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DYNAMIC TESTING AND ANALYSES OF WOOD SHEATHED / CFS FRAMED SHEAR WALLS

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ABSTRACT

Seismic design provisions for cold-formed steel (CFS) lateral framing systems are not available in the 2005 National Building Code of Canada (NBCC) or in the CSA S136 Standard. In order to provide Canadian engineers with seismic design information for wood sheathed / CFS framed shear walls the AISI S213 North American Lateral Design Standard was expanded in 2007. Design information consisted of detailing requirements, nominal shear resistances, resistance factors, overstrength factors, as well as R-values and a height limit for wood sheathed and combined wood/gypsum sheathed CFS framed shear walls. To justify the listed R_d and R_o values and height limits it is necessary to investigate the performance of representative shear walls by means of dynamic shake table tests. The intent is first to use the results of the dynamic tests to validate and improve the accuracy of the response provided by numerical models and second to monitor the behaviour of the walls in order to identify whether the assumptions made in modeling, especially at the floor connections, are appropriate. The scope of testing included five full scale specimens; three single-storey and two double-storey walls, which were constructed of a typical CFS frame sheathed with either Douglas fir plywood or Canadian softwood plywood and gypsum panels. The walls were designed and detailed as per the Canadian provisions in AISI S213. Each wall was subjected to a suite of excitations; impact test to measure the linear-viscous damping ratio, harmonic excitation to estimate the natural period of vibration, and ground motions representative of the seismic hazard in Quebec. This paper presents the results of the dynamic testing as well as preliminary comparisons with the predictions obtained from dynamic models created in OpenSees.

Introduction

Seismic design provisions for cold-formed steel (CFS) lateral force resisting systems are not available in the National Building Code of Canada (NBCC) (NRCC 2005) or in the CSA S136 CFS Design Standard (2007). In order to propose the addition of new seismic lateral systems to the NBCC it is necessary to carry out static, reversed cyclic and dynamic testing, as well as dynamic analyses of representative buildings to validate the performance and probability of failure under ground motion excitation. This paper addresses the dynamic testing of wood

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sheathed / CFS framed shear walls and a preliminary comparison with numerical models. Prior to the shake table testing described herein over 180 single-storey assembly wood sheathed CFS shear walls were subjected to displacement based monotonic and reversed cyclic protocols (Branston et al. 2006a; Chen et al. 2006; Hikita & Rogers 2007). A design method was proposed by Branston et al. (2006b) which utilizes the equivalent energy elastic plastic (EEEP) data analysis method. "Test-based" R-values obtained from the measured ductility and overstrength of the wall specimens were recommended (Boudreault et al. 2007). This method and the corresponding nominal shear resistance values were then included in the Canadian section of the American Iron and Steel Institute (AISI) S213 North American lateral design standard for cold-formed steel (2007). A limited number of dynamic non-linear time history analyses of multi-storey building models subjected to earthquake ground motion records were then carried out to evaluate the inelastic performance of wood sheathed / CFS framed lateral systems in terms of the probability of collapse as per the FEMA P695 (2009) analysis procedure (Morello 2009). However, the force vs. deformation hysteretic behaviour of the wall elements in these multi-storey models was calibrated using only the response measured from the single-storey displacement based reversed cyclic shear wall tests; furthermore, simplifying assumptions regarding the floor framing were made.

It was necessary to improve upon these models by incorporating the results of dynamic tests on single and multi-storey shear walls in the calibration and modeling procedures. It was also important to physically validate the assumed inelastic performance of wood sheathed / CFS framed shear walls when subjected to realistic ground motions. This paper contains a description of the response of five full scale wall specimens, which were constructed of a typical CFS frame sheathed with either Douglas fir plywood or Canadian softwood plywood and gypsum panels. Each wall was subjected to impact tests to measure the linear-viscous damping ratio, harmonic excitation to estimate the natural period of vibration and ground motions representative of the seismic hazard in Quebec. This paper also highlights a preliminary comparison of the predictions obtained from dynamic models created in OpenSees (McKenna et al. 2006) with the measured performance of the shear walls.

Shake Table (Dynamic) Test Program

Configuration of Shear Walls

The scope of testing included five full-scale specimens: three single-storey and two double-storey walls (Table 1) (Figure 1). Three different shear wall configurations were used for the 1.22 m × 2.44 m single-storey walls; Douglas fir plywood (DFP) sheathed shear walls, Canadian softwood plywood (CSP) sheathed shear walls, and a shear wall that combined both gypsum and CSP panels. The two 1.22 × 5.18 m storey test walls were sheathed with DFP or CSP panels. The wood sheathing and gypsum panel were connected to the CFS framing members with No. 8 × 38.1 mm self-drilling bugle head screws and No. 6 × 25.4 mm bugle head drywall screws, respectively. The CFS frame was designed according to the capacity method outlined in AISI S213. It was assumed that the full capacity of the wall would be mobilized during shake table testing. Back-to-back studs connected with two No.10 × 19.1 mm self-drilling hex head screws at 300 mm o/c were relied on to transfer compression and tension forces at the chord (end wall) locations. Simpson Strong-Tie S/HD10S hold-downs were fastened to the chord studs with 24 No. 10 × 25.4 mm hex head self-drilling screws and to the test frame with a 22 mm diameter Grade A193 B7 anchor rod. Connections between the track and stud members were made with No. 8 x 12.5 mm self-drilling wafer head screws. The floor assembly consisted of a 305 mm × 96.8 mm CFS rim joist (1.73 mm) supporting three floor joists 304 mm × 41.3 mm × 12.7 mm (1.73 mm) on which a 12.5 mm plywood panel was attached. The joists were reinforced with 92.1 mm × 41.3 mm × 12.7 mm bearing stiffeners at the stud locations. The connection from the wall track to the rim joist was made with 19.1 mm diameter A325 shear anchors. The hold-downs at the top of the lower wall segment were connected to the hold-downs at the base of the upper wall segment using 22 mm threaded rods that passed through the floor structure. Typical details of the test walls are provided in Figure 2.

Table 1. Wood panel shear wall configurations for shake table tests.

Specimen ID	Size (m)	Sheathing & Fastener Spacing (mm)	Nominal Shear Strength (kN/m)	Probable Shear Strength (kN/m)	Double-chord Stud Dimensions (mm)	Track Dimensions (mm)
ST1-1	1.22×2.44	12.5 mm DFP (75/100)	22.1	29.4	1.37 x 92.1 x 41.3 x 12.7	1.37 x 92.1 x 31.8
ST1-2	1.22×2.44	12.5 mm CSP 100/300	13.0	18.9	1.09 x 92.1 x 41.3 x 12.7	1.09 x 92.1 x 31.8
ST1-3	1.22×2.44	12.5 mm CSP & 12.5 mm gypsum (100/300)	13.0	18.9	1.09 x 92.1 x 41.3 x 12.7	1.09 x 92.1 x 31.8
ST2-1	1.22×5.18	12.5 mm DFP (75/300)	22.1	29.4	1.73 x 92.1 x 41.3 x 12.7	1.73 x 92.1 x 31.8
		12.5 mm DFP (150/300)	11.6	15.4	1.09 x 92.1 x 41.3 x 12.7	1.09 x 92.1 x 31.8
ST2-2	1.22×5.18	12.5 mm CSP (100/300)	13.0	18.9	1.37 x 92.1 x 41.3 x 12.7	1.37 x 92.1 x 31.8
		12.5 mm CSP (150/300)	9.5	13.8	1.09 x 92.1 x 41.3 x 12.7	1.09 x 92.1 x 31.8

1.09 mm thick CFS members – 230 MPa Grade; all other CFS members – 345 MPa Grade

The test walls were installed on a shake table (Figure 3) and then connected to steel plates with a weight of approximately 30 kN at each storey. These plates, which represented the seismic weight, were supported on four steel column segments. Cylindrical rockers were placed between the steel plates and the column ends such that the gravity load system had no lateral resistance or stiffness; the lateral loads and P-Δ effects were entirely resisted by the test specimen. In order to minimize the loads on the linear bearings supporting the shake table the seismic weight/gravity load system was constructed on the laboratory strong floor in line with the table. At all floors, horizontal steel struts with high axial stiffness link the steel plates (seismic weight) to the test specimen. Load cells were connected between the struts and the test specimen to monitor the floor inertia forces developing during the tests. At the base of the gravity load system, the steel columns were pin-supported on longitudinal horizontal steel members that extended up to the earthquake simulator to which they were connected. These members were vertically mounted on frictionless roller bearings that roll on polished and levelled high strength steel plates placed on the laboratory strong floor. This arrangement allowed for the base of the gravity system to experience the same horizontal displacement as the

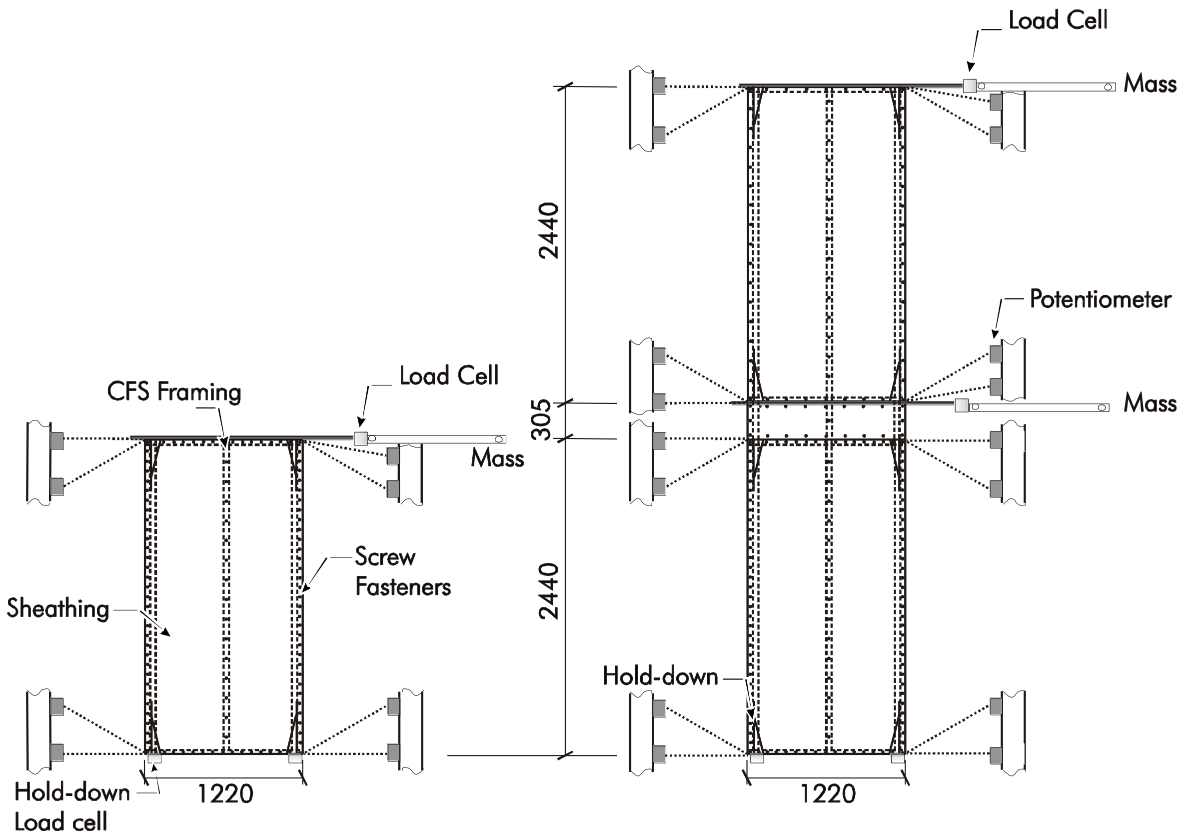


Figure 1. Single and double storey shear wall test specimens (mm).


Figure 2. Typical details of shear wall construction.

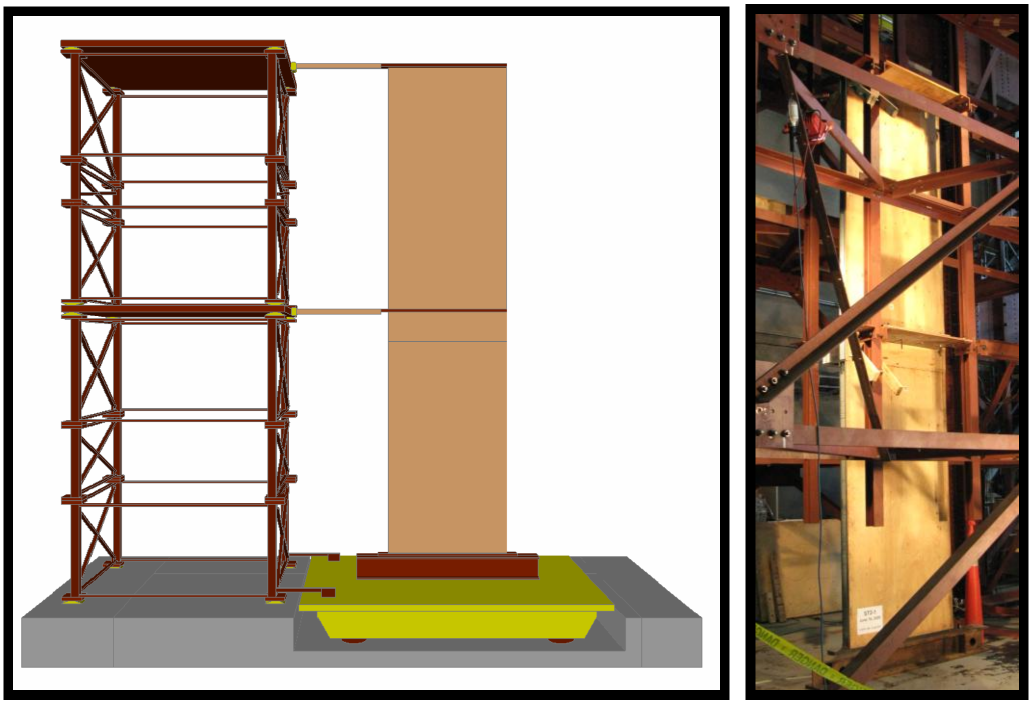


Figure 3. Two storey wood sheathed / CFS framed shear wall dynamic test assembly.

earthquake simulator. The test specimen was thus subjected to P-Δ forces induced by the lateral displacements of the specimen relative to the ground, as is the case in actual buildings. The test walls and seismic weight/gravity system were enclosed in a braced steel frame that provided for out-of-plane stability through the use of lateral roller supports (Figure 2 & 3) and prevented collapse of the test assembly in case of failure.

Two string potentiometers were positioned, one horizontal and one inclined, at the top and bottom of each wall segment in order to determine the horizontal and vertical displacement at these points using triangulation (Figure 1). Another string potentiometer was installed to measure the horizontal displacement of the shake table. Lateral forces were measured through the use of load cells on the links connected to the seismic weights, as well as to the base support structure of the pin ended columns. Load cells were also installed on the hold-downs located at the base of each wall (lower wall segment for the two storey specimens). Finally, an accelerometer was attached at the top of each floor, in addition to the accelerometers placed on the shake table.

Test Protocols

A suit of dynamic tests was carried out on each wall specimen prior to performing the main dynamic test, or full earthquake acceleration time history record. Each wall was subjected to impact tests (free vibration) to measure the linear-viscous damping ratio, harmonic excitation to estimate the natural period of vibration and ground motions representative of the seismic hazard in Quebec. A simulated seismic time history representative of eastern Canada was used (Figure 4); it corresponds to an M7.0 event at 70 km in Quebec City (Atkinson 2009). The signal was modified using an FFT approach to closely match the 2005 NBCC elastic uniform hazard spectrum (UHS) for Quebec used in design. Several tests were performed on each wall for which the amplitude of the earthquake ground motion was systematically increased. The Equivalent Static Force Procedure (ESFP) was deemed appropriate to calculate the expected earthquake force. The fundamental period of the structure was computed as per the NBCC empirical equation specified for shear walls. Seismic force reduction factors, R_d and R_o equal to 2.5 and 1.7, respectively, were chosen according to AISI S213. The earthquake intensity that could mobilize the entire capacity of each respective wall specimen was then determined as outlined in Table 2. The performance of each wall was predicted prior to testing by applying the various earthquake records to dynamic models created in OpenSees.

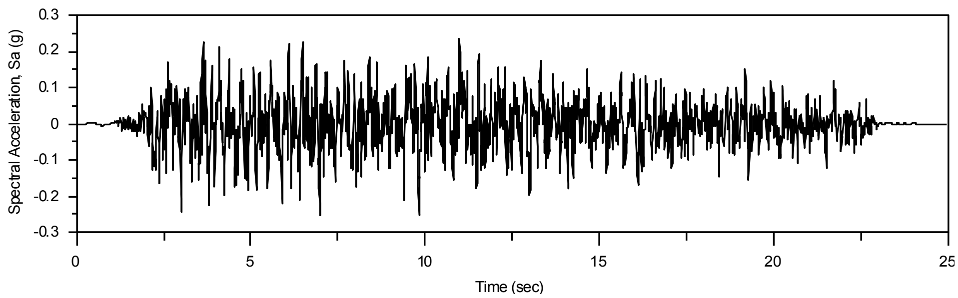


Figure 4. Earthquake record closely matched to UHS for Quebec City.

Table 2. Earthquake amplification factor computation according to EFSP approach.

Specimen ID	**Height (m)**	**T_a[1] (sec)**	**C[2]**	**Storey Shear[3] (kN)**	**Probable Shear Strength (kN/m)**	**Intensifying Factor[4] %**	**Selected Factor for Test (After OpenSees Analysis)**
ST1-1	2.44	0.098	0.139 (>0.093)	3.1	29.4	558	340%
ST1-2	2.44	0.098	0.139 (>0.093)	3.1	18.9	359	400%
ST1-3	2.44	0.098	0.139 (>0.093)	3.1	18.9	359	400%
ST2-1	1st 2.44	0.172	0.139 (>0.093)	6.4	29.4	270	400%
	2nd 5.18			4.25	15.4	213	
ST2-2	1st 2.44	0.172	0.139 (>0.093)	6.4	18.9	174	270%
	2nd 5.18			4.25	13.8	191	

[1] $T_a = 0.05h_n^{3/4}$ [2] $C = S(T_a)\, M_v I_e / R_d R_o \leq 2/3\; S(0.2) I_e / R_d R_o$ [3] Story Shear = C×mass [4] Intensity (Amplification) Factor = 100×Probable Shear Strength / (Storey Shear×R_o)

Test Results

In general, the failure modes of the dynamic test specimens were similar to those observed during displacement based tests by Branston et al. (2006a) and Chen et al. (2006). The CFS frame was subject to minor inelastic damage; whereas, the wood sheathing and gypsum panel suffered from bearing, fastener pull through and plug shear failures at the connection locations, along with screw tilting (Figure 5). This connection damage was typically distributed throughout a large proportion of a panel. The two storey specimen ST2-1 was observed to have connection damage mainly in the upper storey, likely due to the change in screw spacing from 75 mm to 150 mm from bottom to top storey. In contrast, specimen ST2-2 showed most of its damage to occur in the lower storey. The change in screw spacing and shear resistance of the lower to upper panels was less severe than for wall ST2-1.

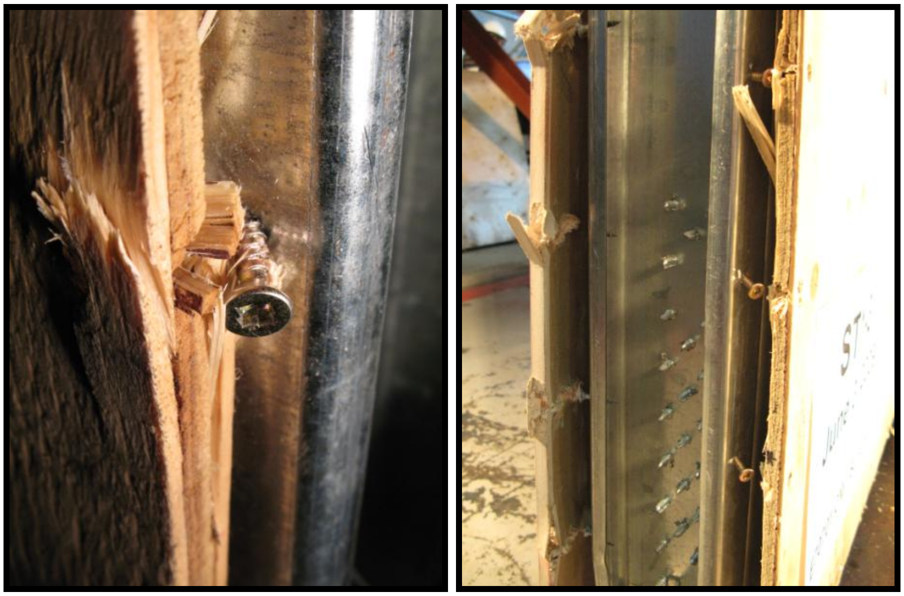


Figure 5. Typical connection based shear wall failure.

Table 3. Linear viscous damping ratio and natural frequency of vibration

Type of Method	Specimen ID						
	One Storey Wall			Two Storey Wall			
	ST1-1	ST1-2	ST1-3	ST2-1		ST2-2	
				1st mode	2nd mode	1st mode	2nd mode
Half-Power Bandwidth (% ζ)	1.5	4.0	3.2	5.2	5.5	4.3	3.7
Resonant Frequency (% ζ)	5.36	7.8	9.12	3.9	7.5	3.7	8.84
Decay of Motion (% ζ)	10.18	12.6	12.74	6.48	7.06	5.1	3.45
Frequency (Hz) (Harmonic Force Vibration Test)	2.8 (3.02[1])	3.1 (3.14[1])	3.7 (3.88[1])	1.65 (1.82[1])	4.6 (4.29[1])	1.5 (1.53[1])	4.1 (3.61[1])

[1] $f_n = \sqrt{\frac{k}{m}}$, stiffness , k, is from harmonic force vibration and 50% EQ test

The viscous damping ratio (VDR) was obtained by employing the half-power bandwidth method, decay of motion method and by computing the response amplitude at resonant frequency of a wall using the results of the impact tests. In the case where the impact test was not carried out, the free vibration portion of the harmonic forced vibration test was instead used. To estimate the VDR of the two storey walls the modal displacement was required to be extracted from the measured displacements. Table 3 presents the viscous damping ratios and natural period of vibrations for all test walls. The measured frequency results are in reasonably good agreement with the frequency as calculated using the measured stiffness and mass. The empirical estimate of the frequency ($1/T_a$), however, provides values that are much higher than measured; 10.2 Hz and 5.8 Hz for the single- and double-storey walls, respectively. This result is likely due to the fact that the one and two storey walls were constructed of light framing materials, a bare frame and sheathing, while the code based expression for T_a of shear walls is intended for use in taller reinforced concrete structures. In addition, non-structural components were not part of the test wall construction. The damping ratios provided by the three methods vary; evaluation of these values and recommendations as to what should be used in modeling are ongoing.

Figure 6 presents a comparison of the total drift at the 1st and 2nd floor of test specimen ST2-2 while undergoing the 270% earthquake record during which failure was expected. The wall tended to respond in the first mode, where the total drift of the upper storey always exceeded that of the lower. As expected, the storey drift measured at the 1st floor exceeded that obtained at the 2nd floor. The force levels for each of the shear walls in ST2-2 are also presented (Figure 7).

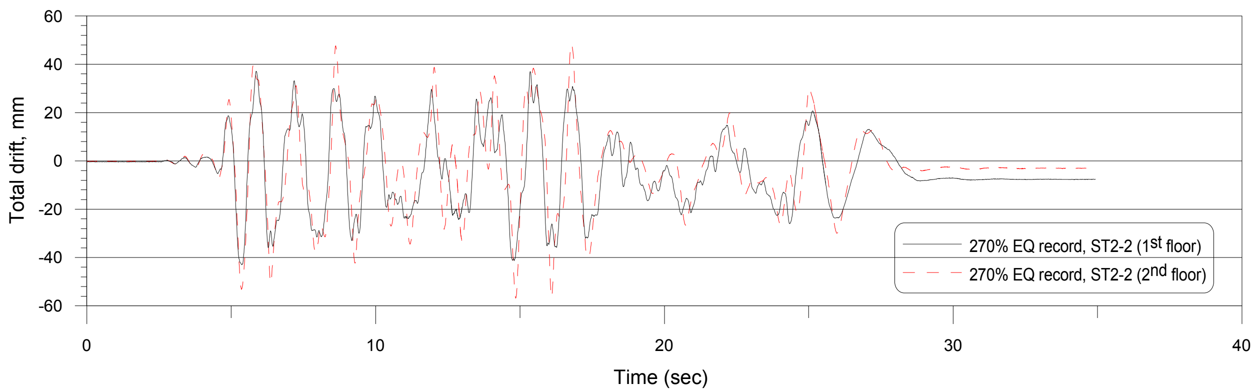


Figure 6. Comparison of total drifts of ST2-2 test specimen (270% EQ record).

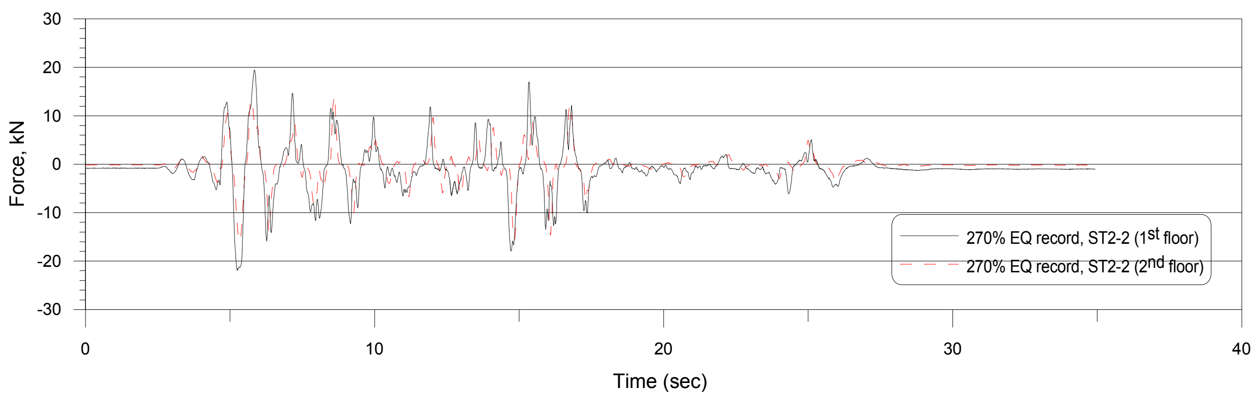


Figure 7. Comparison of inertia forces of ST2-2 test specimen (270% EQ record).

Modeling

Preliminary dynamic nonlinear time-history analyses were carried out to predict the shear resistance vs. drift behaviour of the tested walls. OpenSees (McKenna et al. 2006) was used since it could model the strength degradation of such walls. A truss model (Fig. 8) with an attached gravity column (to account for the P-Δ effects) was developed in which the Pinching4 element was used to represent the force vs. displacement hysteresis behaviour of the wood panel shear walls; this type of material comprises a suite of factors to identify the positive and negative backbones, types of degradations (strength, reloading, and unloading degradation), amount of energy dissipation under cyclic loading, and finally damage type. The columns, beams and P-Δ frame were rigid truss elements. To calibrate the model, results of the preceding reversed cyclic tests were utilized; these tests had been carried out on the same wall configurations as those used in shake table tests. With respect to damping, as an initial trial, the damping ratio was selected to be in the range listed in Table 3 and then corrected to 6% by trial and error based on the results of the 50% earthquake dynamic test. Figure 9 contains a comparison between the test results and OpenSees model results for two representative tests. These preliminary results show promising compatibility between the measured and predicted wall performance, however it has since been observed that the critical damping ratio decreases as the wall enters into the inelastic range. Further study regarding this issue is ongoing.

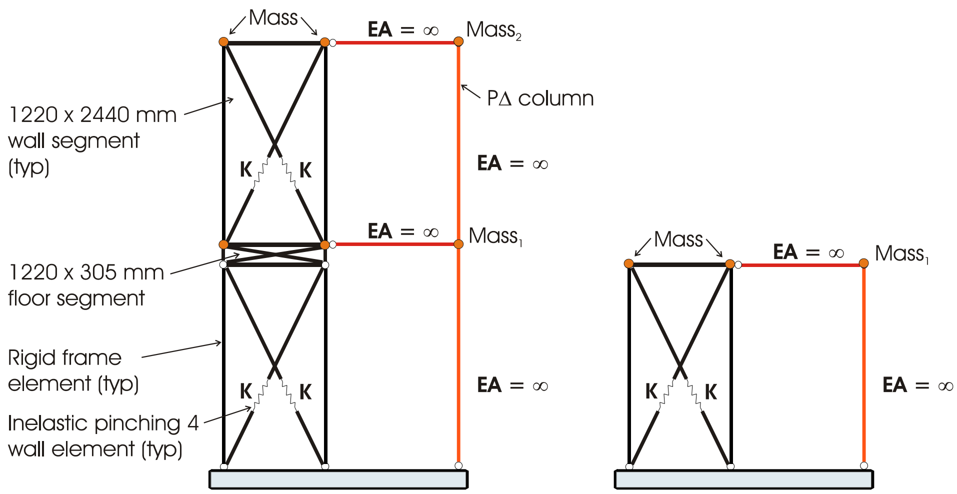
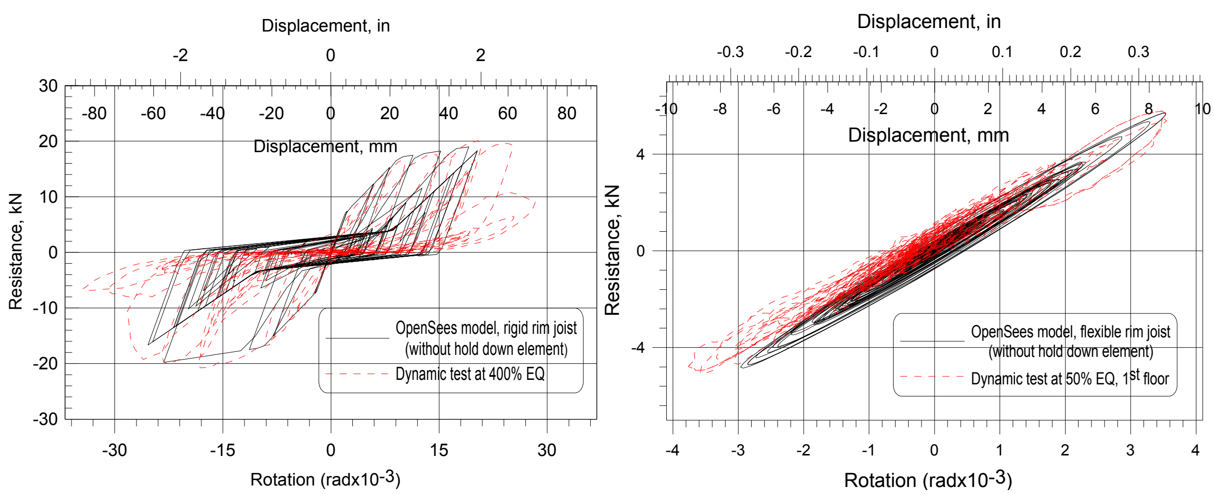


Figure 8. Double- and single-storey OpenSees dynamic models



a: Full earthquake b: 50% earthquake

Figure 9. Shear resistance-deformation hystereses: a) ST1-2 b) ST2-1 (1st floor)

Conclusions

Dynamic shake table testing of single and double-storey CFS framed / wood sheathed shear walls was carried out. The general behaviour of the walls resembled that observed during previous single-storey displacement based tests. This behaviour is largely influenced by the connections between the wood sheathing and the supporting steel frame. The results of testing allowed for the determination of representative frequency and damping values. A comparison of preliminary results from time history dynamic models showed that force and deformation behaviour of the test walls could be reasonably well predicted. Further development of these models will take place, including the introduction of inelastic elements for the chord studs and possibly the floor structure, with the ultimate objective of evaluating the performance and probability of failure of this structural framing type using the FEMA P695 methodology.

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