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SHAKE TABLE TESTING OF A FULL-SCALE WOOD FRAME BUILDING CONSIDERING THE GROUND MOTION DURATION

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Abstract

This paper presents the results of a series of shake table tests performed on a full-scaled wood frame building with shear wall at the Earthquake Engineering Research Facility of the University of British Columbia. The test specimen represented typical two-story school buildings constructed in British Columbia according to National Building Code of Canada (NBCC). The objectives of these tests were to determine the seismic responses of the specimen and evaluate the performance of typical school wood frame buildings subjected to short and long duration ground motions. The wood frame specimen 7.7 m long, 6.15 m wide and 3.3 m high was designed and constructed in accordance with NBCC and Canadian Standard Association (CSA), respectively. The lateral seismic resistant system was included wooden diaphragm and shear walls made up of plywood sheets connected to wood studs using nails and covered by drywall inside with symmetric layout in plan. Heavy steel plates at roof replicated the weight of second story of the building. The large shake table at the UBC Earthquake Engineering Research Facility was used for earthquake motions simulation. A suite of short and long duration ground motions included 2011 Tohoku subduction with magnitude of 9 Mw, 2019 Maule earthquake with magnitude of 8.8 Mw and 2003 Hokkaido earthquake with magnitude of 8.3 Mw were used as earthquake excitation. The ground motions were scaled to match an averaged spectral velocity from the Uniform Hazard Spectrum, with a probability of exceedance of 2% in 50 years, for a building located on Site Class C soils in Victoria. The amplitude level of the excitation was increased after each test and test repeated to represent aftershocks. The specimen was then repaired and was prepared for the next ground motion. As results, seismic responses and modal frequency of the structure were presented. The observed damage and behavior of the shear panels subjected to prescribed short and long duration earthquake ground motions were also reported and compared. The results of the dynamic tests demonstrated that the typical school wood frame buildings designed according to NBCC can achieve life safety performance at most twice the minimum code level of shaking for the design of Victoria buildings founded on Site Class C soils. The tests clearly showed that the performance of the building was governed by the rocking strength of the shear walls.

Keywords: shake table test; wood frame building; ground motion; long duration; seismic design

1. Introduction

This paper presents the description and results of a full-scale test program conducted at the University of British Columbia on school wood frame structure between January 2018 and September 2019 as part of the Seismic Retrofit Guidelines Third Edition [1]. The experimental program consisted of series of shake table tests on a full-scale specimen represents a two-story wood frame classroom in either Victoria or Masset. The results from this test program was to provide full scale test data, as well as to use for post-earthquake evaluation training and to verify the critical performance-based hypothesis. They all show that the long duration ground motions have significant effect of the level of damage induced in the structures in near collapse state.

Many researchers conducted similar shake table tests on wood-light frames in past decade to study the seismic performance of such buildings in severe earthquakes [2,3,4]. However, in most of those studies, the specimens were designed according to US Uniform Building Code. There are also several studies on the effect of long duration ground motions on response of the structures in collapse state [5,6].

The objectives of the test program were to provide full scale test data; develop the refined post-earthquake evaluation training procedures; and verify the critical performance-based hypothesis that forms the basis of the SRG Post-earthquake Evaluation Guidelines. Collapse prevention performance of the structure subjected to high value strong ground motion and functionality and reliability of the instrumentation components used in the test program were also evaluated.

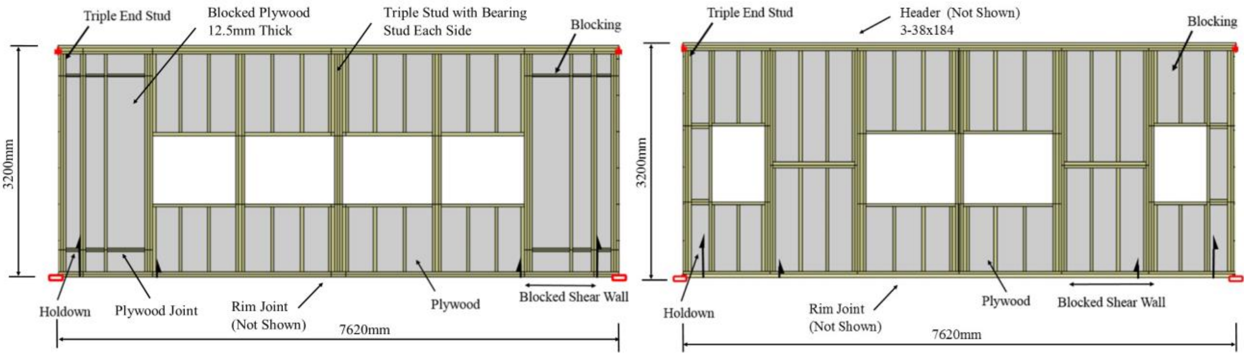
2. Specimen Description

The test specimen was intended to represent a two-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 [7] and Wood Design Manual [8]. 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 HTT5 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 1. 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 amounts 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 [9].

After each test, the test specimen was repaired in preparation for the following test. It is anticipated that replacement of the sheathing on the two exterior walls will be the primary repair. Repairs were made only after all inspections had been conducted.



(a) (b)

Fig. 1- Exterior wall framing and configuration, north and south view: a) Pattern "A", b) Pattern "B" [1]

3. 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 as shown in Figure 2. 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 2.

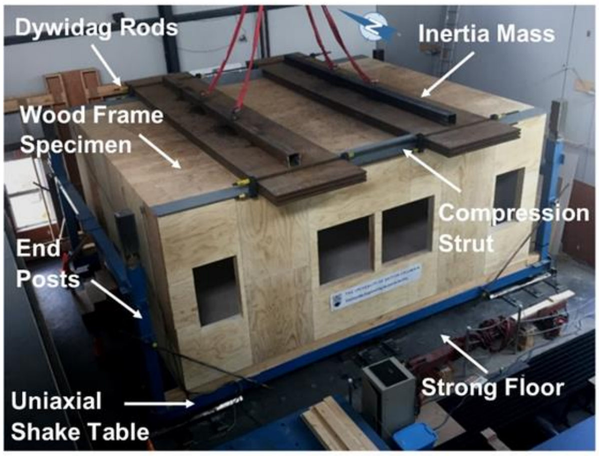


Fig. 2 - General view of the test setup for shake table testing [1]

4. Instrumentation

The test setup had two sets of instrumentation: field sensors and laboratory sensors. 'Field Sensors' refers to what has been installed in the schools in the field (IE. Permanent Strong Motion Monitoring Systems) and 'Laboratory Sensors' refers to typical scientific instrumentation that is used for a test of this nature, to capture all of the appropriate structural responses.

Figure 3 shows the layout of the laboratory and field instrumentations attached to the specimen. The laboratory sensors included two uni-directional accelerometers at either end of the structure, a single accelerometer at the base of the structure on the east end; two displacement sensors at the top of the east end and a single displacement sensor at the bottom of the east end. The accelerometers were uni-directional (one axis) lab accelerometers with +/-5g range. The displacement sensors were string potentiometers with up to 1m extension. Displacement sensors provided relative displacement between the structure and a fixed point on the wall.

The field sensors included three SENSR CX-1 monitors, one at either side of the top West end of the test structure. One sensor was mounted to the floor inside the test structure. Each CX-1 measured three directions of acceleration, and two directions of tilt. The sensor on the North side was mounted to the end wall on the stud, and the sensor on the south side was mounted on the sheathing panel on the outside wall face. These are the actual sensors that have been installed in the schools. In most cases they require a LAN connection, as well as PoE (Power over Ethernet). Data was collected on a laptop for these tests. The intention was to demonstrate the performance of the instrumentation as it would be installed in a school.

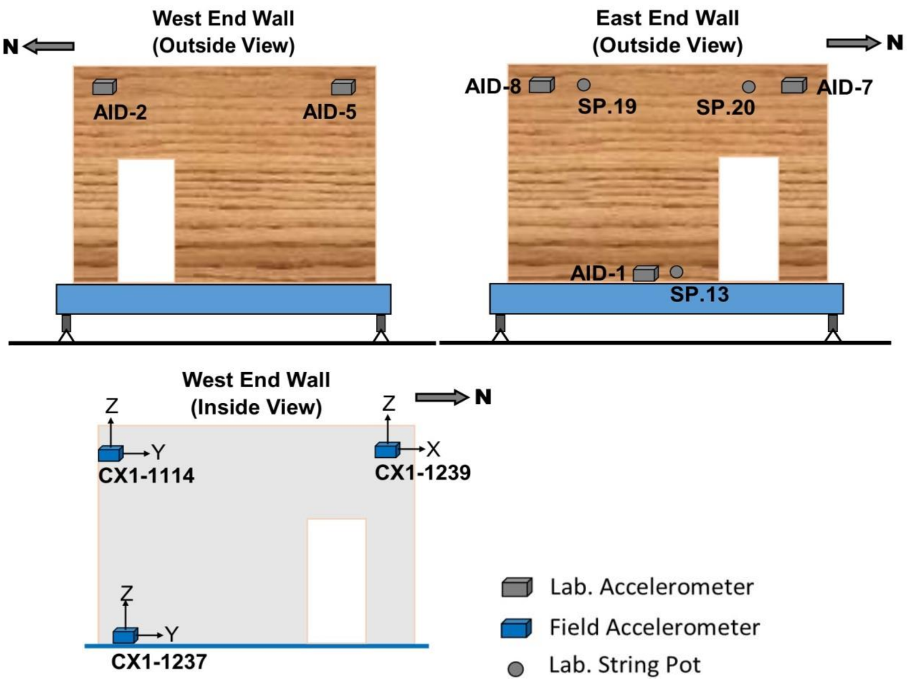


Fig. 3 - Instrumentation layout on the end walls of the specimen [1]

5. Testing Program

A full-scale testing program on a single-storey wood frame school building was performed at the EERF between January 2018 and September 2019. The primary focus of the full-scale test program was for post-earthquake evaluation. For this purpose, the test program included elements of field instrumentation and inspection. Each test day was executed as follows: 1) Run the main shock at full intensity. Test is intended to achieve 3 to 4% drift (damage but not collapse level); 2) Have three separate 15 minute inspections of damaged structure based on ATC-20 rapid inspection, Structural engineers using SRG3 guidelines, and UBC research team using the data from the instrumentation; 3) Have a roundtable to discuss the results of the inspection teams; 4) Run the aftershock; repeat of the first test; 5) Sine sweep tests were conducted before and after each test for the last two specimens. Table 1 describes test specimens (specimen repaired each time called as a separate specimen) and the designed testing program.

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-8	B (Discontinued Studs)	Tohoku MYG	Sp8-1	100 %	29 January 2018
			Sp8-2	100 %	
			Sp8-3	100%	
Sp-9		Tohoku MYG	Sp9-1	100 %	19 July 2018
			Sp9-2	100 %	
			Sp9-3	120%	
Sp-10		Maule	Sp10-1	100 %	30 October 2018
			Sp10-2	100 %	
			Sp10-3	125%	
			Sp10-4	125%	
Sp-11		Hokkaido HKD	Sp11-1	100 %	12 December 2018
			Sp11-2	100 %	
			Sp11-3	140%	
Sp-12		Hokkaido HKD 098	Sp12-1	100%	29 March 2019
			Sp12-2	100 %	
Sp-13	B (Continuous Studs)	Hokkaido HKD 105	Sp13-1	80%	26 September 2019
			Sp13-2	80%	
			Sp13-3	100%	

6. Ground Motion Selections

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. The acceleration, velocity and displacement time histories and response spectrum of the Hokkaido HKD 105 record are shown in Figure 4.

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
Maule, Chile – 2019	8.8	3.1	35	-	0.65	0.45	0.78
Hokkaido HKD – 2003	8.3	161.4	27	-	0.15	0.39	3.08
Hokkaido HKD98-NS - 2003	8.3	103.26	27	654.29	0.37	0.82	0.88
Hokkaido HKD105-EW - 2003	8.3	183.9	27	567.55	0.35	0.85	1.86

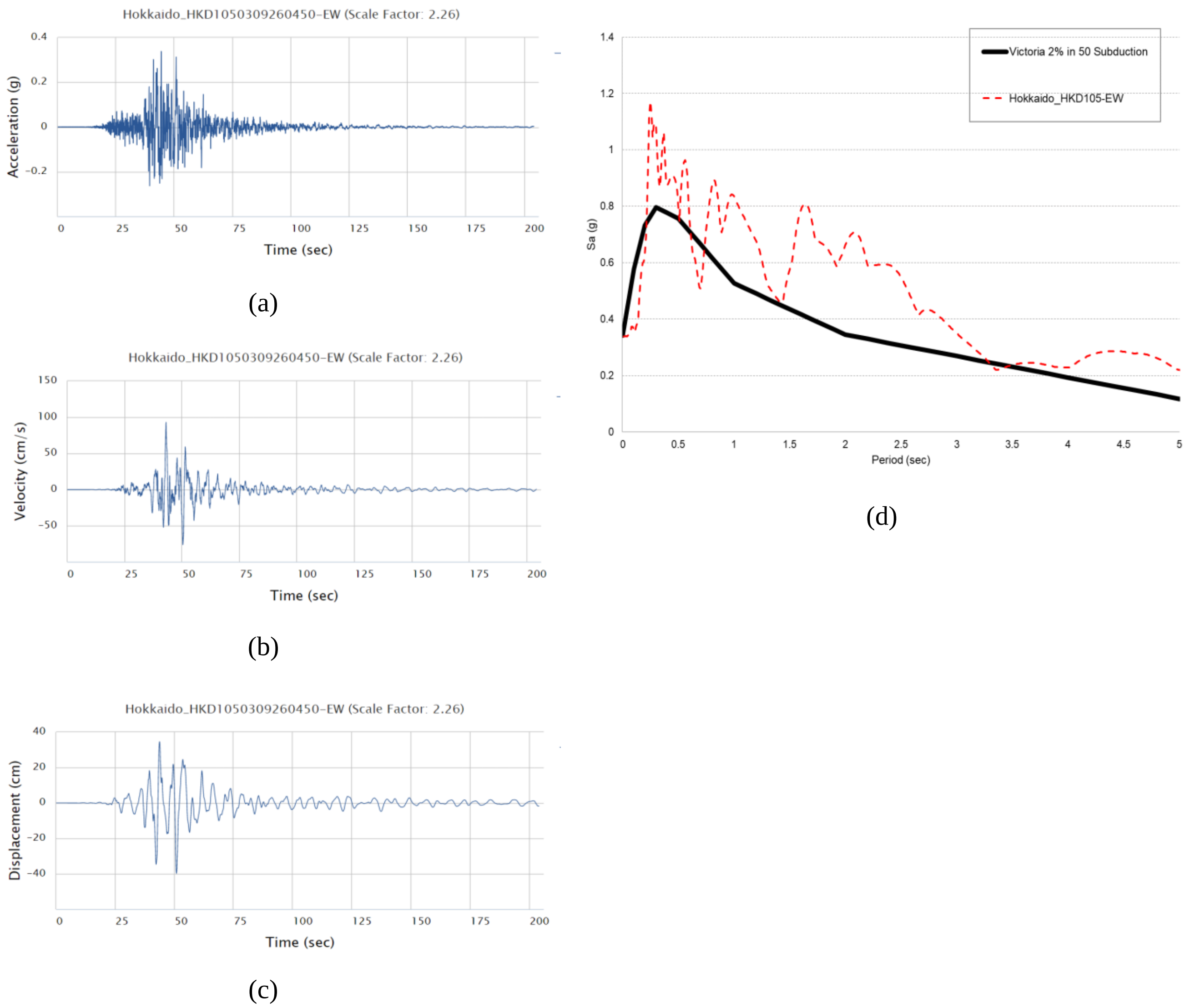


Fig. 4 – Time history records (100% amplitude) and response spectrum of Hokkaido HKD105: a) Acceleration, b) Velocity, c) Displacement, d) Response spectrum matched to design UHS

7. Observations

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 5 illustrates the general view of the Specimen Sp-1 (with Pattern "A") before Test #1 and after Test #3. The deformed shear wall in south-west of the specimen and failure of the connections are shown in Figure 6(a). Figure 6(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 are shown in Figure 7(a). Shear deformation and breaking of the nails are illustrated in Figure 7(b).

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 8. 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.

8. Time History Responses

The results obtained from all seismic tests are summarized in Table 3. The results include the characteristics of the ground motion records as well as the response of the structure such as relative top displacement and the drift ratio.

(a) (b)

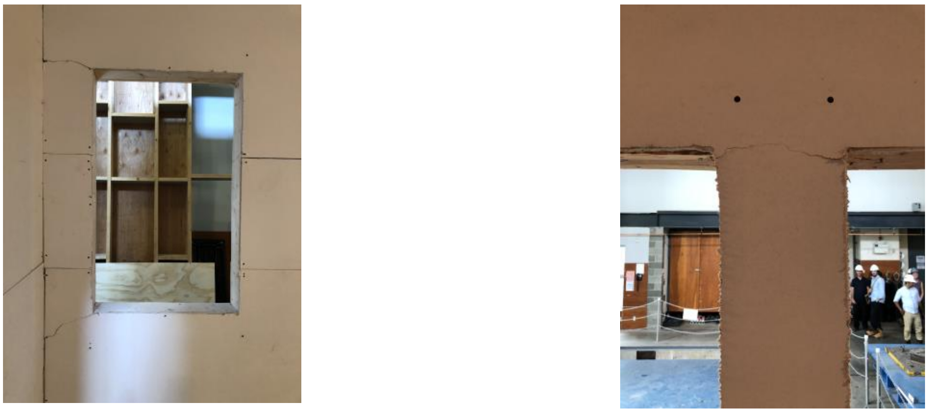
Fig. 5 - Specimen Sp-1: a) Before test #1, b) After test #3 [1]

(a) *(b)*

Fig. 6 - Damage in Specimen Sp-1 after test # 3: a) South-west shear wall, b) Internal view of drywall [1]

(a) (b)

Fig. 7 - Damage in Specimen Sp-10 after test #2: a) South-west shear wall, b) Shear off nail



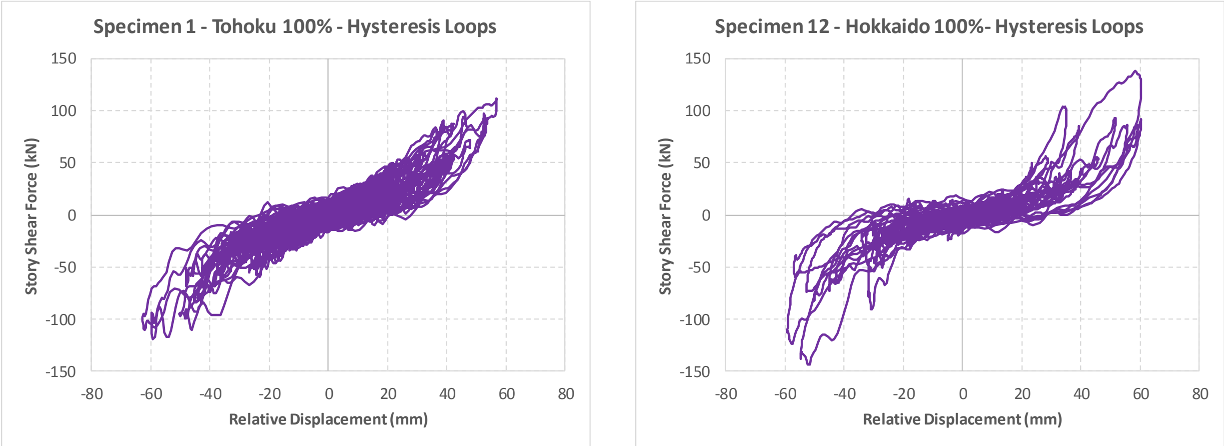
(a) (b)

Fig. 8 - Specimen Sp-9: a) After test #2, b) After test #3

Table 3 - Summary of the results for test responses

Test Specimen	Test No.	Record	Level of Shaking (%)	PGA (g)	PGD (mm)	Relative Top Displacement (mm)	Drift (%)
Sp-1	1	Tohoku SIT	70 %	0.39	62	35	1.11
	2		100 %	0.65	90	67	2.10
	3		100 %	0.70	87	201	6.30
Sp-8	1	Tohoku MYG	100 %	0.70	137	52	1.64
	2		100 %	0.74	139	64	2.00
	3		100%	0.77	170	98	3.05
Sp-9	1	Tohoku MYG	100 %	0.64	130	52	1.63
	2		100 %	0.46	129	62	1.95
	3		120%	0.43	143	75	2.33
Sp-10	1	Maule	100 %	0.74	106	58	1.80
	2		100 %	0.59	104	53	1.67
	3		125%	1.48	127	91	2.83
	4		125%	0.75	127	112	3.50
Sp-11	1	Hokkaido HKD	100 %	0.53	121	19	0.59
	2		100 %	0.53	120	23	0.72
	3		140 %	0.80	169	43	1.34
Sp-12	1	Hokkaido HKD 098	100%	0.75	75	42	1.31
	2		100 %	0.72	85	62	1.94
Sp-13	1	Hokkaido HKD 105	80%	0.60	76	44	1.38
	2		80%	0.54	75	44	1.38
	3		100%	0.67	97	85	2.66

Figure 9 shows the load-deformation hysteretic loops for Specimen Sp-1 and Specimen Sp-12 under Tohoku SIT and Hokkaido HKD098, respectively.



(a) (b)

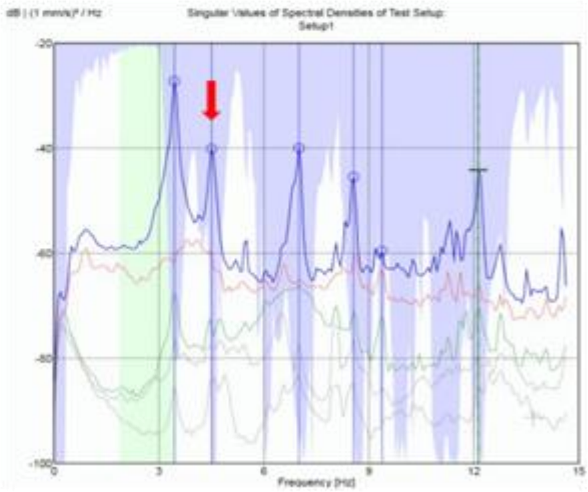
Fig. 9 – Load-deformation hysteresis loops: a) Specimen Sp-1, b) Specimen Sp-12

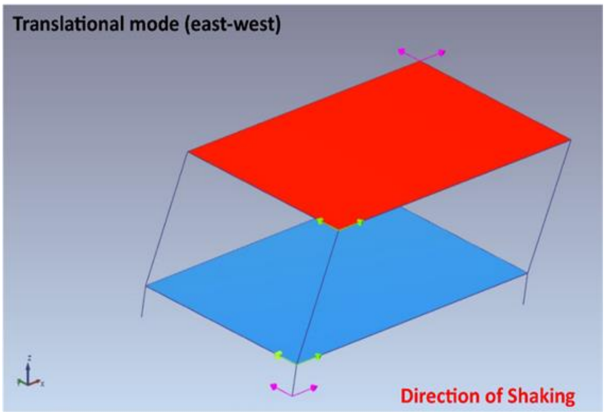
9. Modal Responses

A series of Ambient Vibration Tests (AVT) conducted on the specimen to determine the dynamic properties of the Wood frame structure. Two series of sensors, wireless seismometers and laser vibrometers were used to measure the ambient vibration of the specimen before and after the main shock and after the sequent aftershocks. Modal response analysis was performed to identify the dynamic properties of the specimen [10]. As the results, the natural frequencies, damping ratios and mode shapes of the specimen before and after each shake table test were presented. Figure 10 shows the sensors used for ambient vibration tests on Specimen Sp-12 before and after Shake Table tests. The AVT results, including natural frequencies and damping ratios for this specimen (in low amplitude vibrations) are illustrated in Figure 11. These values are also presented in Table 4.

(a) (b)

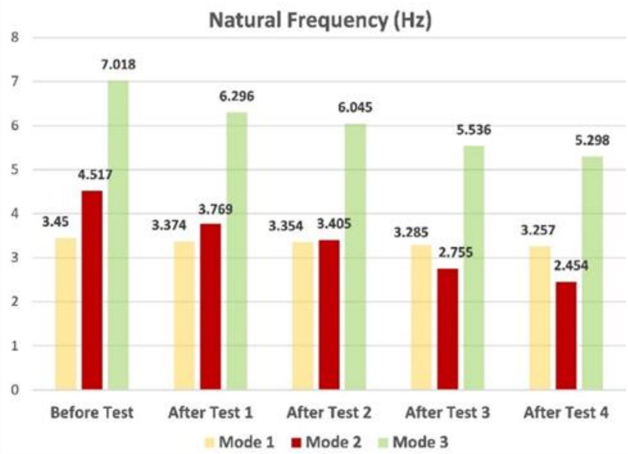
Fig. 10 – Sensors used for Ambient Vibration Tests on Specimen Sp-12 before and after each test: a) Laser Vibrometer, b) Wireless Vibrometers [10]

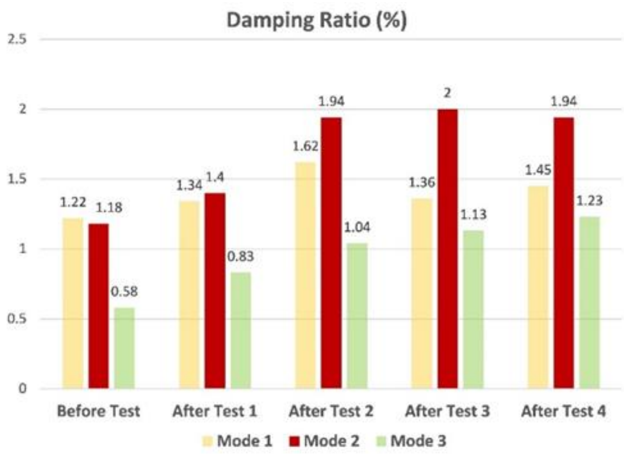




(a)

(b)





(c)

(d)

Fig. 11 – Ambient Vibration Test results for Specimen Sp-12 before and after each Shake Table test: a) Singular Values of Spectral Densities, b) Mode Shape in the direction of the shaking, c) Natural frequencies, d) Damping ratios [10]

Table 4 – Modal response of Specimen Sp-12 in the direction of shaking

	Before Test 1	After Test 1	After Test 2	After Test 3
Frequency (Hz)	4.427	3.617	3.476	2.970
Damping Ratio (%)	1.248	1.772	1.3	1.657

10. Conclusions

An experimental study was performed on light-wood frame buildings using large scale shake table of the Earthquake Engineering Research Facility of the UBC. The scope of this study was limited to low-rise buildings with wood shear walls located in Victoria, BC and designed according to NBCC. The results of the dynamic tests demonstrated that the typical school wood frame buildings designed according to NBCC can achieve life safety performance at most twice the minimum code level of shaking for the design of Victoria buildings founded on Site Class C soils. The tests clearly showed that the performance of the building was governed by the rocking strength of the shear walls.

11. Acknowledgements

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