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Laboratory Testing of Structural Timber Components and a Full-scale Wood Frame to Support Seismic Retrofit Guidelines for School Buildings in British Columbia

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ABSTRACT

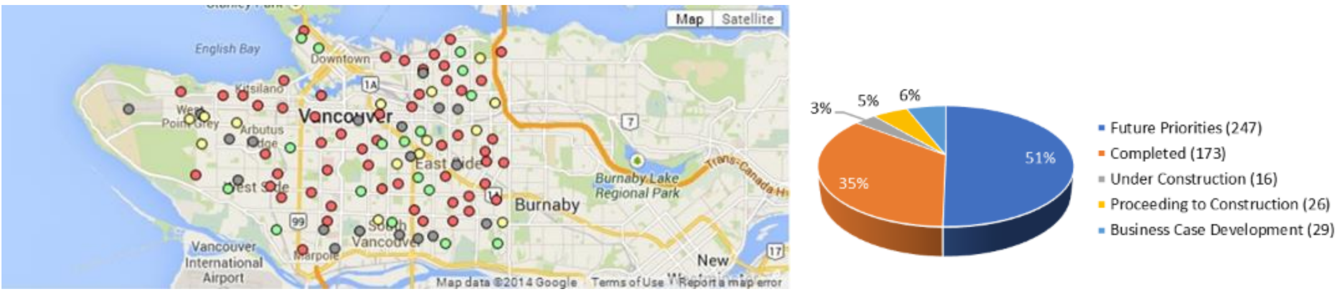
This paper describes a series of experimental works which were carried out as part of the Seismic Retrofit Guidelines (SRG) for school buildings in the Province of British Columbia (BC). The primary objectives of these teste were to provide full-scale test data on Canadian wood constructions subjected to long duration subduction zone motions; and verify the critical performance-based hypothesis that forms the basis of the SRG. The test data can also be used to calibrate and validate for the numerical model developed for wood frame structure subjected to large intensity earthquake loading. These experimental works consist of two phases: 1) a series of shake table tests on a full-scale wood frame; and 2) cyclic quasi-static tests on a wood roof deck diaphragm and shear wall specimens obtained from a school in the province. The full-scale wood frame specimen tested in phase 1 was intended to represent a one-story wood frame classroom in Victoria or Masset, BC constructed in 1980s. The intention of the design was to have a retrofitted structure to resist seismic hazards for the city of Victoria specified by NBCC 2015, with a focused lateral load resisting system. The diaphragm and shear wall specimens tested in phase 2 were two pieces of construction samples taken from the Lake Trial Middle School in Courtenay, BC. The tests were conducted at the Earthquake Engineering Research Facility of the University of British Columbia. The outcomes of these tests were to observe the behavior of the wood frame components and generate test data that provides good insight into the performance of existing wood frame school building components. These results are presented and discussed in this paper.

Keywords: Seismic Retrofit, Timber Structure, Shake Table Test, Cyclic Quasi-static Test, Shear Wall, Diaphragm.

INTRODUCTION

The province of British Columbia (BC) is located on the West Coast of Canada, which is a region of high seismic risk. Southwestern BC, where about 80% of the population of the province lives, has a unique tectonic setting that three sources are contributing to the earthquake hazards: crustal earthquakes that occur along shallow faults in North American plate; subcrustal earthquakes that occur within subducting Juan de Fuca plate; and subduction earthquakes which are caused by slip between two plates [1]. Construction in BC is regulated by the provincial design code, which is based on the National Building Code of Canada, NBCC [2]. The design response spectrum in the NBC is based on a uniform hazard spectrum (UHS) that envelops the spectral acceleration values from all three-earthquake types. With the release of updated seismic hazard model by Geological Survey of Canada (GSC), the design UHS for most of cities in southwestern BC have increased, posing a higher seismic risk for older existing buildings [3].

In 2004, the BC Ministry of Education (MOE) has launched a $1.5 billion seismic mitigation program to ensure the safety of all public elementary and secondary schools. In total, the BC MOE has around 1,600 provincial public schools, of which approximately 750 are located in high seismic region. Figure 1(a) illustrated the school distribution in Vancouver District. Currently, 491 of these school buildings have been categorized as high risk, where 35% have completed seismic mitigation, 3% are under construction, 5% have been proceed to construction. Of the remaining schools, the business case for 6% is being developed and the rest of schools are in the future priorities, as seen in Figure 1(b). The seismic mitigation program is being implemented by the BC MOE in collaboration with the Engineers and Geoscientists BC (EGBC) and The University of British Columbia (UBC). This includes the development state-of-the-art performance-based technical guidelines for practical engineers when conducting seismic risk assessment and retrofit design project of school buildings. An extensive applied research program has been carried out at UBC and evolved over time with enhancements and improvements, including both full-scale experimental testing and numerical studies on a variety of structural components and systems.



(a) *(b)*

Figure 1. (a) school distribution in Vancouver District, (b) status of BC school buildings within seismic mitigation program.

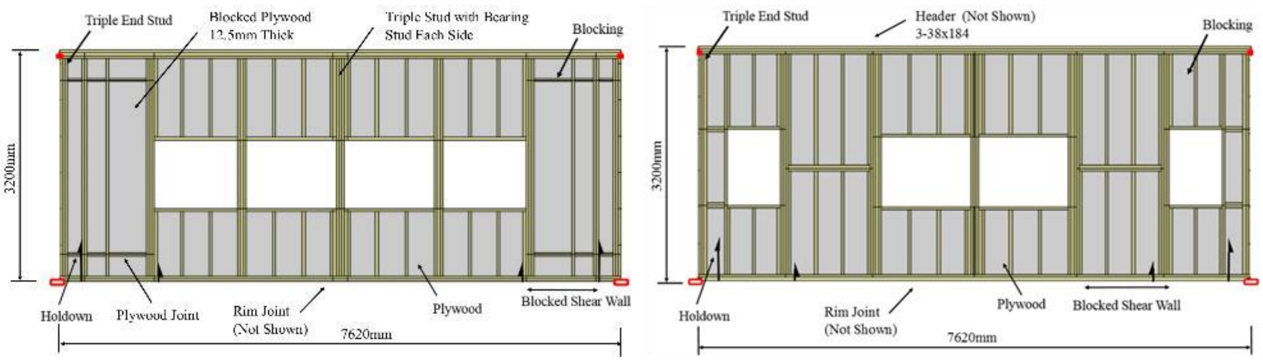
As one of the most conventional construction types in North America, extensive research programs have been conducted on the wood frames in the last two decades [4, 5, 6, 7, and 8], including similar shake table test program. However, in most of those studies, the specimens were designed according to the building code of US. The key characteristics of the subduction earthquake type (e.g., long duration, large magnitude, and aftershocks) were not taken into considered for the assessment. It has been reported that the long duration motions and main shock-aftershock (MSAS) sequences have significant effects on response of the structures in collapse and other damage states [9, 10 and 11].

This paper firstly presents the description and results of a full-scale test program conducted at UBC on school wood frame structure in 2016 and 2017 as part of the Seismic Retrofit Guidelines - 3rd Edition (SRG3) [12]. The experimental program consisted of series of shake table tests on a full-scale specimen represents a one-story wood frame classroom in either Victoria or Masset. Secondly, a series of reverse cyclic tests performed on two structural components cut from an existing wood frame school building are describes, and the test results are presented. The primary objectives of these tests were to: 1) provide full-scale test data; and 2) verify the critical performance-based hypothesis that forms the basis of the SRG Post-earthquake Evaluation Guidelines. The second objective of this research was to evaluate collapse prevention (CP) performance of the wood frame components subjected to high value strong ground motion and large deformation. The test data can also be used to calibrate and validate for the numerical model developed for wood frame structure subjected to large intensity earthquake loading [13 and 14].

TESTING OF THE FULL-SCALE WOOD FRAME

Test Specimen

The test specimen was intended to represent a one-story wood frame classroom in either Victoria or Masset. The design of test specimen was provided by a local engineering consultant according to National Building Code of Canada [2] and Wood Design Manual [15]. As part of the test program, several different final response predictions were made, and the intention was to see damage after the testing, which will provide the basis for the post-earthquake evaluations by the inspectors. The specimen had a plan dimension of 7.62m x 6.096m. For test purposes, the two walls in the direction of shaking have been made identical. Both walls are designed as exterior walls and each included two blocked shear walls to provide the lateral resistance. Each shear wall panel was 1m wide, with a Hold-down at each end. The sheathing nails on the blocked shear wall segment were 8d common nails spaced at 100mm on the sheathing panel edges and 150mm on the interior studs. The unblocked wall sheathing nails were 8d common nails spaced at 150 mm on the sheathing panel edges and 300 mm on the interior studs. The studs were 2x4 Douglas Fir Lumber and the sheathing was 9.5mm plywood panels. Gypsum wall board (Drywall) was used to cover all walls inside the specimen. Two main configurations; Patterns “A” and “B” were considered for the exterior walls in the direction of shaking, as shown in Figure 2. Pattern “A” features four window openings at the middle of the wall and a 1m shear wall at either end. The intention of the design was to have all of the lateral load taken by the shear walls. Each wall had a hold-down at either end (a total of 4 on each side of the structure). The structure was symmetric is both directions. Pattern “B” was similar to Pattern “A” in that was has the same number of openings; this time the shear walls had been moved towards the center, and were slightly wider. In addition, they were split into two panels with a horizontal joint between them, which was intended to have movement and dissipate energy. For this configuration, the hold downs were not moved so the shear walls only had hold-downs on their outer edges. For more specimen description refer to [13]. Table 1 presents the test specimens and testing program.



(a) (b)

Figure 2. Exterior wall framing and configuration for: (a) Pattern "A", and b) Pattern "B".

Table 1. Test specimens and testing program.

Specimen No.	Shear Wall Pattern	Ground Motion Record	Test No.	Level of Intensity	Date of Test
Sp-1	A	Tohoku SIT	Sp1-1	70 %	16 November 2016
			Sp1-2	100 %	
			Sp1-3	100 %	
Sp-2	A	Tohoku SIT	Sp2-1	90 %	28 November 2016
			Sp2-2	90 %	
Sp-3	A	Tohoku SIT	Sp3-1	70 %	15 December 2016
			Sp3-2	70 %	
Sp-4	B	Tohoku MIY	Sp4-1	100 %	23 February 2017
			Sp4-2	125 %	
Sp-5	B	Kobe	Sp5-1	100 %	7 April 2017
		Kobe	Sp5-2	115 %	
		Tohoku SIT	Sp5-3	90 %	
Sp-6	B	Sine Sweep	Sp6-1	0.05 g	11 May 2017
		Geiyo	Sp6-2	100 %	
		Sine Sweep	Sp6-3	0.05 g	
		Geiyo	Sp6-4	100 %	
		Sine Sweep	Sp6-5	0.05 g	
Sp-7	B	Sine Sweep	Sp7-1	0.05 g	17 May 2017
		Tohoku SIT	Sp7-2	70%	
		Sine Sweep	Sp7-3	0.05 g	
		Tohoku SIT	Sp7-4	100%	
		Tohoku SIT	Sp7-5	120%	
		Sine Sweep	Sp7-6	0.05 g	

Test Setup

The large linear shake table with dimensions of 6m x 7.5m at the Earthquake Engineering Research Facility (EERF) of the UBC was used for earthquake motions simulation. The table itself can displace +/- 450mm, with a maximum velocity of 75 cm/s. The dynamic actuator has a maximum pushing force of 260 kN. The shake table is displacement controlled and the earthquake ground motion is put in as a displacement command signal. The test specimen was constructed by a local construction company on the shake table and the calibration of the test records was subsequently performed before specimen fully loaded.

A set of six 1.5m x 7.8m steel plates with total weight of 250 kN was installed on top of the specimen to generate the inertia mass. The plates were connected to the walls using a steel compression strut. The plates were held against the center compression strut by a pair of Dywidag bars with 48mm dia. above and below the plates. Steel safety end posts were installed at either end to prevent a complete collapse of the system. To avoid the contact between inertia plates and the roof, a 203x203x.6.5 mm HSS section was added to each plate, with two high-strength 25 mm dia. rods through to the underside of the plates. The rods pulled the plates together, separating from the roof, and in addition allowed for installation of two 100x100 mm HSS sections below the roof system. This system allowed the wood roof to 'hang' from the HSS and plates above, as per the original design. A general view of the test setup and the specimen, prior to testing, is shown in Figure 3.

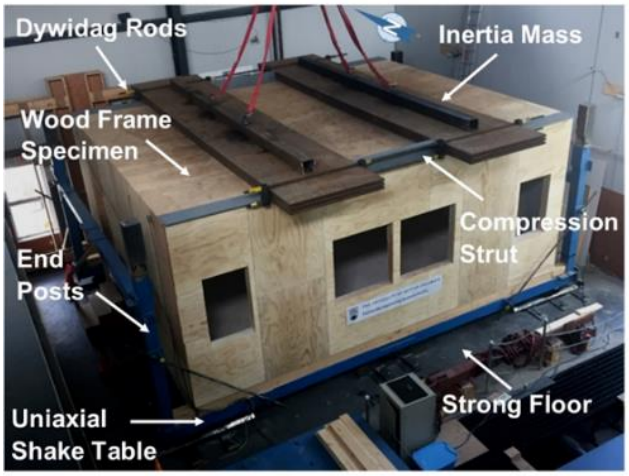


Figure 3. General view of the test setup for shake table testing

The ground motions used for this testing program included subduction and crustal records from SRG3. The details of each record, including the station and relevant parameters are shown in Table 2. The records were then scaled to the Uniform Hazard Spectrum for Vancouver Island, and feature very large displacement at long periods.

Table 2. Selected ground motion records used in testing program.

Earthquake-Station-Direction	Magnitude (M_w)	Distance (km)	Depth (km)	Vs30 (m/s)	PGA (g)	PGV (m/s)	Scale Factor
Tohoku-SIT002-NS	9	180.05	24	351.4	0.11	0.15	3.08
Tohoku-MYG016-NS	9	114.3	24	580.0	0.35	0.30	1.09
Kobe-KAK-090	6.9	22.5	18	312.0	0.34	0.28	1.35
Geiyo-EHM008-EW	6.4	56.0	51	561.0	0.31	0.26	1.44

Observations and Test Results

All specimens experienced damage in shear wall near 2% drift. Failure of the shear wall was localized along the edge panel connections. It was developed by nail pull through and cracking in the sheathings, followed by shear deformation of the nails and separation of the plywood panel from the studs. Also, rocking motion of the panels was observed when separated from the studs. For both configurations the failure mode was found to be similar and independent of the ground motion record. However, specimens with configuration Pattern "B" deformed less than the specimens with Pattern "A" when subjected to the same ground motion. The dominant failure of shear wall panels was nail pull through, where the nail remained attached to the stud but its head was pulled through the sheathing. This observation was found to be consistent with the findings in static cyclic tests performed on wood frame shear walls in SRG-2nd Edition. Nail pull through from stud connection was observed near 4% drift. The end walls in west and east side of the specimens remained with no damage in all tests.

Figure 4 illustrates the general view of the Specimen Sp-1 (with Pattern "A") after Test #3. The deformed shear wall in south-west of the specimen and failure of the connections are shown in Figure 4(a). Figure 4(b) shows the internal damage after the third test run. The result of testing of Specimen Sp-1 confirmed that the shear wall panel reached a near-collapse state at least in 6% drift. The specimen sustained large inelastic deformation. However, it was still standing vertical and supporting the inertial loads. Typical damage in shear wall, nail pull through and separation of the plywood panels from studs were observed in the other specimens testing. Shear deformation and breaking of the nails was also noted.

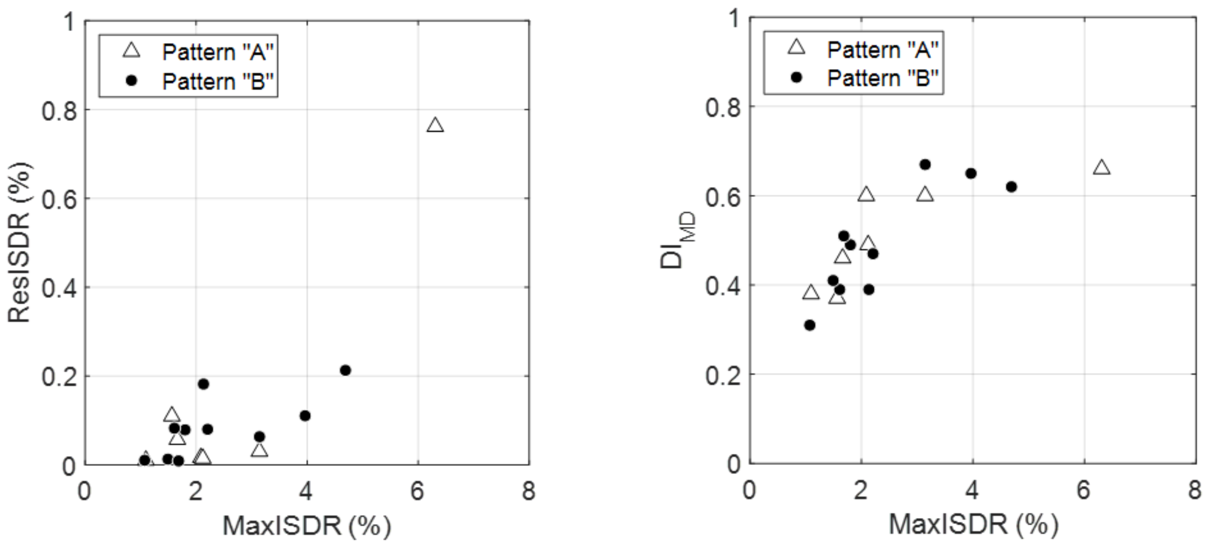
For internal gypsum wall board the failure mode was also found to be independent of the ground motion. The dominant failure mode was found to be tear-through as shown in Figure 4(b). It was observed to have occurred as the nail pushed laterally through the gypsum board resulting in a slotted hole around the nail and no resistance provided by the connection along the gap. Damage was observed to have occurred at both the top and bottom drywall panels. After testing, with only a small push by hand applied out of plane the plywood panels were easily detached from the wood frame.

At the completion of each main test, the three teams performed their inspections. The first team consisted of three UBC graduate students, the second team consisted of the UBC research team and the third team consisted of two engineers from engineering consultant. These inspections were used to verify the post-earthquake evaluation training procedure.

(a) (b)

Figure 4. Damage in Specimen Sp-1 after test #3: a) southwest shear wall, b) internal view of gypsum wall boards.

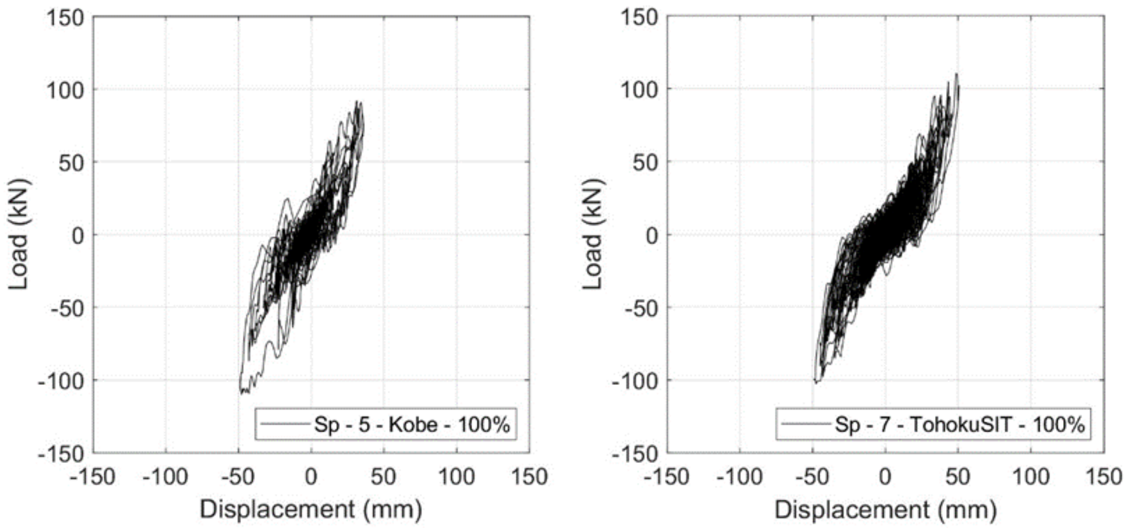
Results of the Maximum Inter-Story Drift Ratio (MaxISDR) and Residual Inter-Story Drift Ratio (ResISDR) for Patterns "A" and "B" are plotted in Figure 5(a). Overall the MaxISDR for each test were in a range from 1% to 5% except for Sp-1 where a 6.3% drift was recorded after two repeated TohokuSIT shaking at 100% intensity. The ResISDR for all the tests fall below 1%. Figure 5(b) summarizes the damage indices (DI_{MD}) for all tests, which are positively correlated with the MaxISDR (up to 3% for pattern "B" specimens). Most of the specimens were at the "Life Safety" performance level, while the last sequence of Sp-1, Sp-4, Sp-5, and Sp-7 fell into "Near Collapse" category with a DI_{MD} of 0.66, 0.65, 0.62, and 0.67, respectively [13].



(a) *(b)*

Figure 5. Maximum inter-story drift ratio for Pattern "A" and Pattern "B" versus: (a) residual drift ratio, and (b) damage index.

Figure 6 then compares the global hysteresis curves (base shear versus roof displacement) of the specimen subject to both Kobe and TohokuSIT motions. Base shear was calculated based on the acceleration measured at top of the specimen and the total mass of the structure including steel plates. Both curves, in general, look similar that the specimen showed distinct nonlinear behavior and clear pinching effect. There was a large impulse from the Kobe motion pushed the specimen to reach the maximum base shear of 110 kN, while during the subduction Tohoku motion, the base shear increased gradually with a large number of reversed cycles [13].



(a) *(b)*

Figure 6. Hysteretic curves: (a) Sp-5 Kobe-100%, (b) Sp-7 TohokuSIT-100%.

TESTING OF STRUCTURAL COMPONENTS CUT FROM AN EXISTING BUILDING

A series of laboratory tests was conducted on two pieces of construction specimens obtained from the Lake Trail Middle School in Courtenay, BC. The specimens were cut from the school building just before the demolishing. These tests were a part of the SRG 2023 Test Program conducted in June 2022. The objectives of these tests were to observe the behavior of the wood frame components and generate test data that provides good insight into the performance of existing wood frame school building components.

Test Specimens

Two construction specimens were determined in a visit carried out from the school building: 1) One piece of roof deck with 3m x 6.1m dimensions consisting of horizontal shiplap sheathing on wood joists; 2) Another piece of stud shear wall with about 3m x 3.2m dimensions with diagonal shiplap sheathing. The specimens were cut from the school building before demolishing, delivered to UBC, and were stored in appropriate conditions until the test day. Detailed description of the test specimens can be seen in Figures 7(a) and 7(b) for the roof deck and shear wall, respectively.

Test Setup

A horizontal large-scale test setup was prepared to test the roof deck specimen. The setup consisted of two fixed reaction steel beams at the ends (north and south sides) of the specimen and two loading steel beams parallel to the longitudinal side of the specimen at the middle of deck span to partially distribute the load along the east side of the specimen. The reaction beams were vertically supported to avoid any possible out-of-plane movement. The loading beams were attached to the specimen and transferred the load generated by the actuator to the sides of the specimen. The reaction beams attached to two supports were used to connect the south side and north side of the test specimen to the strong floor and prevent any movement. HSS 203x203x13mm profile was used for both reaction and loading beams. The roof deck was connected to the reaction and loading beams firmly by several steel bolts. Two hold-down units were used to fix the specimen laterally to the reaction beams to avoid sliding and rocking. The test setup for the roof deck test is shown in Figure 8.

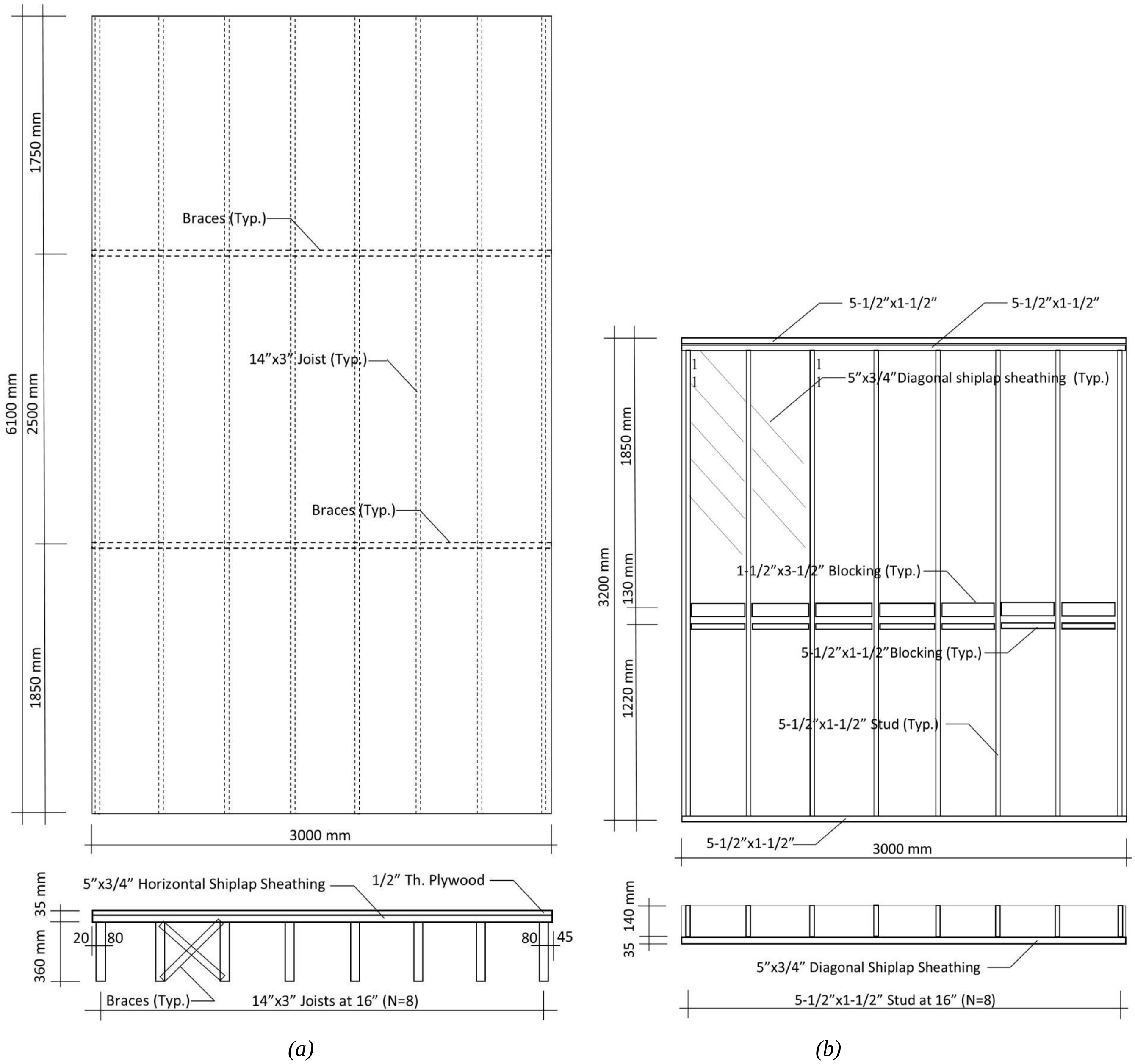


Figure 7. Schematic plan and section view of: (a) Roof deck specimen, (b) Shear wall specimen.

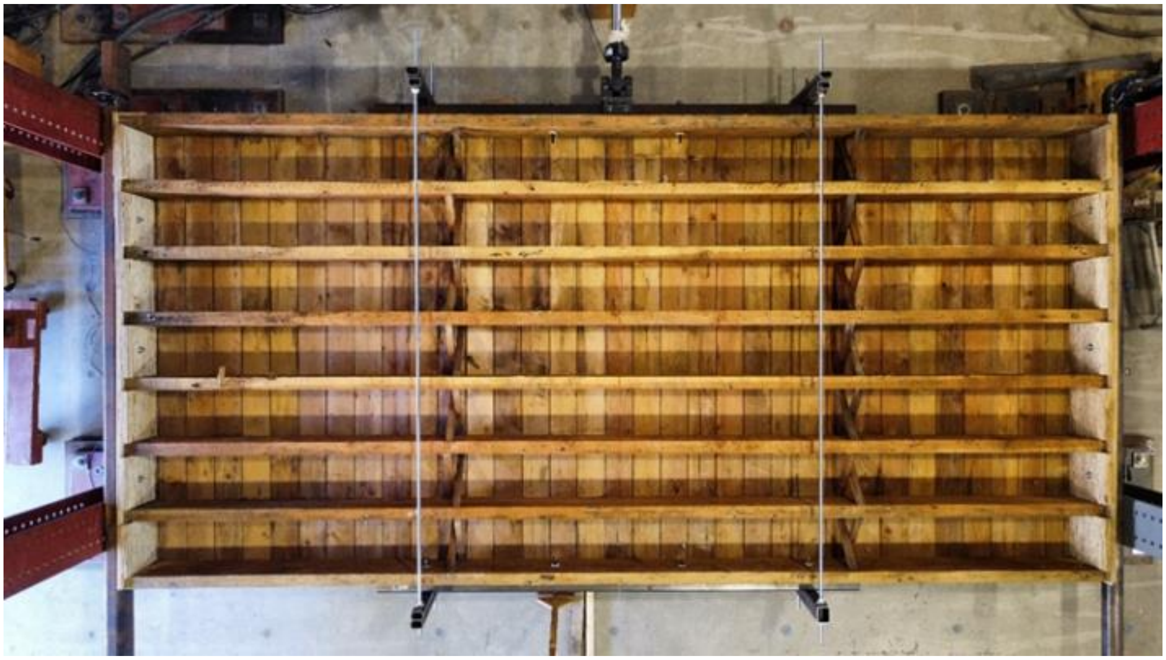


Figure 8. Test setup for the roof deck specimen reversed cyclic testing.

A horizontal large-scale test setup was built to test the shear wall specimen. The setup consisted of a reaction steel frame in one end, which was significantly stiffer than the specimen, a loading beam at the other end, a bearing steel beam at bottom of the specimen and the out-of-plane support system. The loading beam is laterally restrained to avoid any eventual out-of-plane movement. The out-of-plane restraint system consisted of a beam attached to the strong reaction wall equipped with four rollers at the same height as the top loading beam and located on both sides of the loading beam. The loading beam could freely move between the rollers in the north-south direction. The bearing beam attached to the strong floor was used to connect the bottom side of the wall specimen to the strong floor and prevent any movement. HSS 152x152x9.5mm and HSS 152x102x7.9mm profiles were used for bearing and loading beams, respectively. The test specimen was fastened to the test frame using the bearing and loading beams. Seven steel bolts were used to connect the test specimen to the bearing and loading beams firmly. Two hold-downs were used at bottom and top of the specimen at each side to prevent it from uplifting. The test setup for the shear wall test specimen is shown in Figure 9.

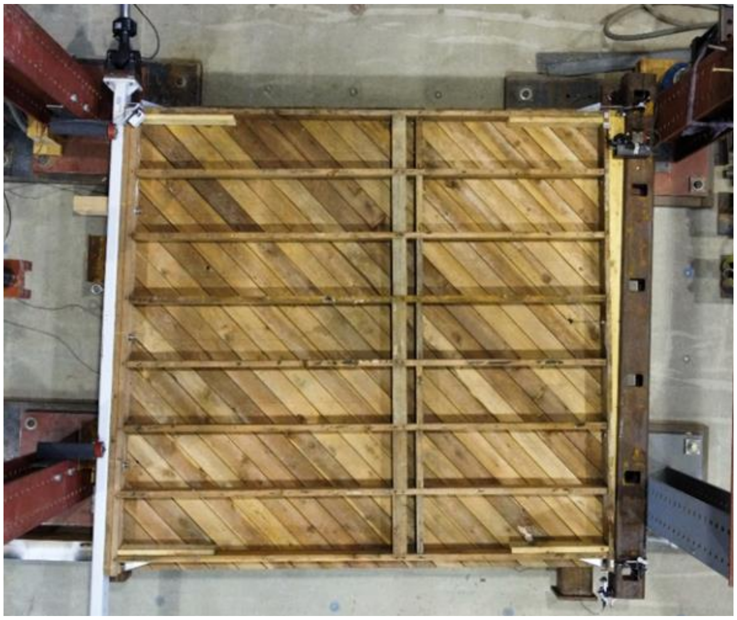
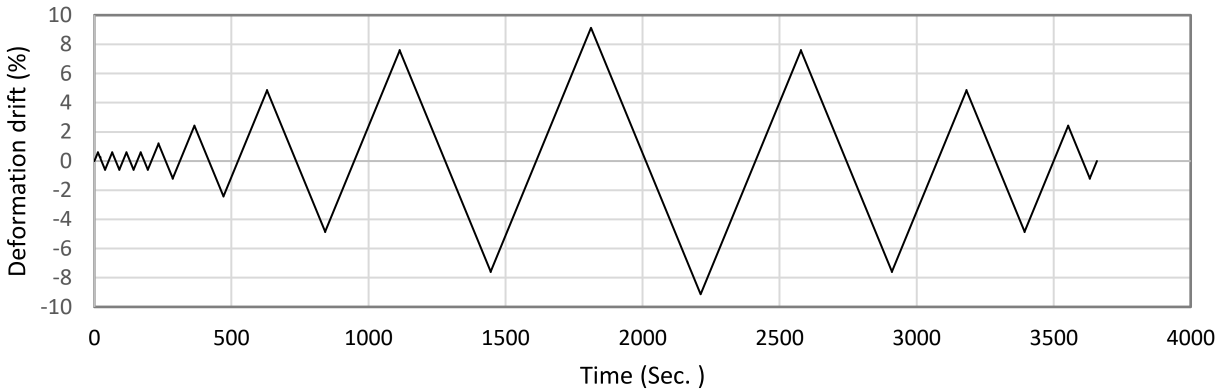
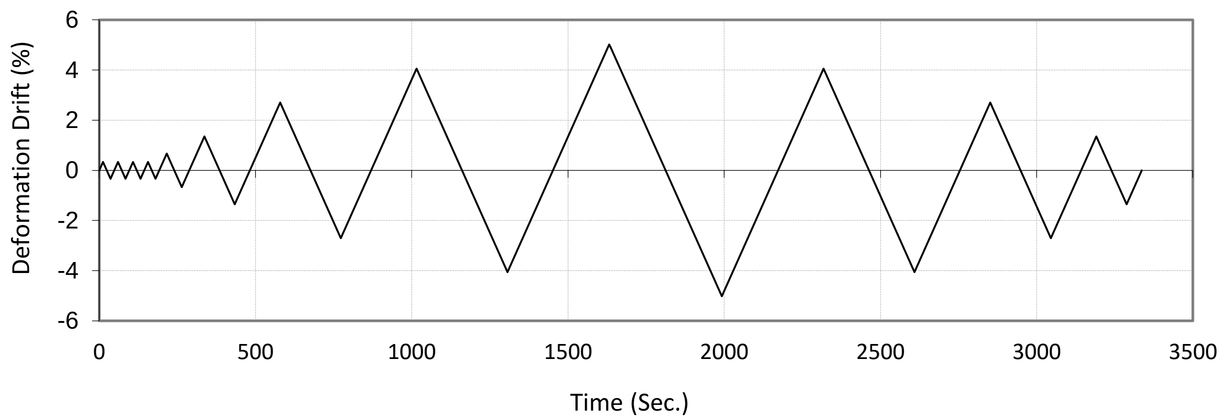


Figure 9. Test setup for the shear wall specimen reversed cyclic testing.

The load was applied by a hydraulic actuator with a two-way capacity of ±200 kN force and a stroke limit of ±300 mm. The actuator was connected to a strong reaction wall and applied load to the loading beam. It was pinned at both ends to rotate freely in the horizontal direction. Two loading protocols were developed for performing reversed cyclic tests on the specimens. These tests were displacement-controlled using a gradually increasing displacement at 1 mm/sec, which allows for a good comparison of the results among the specimens. Figure 10 shows the loading protocols used for cyclic testing of the roof deck and shear wall specimen. The maximum applied drift was 9% and 5% for the roof deck and shear wall specimen, respectively.



(a)



(b)

Figure 10. Loading protocol for: (a) Roof deck specimen, (b) Shear wall specimen.

The load signals were measured by a load-cell mounted to the actuator. The displacement at the location of the loading beam was also measured by the internal linear displacement sensor of the actuator directly. A position transducer was employed to measure the horizontal displacement of the specimens. For controlling the uplift of the shear wall specimen, two LVDTs (Linear Variable Differential Transformer) with a capacity of ±50 mm were installed on the strong floor to capture the vertical displacement in each corner of the specimen at the sides. A position transducer was mounted on the wall specimen at the bottom corner point (south side) to control the unexpected sliding of the specimen against the bearing beam.

Test Results

The behavior of the roof deck specimen during the test was captured and presented in the Figures 11 and 12. Bending action was observed in the roof deck when subjected to significant cyclic loading as shown in Figure 11. Shear deformation of the shiplap sheathing was also noted at the area of close to the end sides as expected (Figure 12). The test could not be completed as the side joist failed when subjected to a cyclic displacement of 200mm. The hysteresis behavior of roof diaphragm when subjected to the loading protocol is shown in Figure 14. It is observed from the results that the hysteresis loops are symmetric and reached a maximum restoring force of 91 KN when subjected to a displacement of 200 mm.

(a) (b)

Figure 11. Deformation of the roof deck specimen and the shiplap sheathing on the joists: (a) General view, (b) Close up.

Figure 12. Deformation of the roof deck joists.

Shear deformation and damage in the diagonal shiplap sheathing was observed in the shear wall specimen in cyclic test as presented in Figure 13. As the load increased, it was observed that the nails connecting shiplap sheathing and studs failed. The connections between the horizontal blocking and the studs also failed. The hysteresis behavior of the shear wall when subjected to the loading protocol is shown in Figure 15. It is observed from the results that the hysteresis loops are not perfectly symmetric.

(a) (b)

Figure 13. Deformation of the shear wall specimen and diagonal shiplap sheathing: (a) General view, (b) Close up.

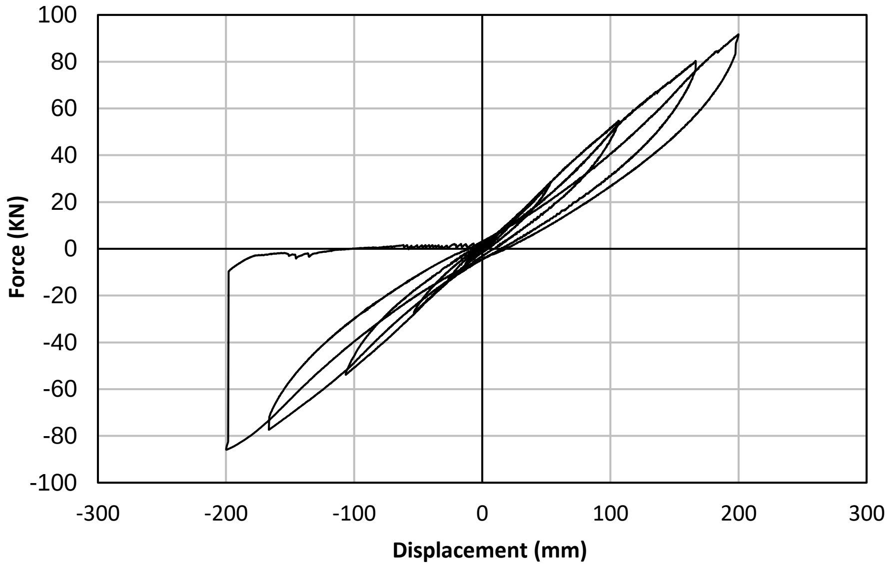


Figure 14. Hysteresis loops of the roof deck specimen.

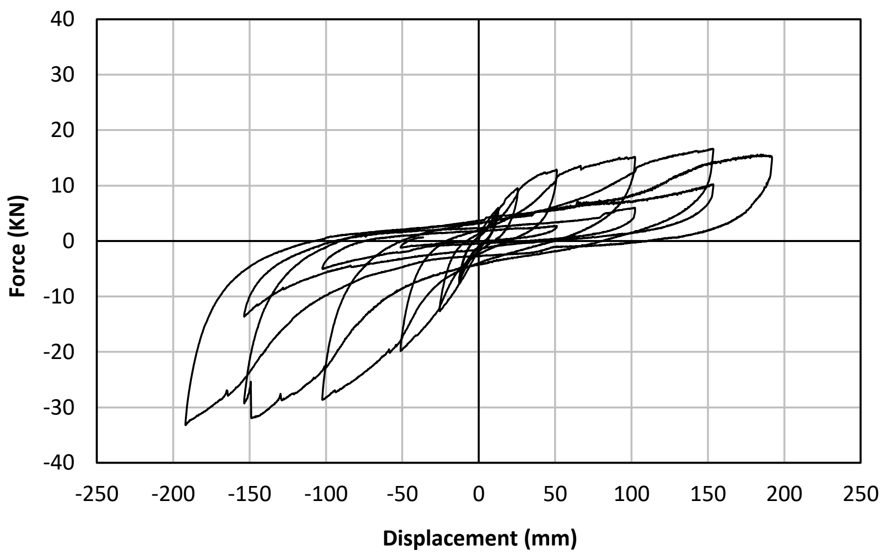


Figure 15. Hysteresis loops of the shear wall specimen.

CONCLUSIONS

This paper presented a series of experimental works which were carried out as part of the Seismic Retrofit Guidelines (SRG) for school buildings in the Province of British Columbia. These experimental works consisted of two phases: 1) a series of shake table tests on a full-scale wood frame; and 2) cyclic quasi-static tests on a wood roof deck diaphragm and shear wall specimens cut from an existing school building in the province. The shake table test results can be used to develop the refined post-earthquake evaluation training procedures and to verify the critical performance-based hypothesis that forms the basis of the SRG. The shake table test results showed that the test structure can achieve life safety performance with minimum base shear capacity equals to 50% of the weight of the structure which is twice the design base shear of a similar building located in the city of Victoria, founded on Site Class C soils and with 5% probability of exceedance in 50 years according to NBCC 2015. The tests clearly demonstrated that the performance of the buildings was governed by the rocking strength of the shear walls. All specimens experienced damage in shear wall near 2% drift and nail pull through from stud connection near 4% drift. The failure mode of internal gypsum wall board was found to be independent of the ground motion. Configuration Pattern “B” deformed less than the specimens with Pattern “A” when subjected to the same ground motion.

The reversed cyclic testing of the structural components generates test data to provide good insight into the performance of structural components of existing timber structures. The test data can be used to calibrate the hypothetical computer models. In the roof deck diaphragm test, a bending action and bending deformation was observed. Nails connecting the sheathing to the joists failed during the test. In the shear wall test, shear deformation and failure at the shiplap sheathing was observed. The hysteresis loops in the shear wall test were not symmetric. This asymmetric behavior could be attributed to the fact that the diagonal sheathing on the shear wall was only in one direction. Bending action of both of the specimens was observed during the tests. Shear deformation of the sheathing panels was also observed in both specimens.

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REFERENCES

[1] Ventura, C., Bebamzadeh, A., Fairhurst, M., Taylor, G., and Finn, W. (2015). "Performance-based retrofit of school buildings in British Columbia, Canada: An update". In *Improving the Seismic Performance of Existing Buildings and Other Structures-ASCE*, San Francisco, CA, USA.

[2] National Research Council of Canada -NRCC (2015). *NBCC: National Building Code of Canada*, Prepared by the NRCC, Ottawa, ON, Canada.

[3] Allen, T., Halchuk, S., Adams, J., and Rogers, G. (2017). "Canada's 5th generation seismic hazard model: 2015 hazard values and future model updates." In *16th World Conference on Earthquake Engineering*, Santiago, Chile.

[4] Ellingwood, B. R., Rosowsky, D. V., and Pang, W. (2008). "Performance of light-frame wood residential construction subjected to earthquakes in regions of moderate seismicity". *Journal of Structural Engineering*, 134(8), 1353-1363.

[5] Filiatrault, A., Fischer, D., Folz, B., and Uang CM. (2002). "Seismic testing of a two-story wood-frame house: influence of wall finish materials". *Journal of Structural Engineering*, 128 (10), 1337-1345.

[6] Filiatrault. A., Christovasilis, I., Wanitkorkul, A., and Van de Lindt, J. (2010). "Experimental seismic response of a full-scale light-frame wood building". *Journal of Structural Engineering,* 135 (3), 246-254.

[7] Pei, S., van de Lindt, J. W., Wehbe, N., and Liu, H. (2013). "Experimental study of collapse limits for wood frame shear walls". *Journal of Structural Engineering*, 139(9), 1489-1497.

[8] Ventura, CE., Taylor, G., Prion, H., Kharrazi, M. (2002). "Full-scale shake table studies of wood-frame residential construction". In *7th US National Conference on Earthquake Engineering*, Boston, MA, USA.

[9] Seim, W., Kramar, M., Pazlar, T., and Vogt, T. (2016). "OSB and GFB as sheathing materials for timber-framed shear walls: Comparative study of seismic resistance". *Journal of Structural Engineering*, 142(4), 401-414.

[10] Goda, K., and Salami, M. R. (2014). "Inelastic seismic demand estimation of wood-frame houses subjected to mainshock-aftershock sequences". *Bulletin of Earthquake Engineering*, 12(2), 855-874.

[11] Ruiz-García, J., and Miranda, E. (2006). "Residual displacement ratios for assessment of existing structures". *Earthquake Engineering & Structural Dynamics*, 35(3), 315-336.

[12] APEGBC. (2017). *Seismic retrofit guidelines, 3rd Edition: Structural engineering guidelines for the performance-based seismic assessment and retrofit of low-rise British Columbia school buildings*. Association of Professional Engineers and Geoscientists, Burnaby, BC, Canada.

[13] Ventura, CE., Motamedi, M., Pan, Y., Tesfamarian, S., and Xiong, H. (2-23), "Drift- and energy-based seismic performance assessment of retrofitted wood frame shear wall buildings: Shake table tests". *Earthquake Engineering and Structural Dynamics*, 52(6), 1844-1860.

[14] Mulder, MJ., (2017). "*Numerical modeling of seismic performance of light-frame wood building*". M.Sc. Thesis, University of British Columbia, Vancouver, BC, Canada.

[15] Canadian Wood Council - CWC. (2010). *Wood Design Manual*. Prepared by the CWC, Ottawa, ON, Canada.