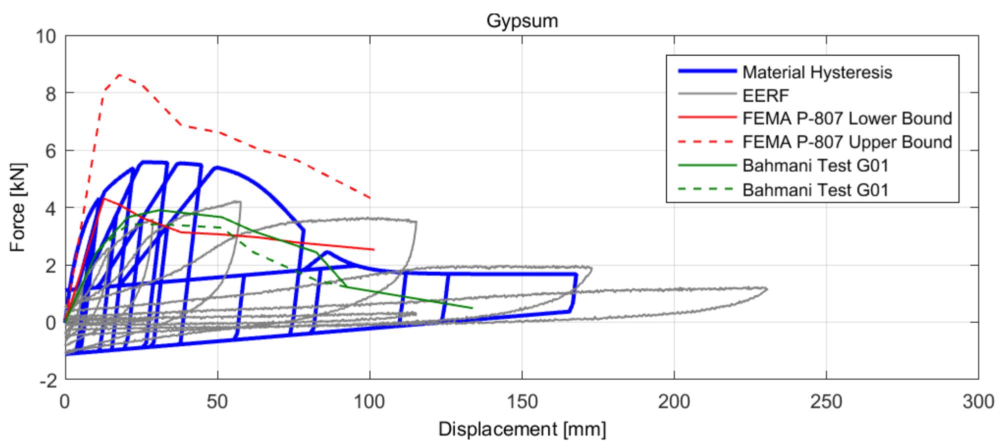


Figure G.12: Gypsum Material Model compared to experimental data (8ft wall segment)

The comparison of ultimate capacity (kN and percentage of the weight) for the segmented and perforated wall approach is summarized in the following table:

Upper-bound and lower-bound ultimate capacity of classroom model

Segmented Approach		Perforated Wall Approach	
Unfactored Code Resistance (R_o=1.7)	**Timber 3D Model (4.0m Blocked Wall)**	**Perforated Wall System (FEMA-P807 Opening Factor – Modeling Recommendation)**	**M-CASHEW2 Model**
56.0kN	71.1kN	104kN	160.0kN
17%W	34%W	43%W	78%W

Appendix H Summary of Weight for School Building Block

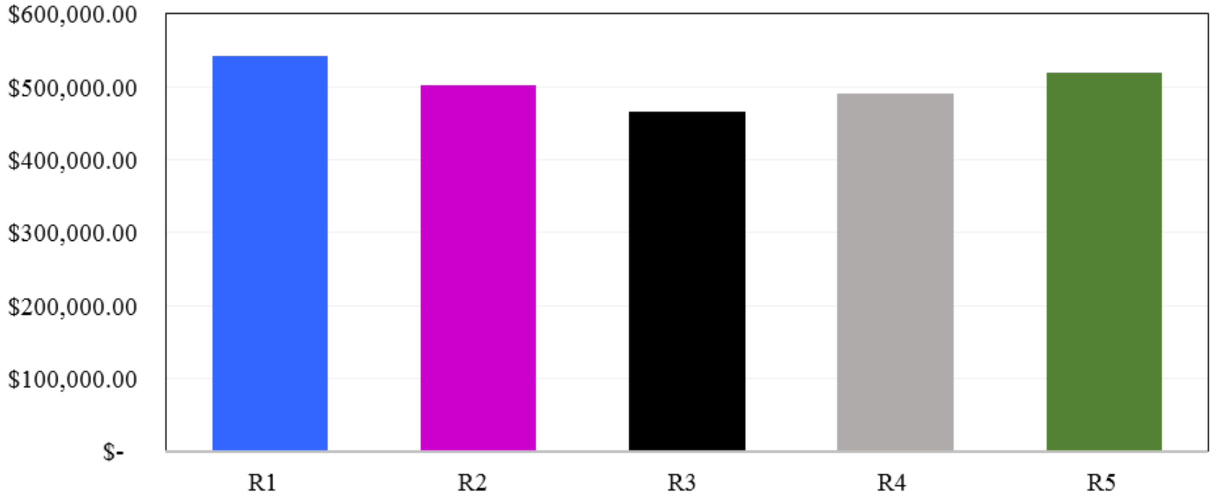
The breakdown of the weight calculation is summarized below:

		Imperial				Metric			
		l	w	Area		Area		kN/m2	Total Mass [kN]
Ceiling Classroom	5/8 Gypsum	1563	284	443892	in2	286.3814	m2	0.097	28.0
	3/8 Plywood	1563	284	443892	in2	286.3814	m2	0.048	13.6
	Shiplap	1563	284	443892	in2	286.3814	m2	0.17	48.7
	Fibre Board	1563	284	443892	in2	286.3814	m2	0.07	20.0
	3x14 @ 16 o/s	1563	284	443892	in2	286.3814	m2	0.29	83.1
	Tar and gravel *(roof)	1563	284	443892	in2	286.3814	m2	0.31	88.8
	3" of insulation (roof)	1563	284	443892	in2	286.3814	m2	0.038	10.8
	Tile (floor)	1563	284	443892	in2	286.3814	m2	0.07	20.0
Ceiling Corridor	3/8 Plywood	1563	109	170367	in2	109.914	m2	0.048	5.2
	Shiplap	1563	109	170367	in2	109.914	m2	0.17	18.7
	Fibre Board	1563	109	170367	in2	109.914	m2	0.07	7.7
	2x4 @ 16 o/s	1563	109	170367	in2	109.914	m2	0.05	5.5
	2x8 @ 16 o/s	1563	109	170367	in2	109.914	m2	0.09	9.9
	Tar and gravel *(roof)	1563	109	170367	in2	109.914	m2	0.31	34.1
	3" of insulation (roof)	1563	109	170367	in2	109.914	m2	0.038	4.1
	Tile (floor)	1563	109	170367	in2	109.914	m2	0.07	7.7
Stair	5/8 Gypsum	333	151	50283	in2	32.44	m2	0.098	3.2
	3/8 Plywood	333	151	50283	in2	32.44	m2	0.048	1.5
	Shiplap	333	151	50283	in2	32.44	m2	0.17	5.5
	Fibre Board	333	151	50283	in2	32.44	m2	0.07	2.3
	2x10 @ 16 o/s	333	151	50283	in2	32.44	m2	0.12	3.9
	Tar and gravel *(roof)	333	151	50283	in2	32.44	m2	0.26	8.4
	3" of insulation (roof)	333	151	50283	in2	32.44	m2	0.038	1.2
	Tile (floor)	333	151	50283	in2	32.44	m2	0.07	2.3

		Imperial				Metric			
		l	**w**	**Area**		**Area**		**kN/m2**	**Total Mass [kN]**
East Wall 10ft section	Windows	40	72	2880	in2	1.85	m2	0.48	0.89
	Stucco	120	140	13920	in2	8.98	m2	0.48	4.31
	2" Insulation	120	140	13920	in2	8.98	m2	0.025	0.22
	Shiplap	120	140	13920	in2	8.98	m2	0.17	1.53
	5/8 Gypsum	120	140	13920	in2	8.98	m2	0.09796	0.88
	3/8 plywood	120	140	13920	in2	8.98	m2	0.0475	0.43
West Wall	Windows	84	24	2016	in2	1.30	m2	0.48	0.62
	Vertical Cedar Siding	237	140	31164	in2	20.10	m2	0.048	0.97
	Shiplap	237	140	31164	in2	20.10	m2	0.1	2.01
	Stucco	237	140	2016	in2	1.300	m2	0.48	0.62
	2 Insulation	237	140	31164	in2	20.10 577	m2	0.025	0.50
	5/8 Gypsum	237	140	31164	in2	20.10	m2	0.09796	1.97
	3/8 plywood	237	140	31164	in2	20.10	m2	0.0475	0.96
	Studs	237	140	31164	in2	20.10	m2	0.07	1.41
Corridor Wall	5/8 Gypsum	16	140	2240	in2	1.45	m2	0.09796	0.14
	3/8 plywood	16	140	2240	in2	1.45	m2	0.0475	0.07
	5/8 Gypsum	16	140	2240	in2	1.45	m2	0.09796	0.14
	3/8 plywood	16	140	2240	in2	1.45	m2	0.0475	0.068
	Studs	16	140	2240	in2	1.45	m2	0.07	0.10
North/South	Stucco	16	140	2240	in2	1.45	m2	0.48	0.69
	Shiplap	16	140	2240	in2	1.45	m2	0.1	0.14
	2 Insulation	16	140	2240	in2	1.45	m2	0.025	0.036
	5/8 Gypsum	16	140	2240	in2	1.45	m2	0.09796	0.14
	3/8 plywood	16	140	2240	in2	1.45	m2	0.0475	0.068
	Studs	16	140	2240	in2	1.45	m2	0.07	0.10

Appendix I Cost Summary of Retrofits

A bar chart comparing a preliminary cost estimation of the retrofits is shown in the figure below; the costs of the retrofits are fairly similar.



The cost breakdown of the retrofit options is summarized below:

Retrofit 1: Shear Walls				
Seismic Upgrade Work				
	Quantity		**Cost per Unit**	**Total Cost**
Selective Demolition				
General interior tear out finishes, millwork etc	1016	m2	12	$ 12,192.00
Slab removal in strip 600mm	82	m	215	$ 17,630.00
Interior wall finishes for sheathing	310	m2	58	$ 17,980.00
Earthwork				
New foundations	20	m3	350	$ 7,000.00
New foundations exterior	60	m3	350	$ 21,000.00
Concrete Work				
Concrete Foundations - reinste slab, dowel anchors to fndn	94	m	185	$ 17,390.00
Crawlspace work - grade bearms on top of seal coat	60	m	350	$ 21,000.00
DWIDAG continuous reiniforcing rods	87	m	375	$ 32,625.00
Concrete 600mm strip at perimeter adj fndn wall	87	m	95.15	$ 8,278.05
Drilled epoxy anchors/rebar to existing	7	m	450	$ 3,150.00
	604	No.	21	$ 12,684.00
Shearwalls				
Plywood shearwalls with blocking and hold-downs	310	m2	88	$ 27,280.00
Connections at top of wall to existing	97	m	85	$ 8,245.00
Diaphragm Upgrades & Connections				
Plywood sheating, metal straps	508	m2	62	$ 31,496.00
Roof Parapet	90	m	42	$ 3,780.00
Exterior Envelope Work				
Reroofing associated with seismic work	508	m2	215	$ 109,220.00
Flashing - roof to wall	90	m	85	$ 7,650.00
Interior Work				
New Drywall on upgraded side walls	310	m2	68	$ 21,080.00
Finishes - Floor repair	8	m2	85	$ 680.00
Finishes - Ceiling repair	97	m2	25	$ 2,425.00
Finishes - Wall repair	310	m2	12	$ 3,720.00
Reinstall Millwork	1016	m2	10	$ 10,160.00
Reinstall Whiteboards	1016	m2	8	$ 8,128.00
Specialties	1016	m2	7	$ 7,112.00
Electrical Work				
Nominal Elc work	1016	m2	28	$ 28,448.00
Mechanical Work				
HVAC	1016	m2	35	$ 35,560.00
Asbestos & Lead Paint Remediation				
Asbestos removel from interior locations, flooring, mech, drywall	1016	m2	65	$ 66,040.00
TOTAL				$ 541,953.05

Retrofit 2: New Stucco Walls

Seismic Upgrade Work

	Quantity		Cost per Unit	Total Cost
Selective Demolition				
General interior tear out finishes, millwork etc	1016	m2	12	$ 12,192.00
Slab removal in strip 600mm	82	m	215	$ 17,630.00
Miscell demolition				
Remove exterior stucco finishes & sheathing to expose wall	271.472	m2	48	$ 13,030.66
Earthwork				
New foundations	20	m3	350	$ 7,000.00
new foundations exterior	60	m3	350	$ 21,000.00
Concrete Work				
Concrete Foundations - reinste slab, dowel anchors to fndn	94	m	185	$ 17,390.00
Crawlspace work - grade beams on top of seal coat	60	m	350	$ 21,000.00
DWIDAG continuous reinforcing rods	87	m	375	$ 32,625.00
Concrete 600mm strip at perimeter fndn wall	87	m	95.15	$ 8,278.05
Drilled epoxy anchors/rebar to existing	7	m	450	$ 3,150.00
Diaphragm Upgrades & Connections				
Plywood sheathing, metal straps	508	m2	62	$ 31,496.00
Roof Parapet	90	m	42	$ 3,780.00
Exterior Envelope Work				
Reroofing associated with seismic work	508	m2	215	$ 109,220.00
Flashing - roof to wall	90	m	85	$ 7,650.00
New Stucco Construction		m2	166.6666667	
Interior Work				
New Drywall on upgraded side walls	310	m2	68	$ 21,080.00
New partitions - stud/drywall both sides			128	
Stair Vestibules			425	
Door/Frames/Hardware			200	
Finishes - Floor repair	8	m2	85	$ 680.00
Finishes - Ceiling repair	97	m2	25	$ 2,425.00
Finishes - Wall repair	310	m2	12	$ 3,720.00
Reinstall Millwork	1016	m2	10	$ 10,160.00
Reinstall Whiteboards	1016	m2	8	$ 8,128.00
Specialties	1016	m2	7	$ 7,112.00
Electrical Work				
Nominal Elc work	1016	m2	28	$ 28,448.00
Mechanical Work				
HVAC	1016	m2	35	$ 35,560.00
Asbestos & Lead Paint Remediation				
Asbestos removal from interior locations, flooring, mech, drywall	1016	m2	65	$ 66,040.00
				$ 501,478.71

Retrofit 3: CLT Walls

Seismic Upgrade Work

	Quantity		Cost per Unit	Total Cost
Selective Demolition				
Slab removal in strip 600mm	82	m	215	$ 17,630.00
Earthwork				
New foundations	20	m3	350	$ 7,000.00
new foundations exterior	60	m3	350	$ 21,000.00
Concrete Work				
Concrete Foundations - reinstall slab, dowel anchors to fndn	94	m	185	$ 17,390.00
Crawlspace work - grade beams on top of seal coat	60	m	350	$ 21,000.00
DWIDAG continuous reinforcing rods	87	m	375	$ 32,625.00
Concrete 600mm strip at perimeter fndn wall	87	m	95.15	$ 8,278.05
Drilled epoxy anchors/rebar to existing	7	m	450	$ 3,150.00
Shearwalls				
CLT Walls	12	No.	1200	$ 14,400.00
Diaphragm Upgrades & Connections				
Plywood sheathing, metal straps	508	m2	62	$ 31,496.00
Roof Parapet	90	m	42	$ 3,780.00
Exterior Envelope Work				
Reroofing associated with seismic work	508	m2	215	$ 109,220.00
Flashing - roof to wall	90	m	85	$ 7,650.00
Interior Work				
Finishes - Floor repair	8	m2	85	$ 680.00
Finishes - Ceiling repair	97	m2	25	$ 2,425.00
Finishes - Wall repair	310	m2	12	$ 3,720.00
Reinstall Millwork	1016	m2	10	$ 10,160.00
Reinstall Whiteboards	1016	m2	8	$ 8,128.00
Specialties	1016	m2	7	$ 7,112.00
Electrical Work				
Nominal Elc work	1016	m2	28	$ 28,448.00
Mechanical Work				
HVAC	1016	m2	35	$ 35,560.00
Asbestos & Lead Paint Remediation				
Asbestos removal from interior locations, flooring, mech, drywall	1016	m2	65	$ 66,040.00
				$ 469,576.05

Retrofit 4: SMF Simpson Strong Tie

Seismic Upgrade Work

	Quantity		Cost per Unit	Total Cost
Selective Demolition				
General interior tear out finishes, millwork etc	1016	m2	12	$ 12,192.00
Slab removal in strip 600mm	82	m	215	$ 17,630.00
Miscell demolition				
Remove exterior stucco finishes & sheathing to expose wall	271.472	m2	48	$ 13,030.66
Earthwork				
New foundations	20	m3	350	$ 7,000.00
new foundations exterior	60	m3	350	$ 21,000.00
Concrete Work				
Concrete Foundations - reinstall slab, dowel anchors to fndn	94	m	185	$ 17,390.00
Crawlspace work - grade beams on top of seal coat	60	m	350	$ 21,000.00
DWIDAG continuous reinforcing rods	87	m	375	$ 32,625.00
Concrete 600mm strip at perimeter fndn wall	87	m	95.15	$ 8,278.05
Drilled epoxy anchors/rebar to existing	7	m	450	$ 3,150.00
	604	No.	21	$ 12,684.00
Shearwalls				
SMF Simpson Strong Tie	4	No.	10000	$ 40,000.00
Diaphragm Upgrades & Connections				
Plywood sheathing, metal straps	508	m2	62	$ 31,496.00
Roof Parapet	90	m	42	$ 3,780.00
Exterior Envelope Work				
Reroofing associated with seismic work	508	m2	215	$ 109,220.00
Flashing - roof to wall	90	m	85	$ 7,650.00
Interior Work				
Finishes - Floor repair	8	m2	85	$ 680.00
Finishes - Ceiling repair	97	m2	25	$ 2,425.00
Electrical Work				
Nominal Elc work	1016	m2	28	$ 28,448.00
Mechanical Work				
HVAC	1016	m2	35	$ 35,560.00
Asbestos & Lead Paint Remediation				
Asbestos removal from interior locations, flooring, much, drywall	1016	m2	65	$ 66,040.00
				$ 491,278.71

Retrofit 5: Distributed Knee Brace

Seismic Upgrade Work

	Quantity		Cost per Unit	Total Cost
Selective Demolition				
Slab removal in strip 600mm	82	m	215	$ 17,630.00
Interior Wall Sheathing	246	m2	58	$ 14,270.22
Earthwork				
New foundations	20	m3	350	$ 7,000.00
new foundations exterior	60	m3	350	$ 21,000.00
Concrete Work				
Concrete Foundations - reinstall slab, dowel anchors to fndn	94	m	185	$ 17,390.00
Crawlspace work - grade beams on top of seal coat	60	m	350	$ 21,000.00
DWIDAG continuous reinforcing rods	87	m	375	$ 32,625.00
Concrete 600mm strip at perimeter fndn wall	87	m	95.15	$ 8,278.05
Drilled epoxy anchors/rebar to existing	7	m	450	$ 3,150.00
	604	No.	21	$ 12,684.00
Shearwalls				
Plywood shearwalls with blocking and hold-downs	310	m2	88	$ 27,280.00
Knee-Brace installation	13.0048	m	105	$ 1,365.50
Diaphragm Upgrades & Connections				
Plywood sheathing, metal straps	508	m2	62	$ 31,496.00
Roof Parapet	90	m	42	$ 3,780.00
Exterior Envelope Work				
Reroofing associated with seismic work	508	m2	215	$ 109,220.00
Flashing - roof to wall	90	m	85	$ 7,650.00
Interior Work				
New Drywall on upgraded side walls	310	m2	68	$ 21,080.00
Finishes - Floor repair	8	m2	85	$ 680.00
Finishes - Ceiling repair	97	m2	25	$ 2,425.00
Finishes - Wall repair	310	m2	12	$ 3,720.00
Reinstall Millwork	1016	m2	10	$ 10,160.00
Reinstall Whiteboards	1016	m2	8	$ 8,128.00
Specialties	1016	m2	7	$ 7,112.00
Electrical Work				
Nominal Elc work	1016	m2	28	$ 28,448.00
Mechanical Work				
HVAC	1016	m2	35	$ 35,560.00
Asbestos & Lead Paint Remediation				
Asbestos removal from interior locations, flooring, mech, drywall	1016	m2	65	$ 66,040.00
				$ 519,171.77