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# NONLINEAR TIME-HISTORY ANALYSIS OF MID-RISE TIMBER BUILDINGS – A COMPARISON BETWEEN THE USE OF CONTINUOUS ROD AND DISCRETE HOLD-DOWN SYSTEMS IN NORTH AMERICA

Hadiseh Mohammadi[1*], Min Sun[2], Xavier Estrella[3], Sardar Malek[4]

[1] Graduate student, Department of Civil Engineering, University of Victoria, Victoria, BC, Canada
[2] Associate Professor, Department of Civil Engineering, University of Victoria, Victoria, BC, Canada
[3] PostDoc, Department of Civil Engineering, Structural Xploration Lab, EPFL, Fribourg, Switzerland
[4] Assistant Professor, Department of Civil Engineering, University of Victoria, Victoria, BC, Canada
[*] hadisehmohammadi@uvic.ca (Corresponding Author)

## ABSTRACT

In recent years, advancements in timber composite products and timber engineering coupled with sustainability concerns about steel and concrete technologies have given momentum to conducting more research on timber-based buildings in North America. Understanding and comparing the performance of various lateral load-resisting systems is one of the key aspects in the design of such buildings for practitioners. Hence, more detailed investigations under higher lateral load levels, in the context of mid-rise buildings located in high seismic risk areas of Canada (e.g., Vancouver Island), seem necessary. Designing taller wood frame shear walls with significant lateral forces often requires sturdier frame elements or more fasteners to control overturning moments and inter-story drifts. In this study, a commercial finite element software, SAP2000, is used to analyze the behaviour of strong shear walls with continuous rod hold-down and discrete hold-down systems. A nonlinear time-history analysis is undertaken using earthquake records obtained from FEMA P695. Maximum inter-story drifts and base shear using different lateral systems are compared and some insights for practitioners are provided. Failure models, tension in nail, and hold-down systems are compared to evaluate the potential failure mechanisms of walls under strong earthquakes. The results demonstrate the better performance of continuous rod hold-down in terms of lateral resistance and inter-story drift.

**Keywords:** Mid-rise timber buildings, high seismic risk areas of North America, lateral load-resisting systems, continuous rod, discrete hold-down, nonlinear time history analysis, SAP2000.

## INTRODUCTION

The majority of buildings in North America in the past decade have been constructed using light-frame wood due to its affordability and ease of construction. In addition, some of the mechanical properties of light-frame wood, such as its light weight and high ductility, make it an attractive choice for mid-rise structures located in high seismic zones, where the lateral strength of the building is provided by shear walls. So, understanding the behaviour of wood frame shear walls is crucial for the construction of sustainable mid-rise structures in high seismic zones.

Typically, wood frame shear wall consist of studs, sheathing, and fasteners, with visually graded lumber members as studs and plates. The most common materials used for sheathing are Oriented Strand Board (OSB) and Plywood, which are connected to the studs with nail. To prevent overturning of the wood frame wall, discrete hold down connections are usually used at the corners of the wall to connect the shear wall to the floor below or to the foundation. In high seismicity zones, conventional discrete hold-downs are insufficient to resist the large overturning moment in wood frame shear walls, hence, continuous rods hold-downs are used to address this issue [1, 2]. Configuration of a typical wood frame shear wall with discrete hold-down and a wood frame shear wall with continuous hold-down is illustrated in Figure 1.

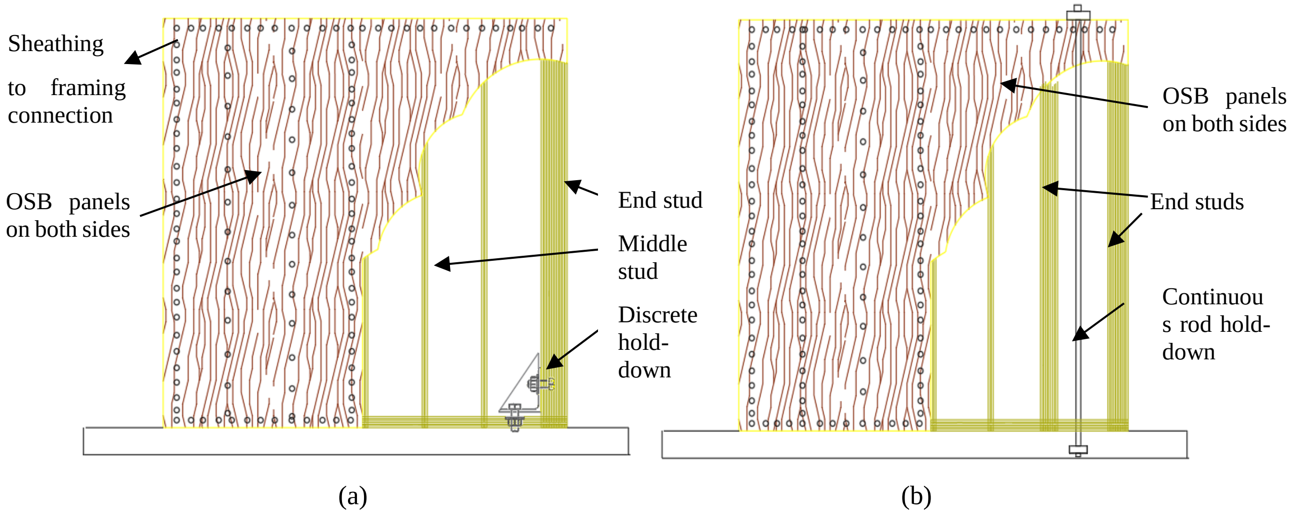



*Figure 1. Configuration of (a) a wood frame shear walls with discrete hold-down, (b) a wood frame wall with continuous rods (reproduced with permission from [1], [2])*

In recent years many studied have done to investigate the behaviour of wood frame shear walls with different hold-down systems [1-12]. Van de Lindt et al. [3] conducted an experiment on a full-scale, six-story, wood frame apartment building with continuous rods at the world's largest shake table in Miki, Japan, to observe its seismic response. According to the results, the building showed very good performance even when subjected to high seismic levels, with a maximum average inter-story drift of 2% and minimal nonstructural damage. Guíñez et al. [4] presented the results of monotonic and cyclic tests in wood frame shear walls with different dimensions that were conducted at Catholic University of Chile. Their main objective was assessing current code expressions, and they found that current code criteria underestimate the shear strength and overestimate the stiffness of the wall in mid-height timber buildings [5]. Estrella et al. [1] carried out experimental-numerical studied on one strory wood frame shear walls with continuous rods with different configurations. They found that the hold down system could increase the strength in wall by 35.8% compare to discrete hold downs. However, investigations on strong wood frame walls are limited. Bagheri et al. [6] have studied on a two-story half-scale light-frame shear wall both experimentally and numerically including the effect of the diaphragm and out of plane stiffness. They found that diaphragm can leads to reduction in deflection of the wall. Recently, Derakhshan et al. [7] conducted experiments to study the behaviour of a new high-capacity shear wall system that features multiple rows of fasteners along the edges of the sheathing. The results showed that compared to a standard shear wall with the same sheathing thickness, nail diameter, and nail spacing, the high-capacity shear wall with two rows of nail had a lateral load resistance that was 1.8-2 times higher. Additionally, the high-capacity shear wall exhibited greater initial stiffness and ultimate displacement than the comparable standard shear wall.

Due to the difficulty, expense, and time required to conduct full-scale tests on wood frame shear walls, researchers have extensively turned to numerical models in recent years as a more practical option for virtual testing, accurately predicting how wood frame shear walls respond to both monotonic and cyclic loads.

Over time, detailed FEM models have been developed by various researchers [8, 9]. Following the Northridge earthquake, which caused substantial damage to wood frame structures, the Consortium of Universities for Research in Earthquake Engineering (CUREE)–Caltech Wood frame Research Project was initiated with the objective of enhancing the timber engineering construction [10]. As part of this project, Folz and Filiatrault [11] proposed a simplified model that can predict the in-plane performance of wood frame shear walls under quasi-static loads. This model was then integrated into a computer program called Cyclic Analysis of Shear Walls (CASHEW). Compared to the experimental results, the model demonstrated precise predictions of the force-displacement response and energy dissipation of wood frame walls under cyclic loads. A modified version of the model was later proposed by Pang and Hassanzadeh [12], that predict the collapse load and failure mechanism of wood frame shear walls. This new model was coded into a computer program called M−CASHEW. Recently, Estrella et al. [2] studied on an efficient and developed approach based on the efforts of previous researchers for monotonic and cyclic nonlinear modeling of strong wood frame shear walls, which demonstrated good agreements between the model predictions and the test results.

Although there has been significant researches on nonlinear models for wood frame walls, there has been a lack of research on modelling approaches for wood frame walls with different hold-down systems in mid-rise buildings located in high seismic regions.
The main objective of this study is to investigate the seismic performance of a midrise wood-frame shear walls with continuous steel rod and discrete hold-downs through a commercial FE software, SAP2000 [13] which could be practical for Engineers. A one-story double-sided wood-frame shear wall with two hold-downs is considered according to the experimental research carried out by Estrella et al. [1]. First, the numerical models are validated with experimental results under monotonic load, then the performance of a six-story wood-frame shear wall is evaluated using time history analysis with the earthquake records which have chosen from North America region. Performance of models with different hold-down systems is compared in terms of inter-story drifts, nonlinear behaviour of nail, hold-down response, failure modes and base shear.

## MATERIALS AND SPECIMENS

A one-story double-sided wood-frame blocked shear wall with continuous rod hold-down and discrete hold-down are used to be studied in this paper. The specimen configuration, sections and material properties are based on Estrella et al. [1] and Guíñeza et al. [4]. The wall configuration consisted of seven studs spaced at 400 mm on center. To withstand the considerable vertical loads and overturning moments present in multistory buildings, the end studs are built using large sections, with both the bottom and top plates consisting of double members as shown in Figure 1. All framing members employing mechanically graded MGP10 Chilean radiata pine and the material properties are according to Estrella et al. [1, 2]. The mean values for longitudinal Young's modulus and the shear modulus of framing members and sheathings are assumed to be 11.4 and 1.3 GPa, respectively. For Young's modulus and shear modulus identical values in longitudinal and transverse directions are assumed. The bottom and top plates were attached to the end studs using 3mm diameter and 100 mm long steel nails. To cover the walls, two OSB panels measuring 11.1 mm in thickness and rated by APA were installed on both sides of the wall (1220 mm in width and 2400 mm in height). The panels were positioned vertically and secured to the frame using 3mm diameter spiral and 70 mm long spiral nail. The nails at the edge were placed 100 mm apart from each other, while the nails in the middle of the panel were spaced 200 mm apart from each other. To anchor the wall to the foundation, continuous rod hold-downs manufactured by Simpson Strong-Tie (fabricated in Pleasanton, CA, USA,) were utilized. The rods are 44.5 mm diameter threaded bars grade 105 (yield strength equal to 724 MPa). To accurately replicate the conditions of continuous rod installation, a timber floor measuring 160 mm in height was added on the top of the wall, resulting in a specimen size of 2440 × 2600 mm (2440 × 2440 mm without the timber floor). Additionally, four shear bolts with a diameter of 25.4 mm were installed at the bottom plate to prevent lateral sliding of the wall.

In addition to strong walls with continuous rods, wood-frame shear walls with discrete hold which were also tested by Guíñeza et al. [4]. The height and length of such walls are 2400, 2470 mm, respectively. All framing materials were 38 × 135 mm (2″ × 6″) dimensional lumber, and studs were spaced at 407 mm on center. The top and bottom plate consisted of double members, whereas the end studs had five members. To cover the walls, two OSB panels measuring 11.1 mm in thickness and rated by APA were installed on both sides of the wall. The panels were positioned vertically and secured to the frame using 3mm diameter spiral and 70 mm long spiral nail. The nails at the edge were placed 100 mm apart from each other, while the nails in the middle of the panel were spaced 200 mm apart from each other. Four horizontal bolts with a diameter of φ1 × 10″ were used to secure the SIMPSON Strong-Tie HD12 hold-down system to the end studs, while one bolt with a diameter of φ1-1/8 × 10″ was utilized to fasten it to the foundation. Furthermore, φ1 × 10″ shear bolts were installed to prevent any lateral movement of the wall. All the section properties and studs configurations are presented in Table 1 and Figure 2 respectively.

*Table 1. Section properties and nail spacing in the models [1], [4]*

| Elements | Section properties | Nail |
|---|---|---|
| End studs (continuous rods) | 4-35x138x2300 mm | 3x70mm @ 100 mm |
| End studs (discrete hold-down) | 5-38x135x2400 mm | |
| Middle stud (continuous rods) | 1-35x138x2300 mm | 3x70mm @ 200 mm |
| Middle stud (discrete hold-down) | 1-38x135x2300 mm | 3x70mm @ 200 mm |
| Top plate (continuous rods) | 2-35x138x2440mm+(3 35x138x2440 mm timber floor + OSB) | 3x70mm @ 100 mm |
| Top plate (discrete hold-down) | 2-38x135x2440 mm | |
| Bottom plate (continuous rods) | 2-35x138x2440 mm | 3x70mm @ 100 mm |
| Bottom plate (discrete hold-down) | 2-38x135x2300 mm | |

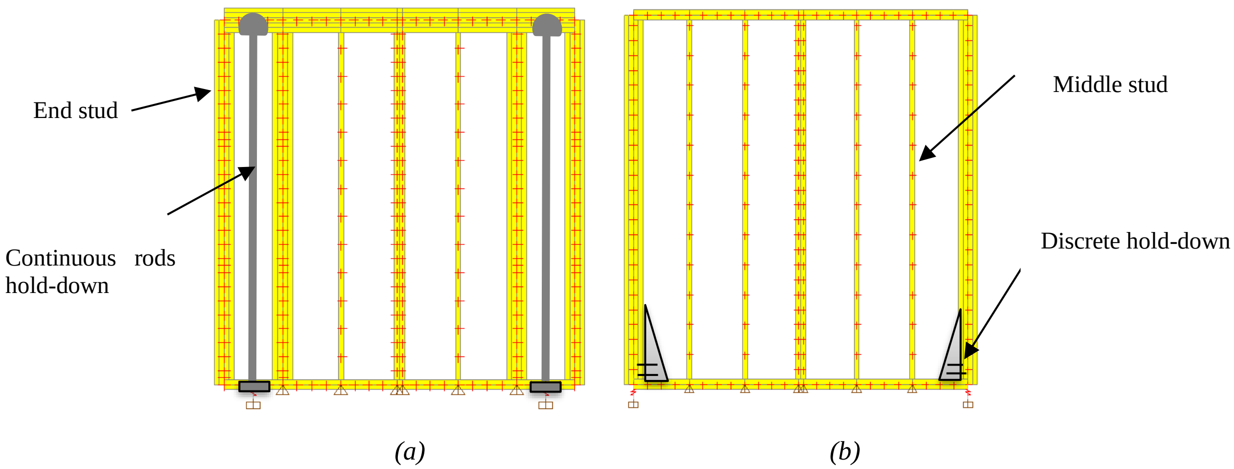



*Figure 2. Wood frame shear walls configuration (a): continuous steel rods, (b): discrete hold-downs according to [1], [4]*

## NONLINEAR NUMERICAL MODELING

The shear walls were modelled in the commercially available software SAP 2000 V.14 [13]; all the wall components were considered. Linear "frame" elements were utilized to model framing elements such as studs, bottom and top plates, while "membrane" elements were employed to model the sheathing panels which are shown in Figure 3. The pin-ended connection between the studs, bottom, and top plates was simulated by providing moment release in these frame elements.

The most important part of energy dissipation during an earthquake occurs through the nonlinear deformation at the nails, which is simulated using multilinear plastic behaviour with Pivot hysteresis type in the software. Defined values for the behaviour of nail obtained based on the shear testing according to Estrella et al. research [1], which had a good agreement with the modified Stewart hysteretic model (MSTEW) proposed by Folz and Filiatrault [11]. It should be noted that the nonlinear behaviour of nail is defined separately for two transitional DOF of the nail which are under shear force in plane of the wall, and this behaviour is illustrated in Figure 4.

Multi-linear elastic behaviour is used to model the nonlinear behaviour of the discrete hold-downs. The vertical links possess a tensile stiffness of $k_t$ = 11.85 kN/mm which is determined using the allowable tensile and deflection data provided in the design catalog [14]. Additionally, the links were given a high compression stiffness to simulate the contact between the bottom plate and the foundation. A simply-supported restraint is implemented at the bottom of the vertical link to prevent sliding of the wall, while the rotational spring has zero stiffness. In addition, continuous rod hold-down was modelled using link elements with force-deformation behaviour which is defined according to Estrella et al. experiment [1], and is shown in Figure 5.

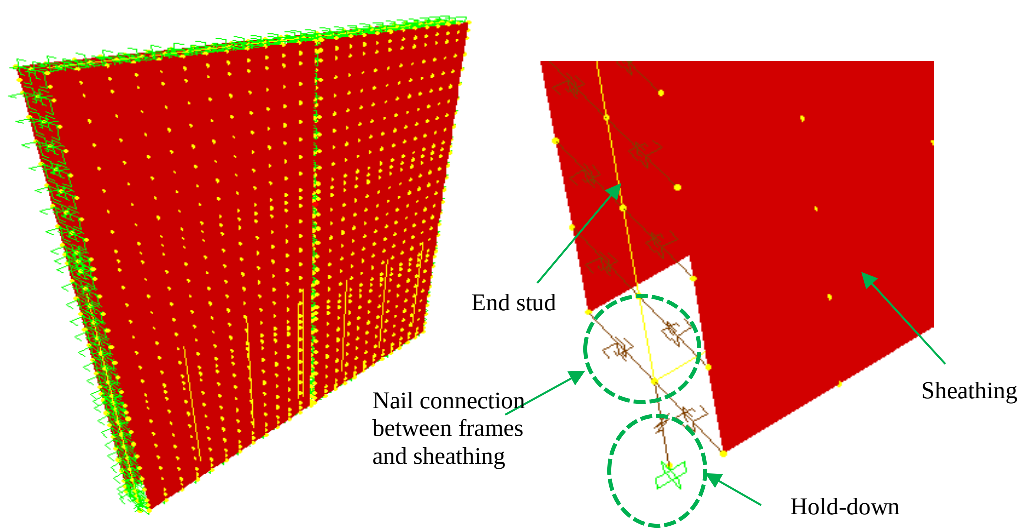



*Figure 3. Wood frame shear wall model configuration in SAP2000*

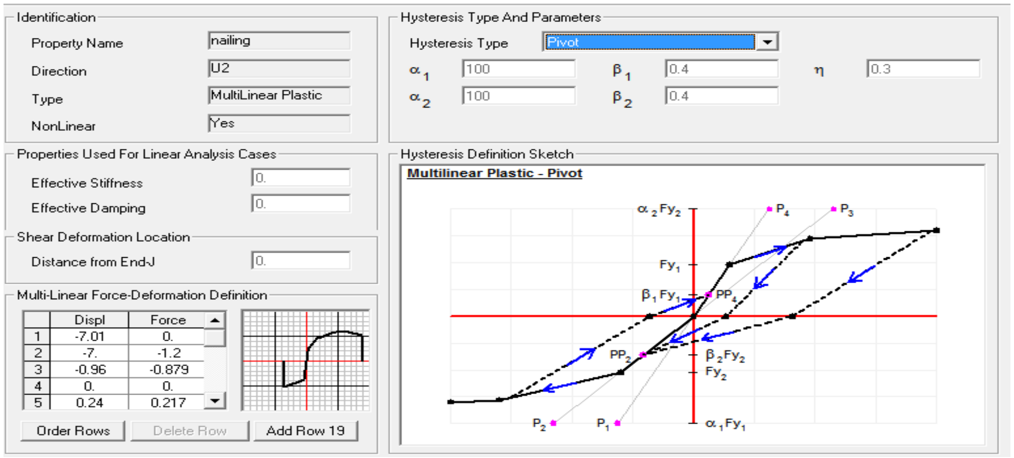
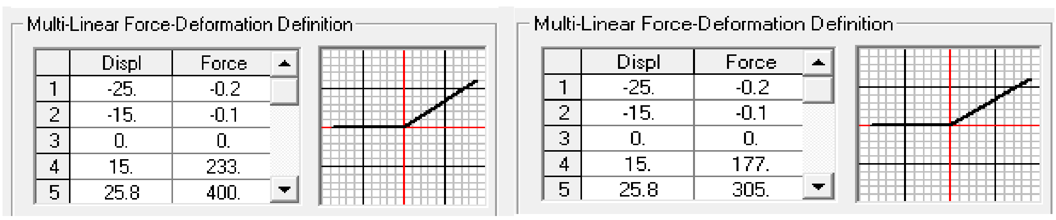



*Figure 4. Nonlinear behaviour of the nail connections between sheathings and frames in SAP2000 obtained from Estrella et al. [1]*




*a* *b*

*Figure 5. Multi-linear behaviour of a) continuous rods, b) discrete hold-down in SAP2000 estimated from Estrella et al. ([1], [2])*

## MODEL VALIDATION

In this section, the accuracy of the proposed model has been investigated based on experimental specimens selected from the reported experiments in Estrella et al. [1] and Guíñeza et al. [4]. Push over (monotonic load) analysis is conducted for the wood frame shear wall model with both discrete hold-down and continuous rods in SAP2000. Although some discrepancies may be expected due to the nature of applied loads, cyclic test data and monotonic estimates have been compared for verification purposes. Figure 6 shows good agreement between test results and SAP2000 models prediction through load-displacement capacity curve.

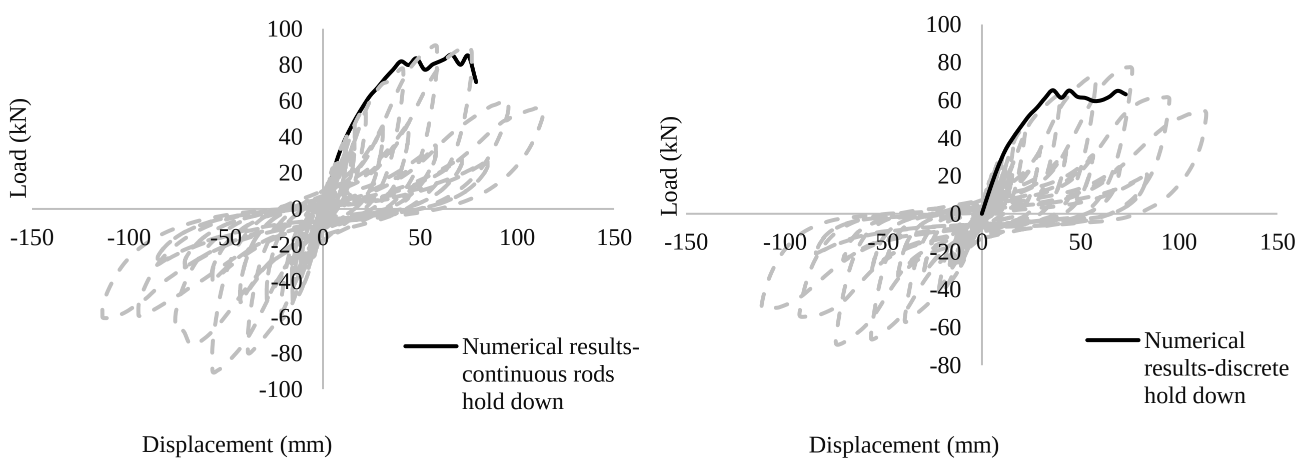



*Figure 6. Comparison between the pushover curves of numerical wood frame shear walls with different hold-down systems in SAP2000 and cyclic curves of different hold-down systems from experiment [1], [4]*

Furthermore, comparing the numerical models with different hold-down systems using SAP2000 in terms of push-over curves according to Figure 7, indicates that the wall with continuous rod hold-down system demonstrates a 23% higher strength and higher initial stiffness than the other model. This suggests that the continuous rod hold-down system provides superior performance in resisting lateral load, making it a more efficient choice for structures that are subjected to such loads. It should be noted that the experiment which was done by Estrella et al. [1] under cyclic load confirms this finding.

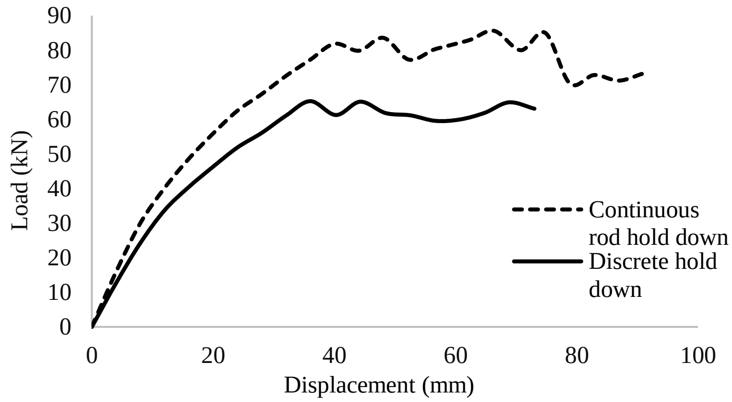



*Figure 7. Comparison between push over curves in wood frame wall with different hold-down systems in SAP2000*

## TIME HISTORY ANALYSIS

To evaluate the seismic performance of multistory wood-frame shear walls with different hold-down systems, a six-story shear wall representing the wall system in light-frame wood building tested by Van de Lindt et al. [3] has been considered. The building has an approximate plan dimension of 18 × 12 m, with a height of 17 m. The considered dead and live loads are 68 Pounds per Square Foot (psf) and 40 psf, respectively, for residential building according to National Building Code of Canada (NBC) [15].

A 2D nonlinear time history analysis was conducted on the six-story wood frame shear wall with the length of 2.44 m. Lumped masses representing the building seismic loads were assigned to the top of each wall at story level. Each mass is calculated based on the loads in each story. The numerical model of the six-story wood frame shear wall with different hold-down systems is shown in Figure 8.

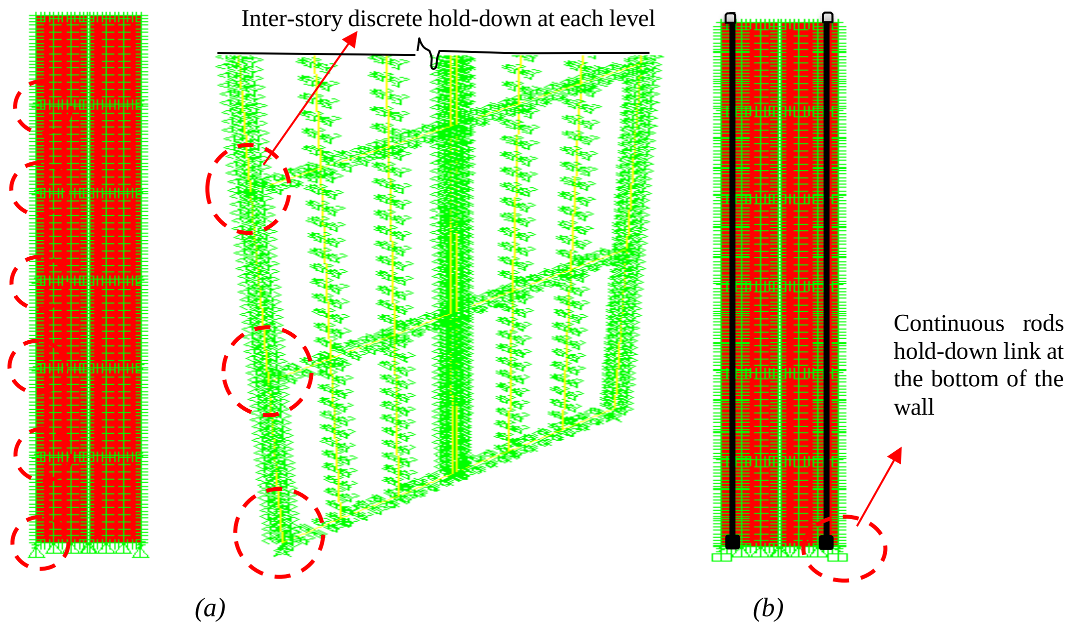



*Figure 8. Numerical model for six-story wood frame shear wall with: (a) discrete hold-down, (b) continuous rods hold-down*

**Loading-Time history records**

The structure is subjected to three earthquake records; Imperial Valley at El Centro station, Northridge at Beverly Hills - Mulhol station, and Loma Prieta at Gilroy Array #3 with scaled peak ground acceleration (PGA) of 0.35g were considered. High magnitude records can be a major threat to structures due to their longer durability and greater amount of released energy. Therefore, the selected earthquakes for the nonlinear time history analysis of the system have magnitudes greater than 6.5 which are selected from the far-field records listed in FEMA-P695, appendix A [16]. The record specifications and horizontal acceleration spectra are shown in the Table 2 and Figure 9 respectively.

*Table 2. Records specification*

| Earthquake | Station | Year | M | PGA(g) | |
|---|---|---|---|---|---|
| | | | | Component 1 | Component 2 |
| Imperial Valley | El Centro Array #11 | 1979 | 6.5 | 0.38 | 0.36 |
| Loma Prieta | Gilroy Array #3 | 1989 | 6.9 | 0.56 | 0.37 |
| Northridge | Beverly Hills - Mulhol | 1994 | 6.7 | 0.52 | 0.42 |

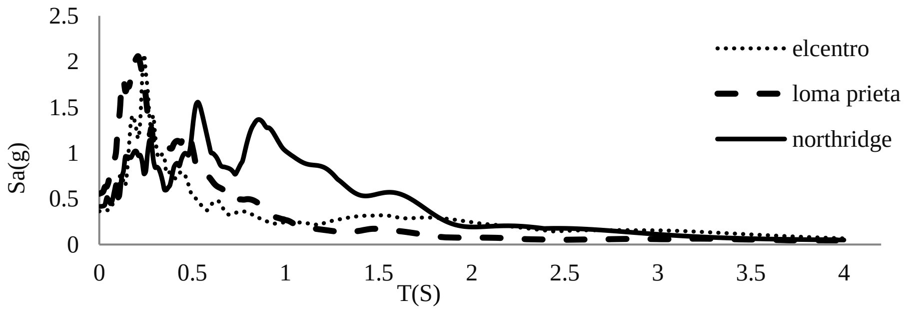



*Figure 9. Horizontal acceleration spectra of the components with maximum PGA*

**Results**

**Inter-story drift**

The inter-story drifts, defined as relative displacement between two floors during the seismic event, estimated from the two analytical models with different hold-down systems are compared in this section. The maximum response obtained from three different time history analysis is reported for this purpose. The results, as illustrated in Figure 10, indicate that the maximum drift in the model with discrete hold-down (i.e. 2%) is almost twice the maximum drift in the model with continuous rods hold down system (1.17%). The uniform drift distribution along the height in the model with continuous rods hold-down demonstrates a better performance of such wall systems, and prevents the formation of soft-story failure during an earthquake [17].

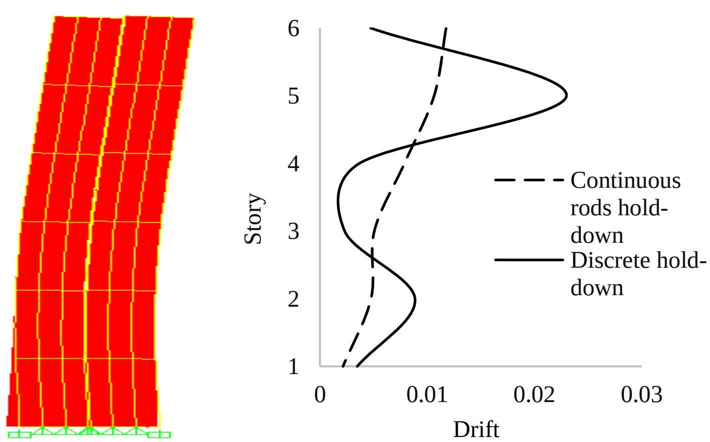



*Figure 10. Inter-story drift in the models with different hold-downs*

**Base Shear**

Base shear is an important response parameter in structural design, as it represents the amount of seismic force that a structure should resist at its base level. As it illustrated in Figure 11, the maximum base shear in the model with discrete hold-downs (516 kN) is 8.5 times greater than the base shear in the model with continuous rods hold-down (70 kN). Results suggest that employing continuous rod hold-down system could provide increased lateral resistance by creating an uninterrupted load path, which can enhance the structural integrity.

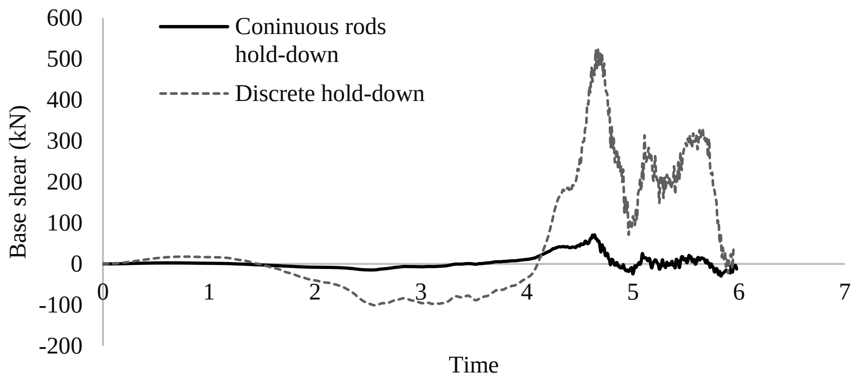



*Figure 11. Base shear in the models with different hold-downs*

**Hold-down system response**

Figure 12 depicts the hold-down response in models with different hold-down systems, where a linear behaviour can be observed. Despite this, the results indicate some hysteresis behaviour under compression and tension loading. The experimental results which was done by Estrella et al. [1] confirm this behaviour in the continuous rod hold-down system. It should be noted that the yield strength (724 MPa) of the steel was not exceeded and the hold-down systems remained in their elastic range under the earthquake loading.

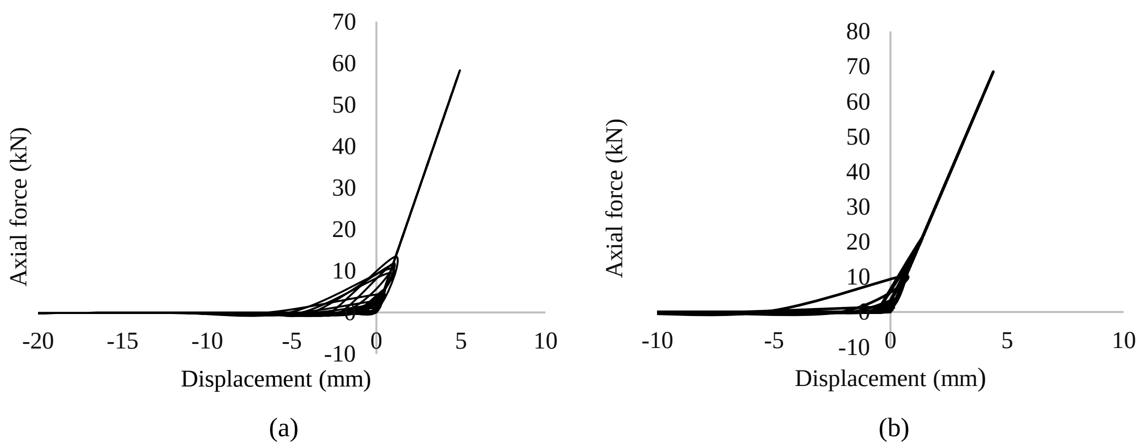
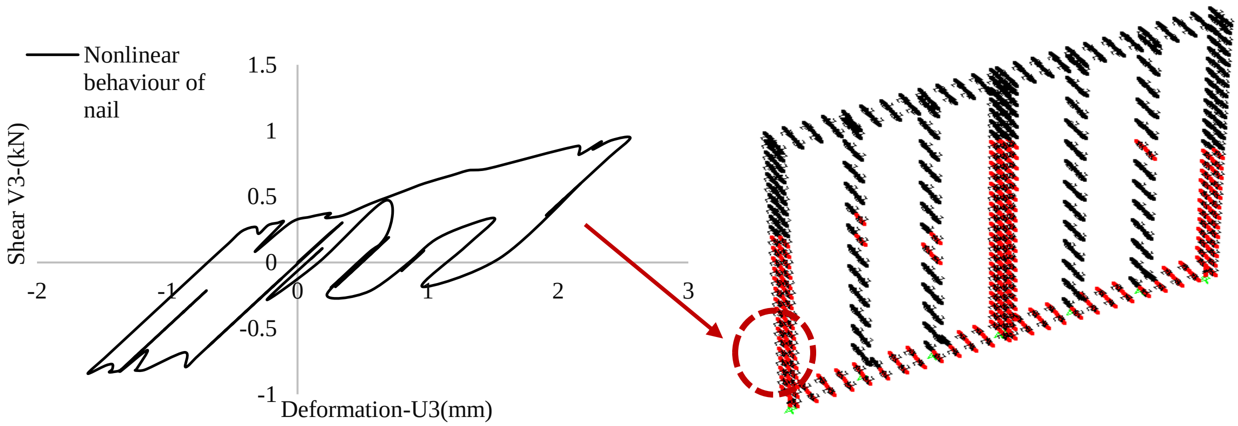



*Figure 12. Behaviour of hold-down in walls with: (a) discrete and (b) continuous rods*

**Nonlinear behaviour of nail and the location of failures**

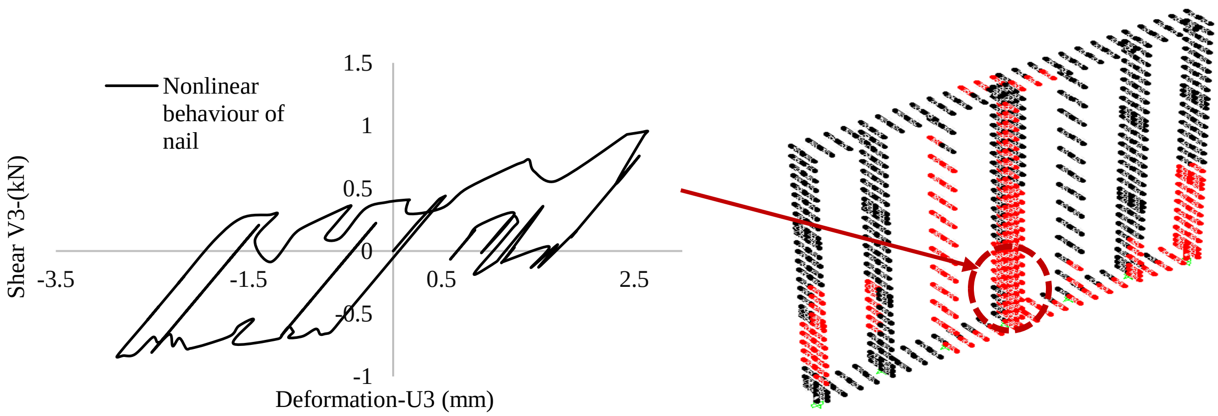
As sheathing to framing connection through the nail has a significant effect on the behaviour of shear wall, the nonlinear behaviour of nail and the location of maximum shear is assessed in two models with different hold-down systems. The nonlinear behaviour of nail is mostly concentrated in the central studs and end studs in the model with discrete hold-down. As shown in Figure 13, the maximum intensity of the nonlinearity is at the bottom of end stud, which is subjected to the maximum shear force of 1.19 kN.

According to Figure 14, the location of failure and nonlinear behaviour of nail is concentrated in the central studs with the maximum shear force of 1.37 kN in the model with continuous rods hold-down system. This may be due to the larger number of sheathing to framing connections at the end studs in the model with continuous rod hold-down system which provides a higher shear stiffness at both ends of the wall. In Figure 14(a), the nonlinear behaviour of the nail at the bottom of the central stud is shown, while the nail in other part of the wall exhibit mainly a linear behaviour. Similar behaviour of nails was reported in Estrella et al. [1].




*(a)* *(b)*

*Figure 13. a) Nonlinear behaviour of one nail at the bottom of end stud, b) Location of max shear forces (0.85<V<1.19 kN) in the first story shear wall under Loma Prieta Earthquake record in the model with discrete hold-down*




*(a)* *(b)*

*Figure* 14. *a) Nonlinear behaviour of one nail at the bottom of central stud*, b) Location of max shear forces (0.85<V<1.37 kN) *in the first story shear wall under Loma Prieta Earthquake record in the model with continuous rods hold-down*

## CONCLUSIONS

This paper presents the development and predictions of a numerical model to better understand the lateral behaviour of wood-frame shear wall with different hold-down systems. Response parameters such as inter-story drift, base shear, hold-down systems response, nail and location of failures are compared in the models with different hold-down systems. Three far field earthquake records with scaled PGA of 0.35g were employed for this purpose. Results demonstrate a better wall performance with continuous rods hold-down system in term of a lower and more uniform inter-story drifts. Also a significant reduction of base shear and hence higher wall strength were noted in walls with continuous rod hold-down system compared to traditional walls with discrete hold-down. Hold-downs remain elastic exhibiting a linear behaviour in both models. It is worth mentioning that utilizing different hold-down systems led to different damage patterns. Employing continuous rods hold-down system can lead to transferring the failure of nails in the end studs to central studs.

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