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Seismic Force Modification Factors for a Novel Sandwich Wall Panel as the SFRS

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ABSTRACT

The performance of a novel sandwich wall panel system is studied as the Seismic Force-Resisting System (SFRS) of a building. The National Building Code of Canada (NBCC) equivalent static force procedure uses R_d and R_o factors to account for the ductility and overstrength of the SFRS [1]. These values are tabulated in NBCC for common SFRS such as steel braced frames, steel moment frames, concrete shear walls, etc.

The methodologies proposed by FEMA P695 [2] and NRCC-CONST-56478E [3] were followed to determine the R_d and R_o factors for the proposed SFRS. A series of experimental tests were carried out under monotonic and cyclic reversed load tests in accordance with ASTM E2126 [4]. The ductility and overstrength factors for the proposed wall system as a component were determined from the hysteresis behaviour of the specimens from the tests and used in a series of Nonlinear Time History Analyses (NTHA) on two archetypes of buildings to establish initial R_d and R_o values for the system.

Keywords: Seismic design, ductility, overstrength, sandwich panel, Non-Linear Time History Analysis (NTHA).

INTRODUCTION

An integral part of the Nexii building system is a lightweight precast concrete sandwich panel that attaches to an existing or newly constructed base building structure. NEXII Structural Insulated Panel (SIP) is a composite "sandwich" panel with an EPS insulating core contained between a thin layer of a concrete-like material named Nexiite on each face. Nexiite is a proprietary product of Nexii Building Solutions Inc. (Nexii). There also exists a rib around the perimeter of the SIP to improve its flexural capacity against in-plane and out-of-plane loads. The composition of the panel is shown in Figure 1.

To evaluate the performance of the NEXII wall panels as the SFRS, experimental tests were conducted according to the test procedure outlined in ASTM E2126 [4].

The methodology provided in FEMA P-695 [2] and NRCC-CONST-56478E [3] is followed to study the in-plane performance of the NEXII wall panels as the SFRS and to determine the appropriate ductility (R_d) and over-strength (R_o) force modification factors for seismic design of the panels.

THE BUILDING ARCHETYPES USED IN THE ANALYSIS

The behaviour of a proposed SFRS was investigated using NEXII wall panels for two Commercial Retail Unit (CRU) building archetypes. As a case study, two buildings recently designed by NEXII were selected. It is assumed that the projects are located on a Vancouver site. Table 1 shows some of the geometric specifications of these two projects. Both CRU1 and CRU2 are single-story buildings. Both buildings have 12 and 5 bays in the longitudinal and transverse directions, respectively. The bay width in each direction is equal for all the bays. Figure 2 shows the 3D model of the two buildings. Since neither of the two buildings has a complex geometry or any irregularity, 2D models were developed in the longitudinal and transverse directions to reduce the computation time. More details about the 2D models can be seen in Figure 8.

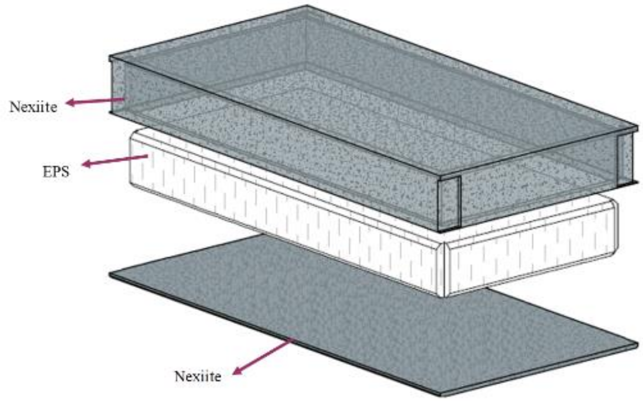
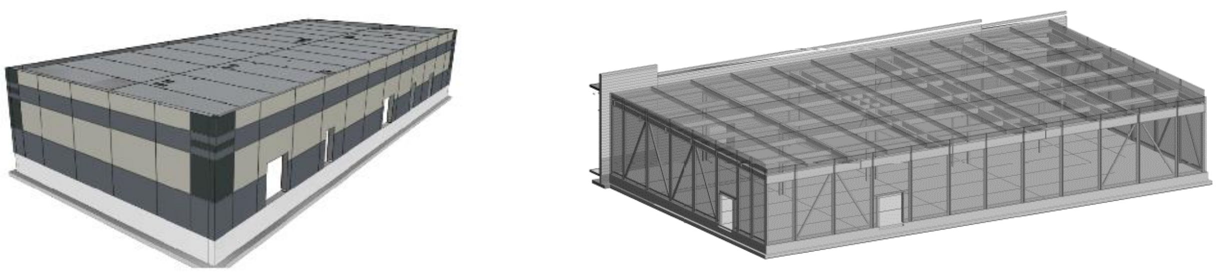


Figure 1. Composition of a typical Nexii wall panel.



(a) CRU1 *(b) CRU2*

Figure 2. 3D Renderings of the Two CRU Buildings.

Table 1. Detailed of the Selected Archetypes.

Building	Direction	Configuration	Height (m)
CRU1	Longitudinal	3.05m wide bays – Total of 12 bays	5.18
	Transverse	3.05m wide bays – Total of 5 bays	5.18
CRU2	Longitudinal	2.29m wide two end bays - 3.05m wide 10 middle bays	6.10
	Transverse	3.05-m wide bays - Total of 7 bays	6.10

EXPERIMENTAL AND ANALYTICAL STUDY

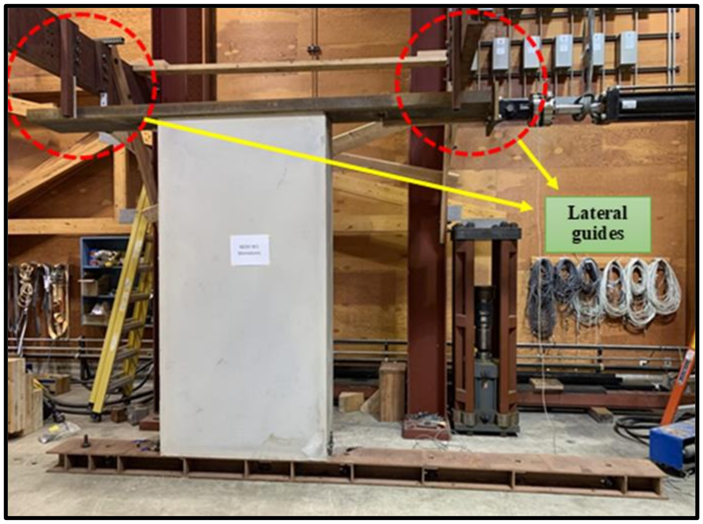
Test approach

The ASTM E2126-19 Standard specifies the testing procedure to study the in-plane behaviour of shear walls. To improve the reliability of the recorded data, four identical specimens were tested, one test was conducted under monotonic loading and three tests were performed under cyclic reverse loading. The shear strength and ductility of the specimens were determined from these tests (Table 3). An actuator with a 50 kips capacity was employed to apply a horizontal cyclic load to the specimens through a stiff steel loading beam as shown in Figure 3a. All the test panels had dimensions of 4ft × 8ft × 12in. The tests were carried out at the FP Innovations (FPI) lab in Vancouver, Canada. The specimens were fabricated at the NEXII Manufacturing Plant in Moose Jaw, SK. Table 2 shows the test matrix, including the IDs assigned to the specimens.

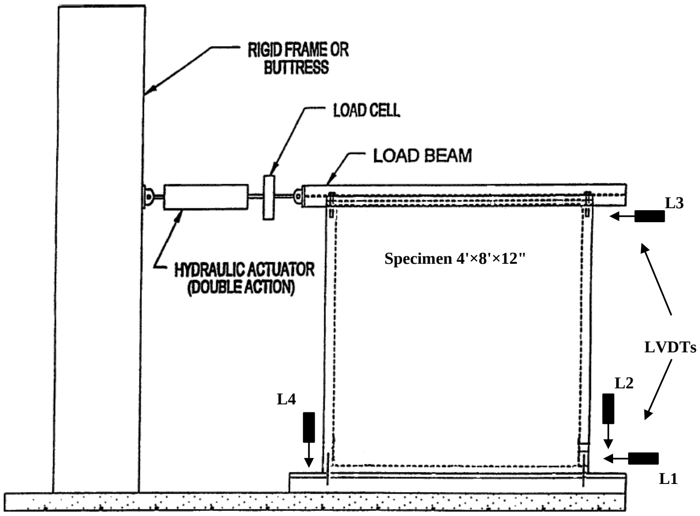
The cyclic load was delivered at a rate of 0.2Hz. The target displacement was 1.68 inches, obtained from the initial monotonic test. The tests were stopped when the load diminished more than 20% of the peak load (P_{peak}).

Table 2. Test Matrix.

Specimen ID	Loading	Dimensions	Number of specimens
W1	Monotonic	4'×8'×12"	1
W2-1	Cyclic Reverse	4'×8'×12"	1
W2-2	Cyclic Reverse	4'×8'×12"	1
W2-3	Cyclic Reverse	4'×8'×12"	1



(a) Sample test specimen inside the test setup (FPI Lab)



(b) Schematic 2D view of test setup

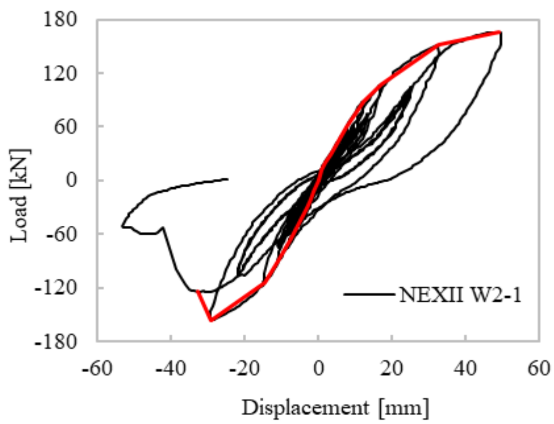
Figure 3. Racking test setup for wall panel specimen (adopted from ASTM E2126 [4]).

Test setup

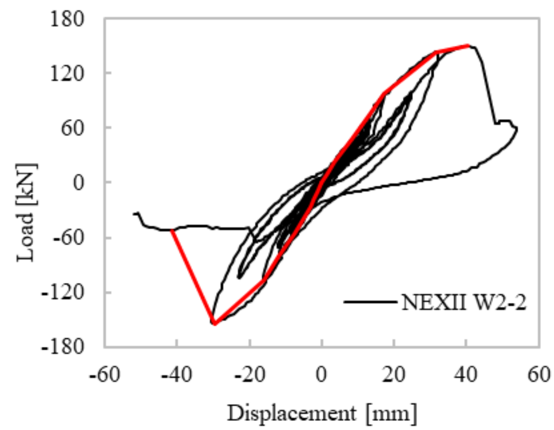
The panels were tested using a reaction frame attached to a strong floor, as illustrated in Figure 3. The bottom corners of the specimens were bolted to the rigid steel base beam to resemble the support conditions at the base of the wall panels. The test frame was laterally supported at its top as shown in Figure 3a, to prevent out-of-plane movement of the panels. The hydraulic actuator was installed in a horizontal orientation at the top of the test frame to employ the cyclic loading to the panel. Four LVDTs in total were employed to measure the displacements as shown in Figure 3b.

Hysteretic Response

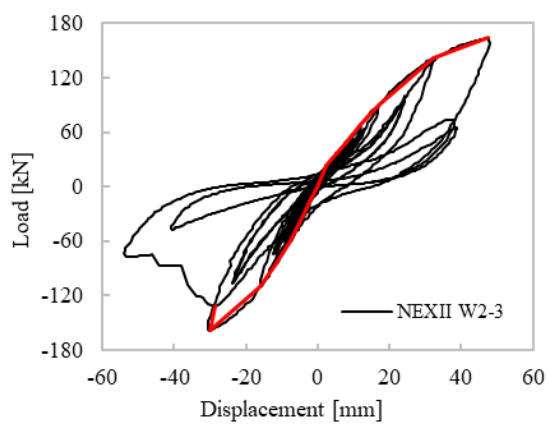
The data collected from the individual specimen recordings were used to draw the hysteresis load-displacement curves for each specimen. The backbone curves from the hysteretic responses of the four panels are presented in Figure 4. As expected, all the specimens responded similarly given their identical composition. The failure observed in all three panels was mainly due to the detachment of the bottom chord face and the Nexiite face.



(a) W2-1



(b) W2-2



(C) W2-3

Figure 4. Cyclic reverse load response of the three tested panels.

The results from the tests in terms of the panel strength (P_y), maximum load (P_{max}), displacement at yield (D_y), and failure displacement (D_u) are summarized in Table 3. The maximum drift limit achieved from the tests was 1.5% which was used as the limit in the NTHA.

Table 3. Average mechanical properties exhibited by the four panels tested.

Wall Specimens	P_y (kN)	D_y (mm)	P_{max} (kN)	D_{max} (mm)	D_u (mm)	μ (ductility)
W1	128	23.1	145	61.9	70.7	3.1
W2-1	137	16.7	154	30.8	40.7	2.4
W2-2	131	20.5	149	30.6	37.1	1.8
W2-3	133	19.3	150	31.1	38.1	1.9
Average	133	18.9	151	30.8	38.6	2.1
Standard deviation	3.26	1.95	2.60	0.27	1.88	0.33
Coefficient of Variation (%)	***2.44***	***10.38***	***1.72***	***0.87***	***4.88***	***15.84***

Idealized Load–Displacement Response

The backbone curves from the hysteretic load-displacement responses of the four tested panels were used to produce the idealized load-displacement responses. The yield-deformation (Δ_y) and ultimate deformation (Δ_u) of the panel indicate the limits of the elastic and inelastic regions, respectively. The idealized equivalent energy elastic-plastic (EEEP) bilinear curves suggested by ASTM E2126 were generated by using the values of Δ_y, P_y, and Δ_u from the experiments. The initial stiffness was taken equal to the secant stiffness at the first major crack, which corresponded approximately to 40% of the ultimate load for each panel (Figure 5). The equivalent-energy approach balances the areas under the backbone curve and the bilinear idealization [5]. The data used to plot EEEP curves are listed in Table 3. Subsequently, the ductility-related force modification factor (R_d) for the tested wall panels was determined using the EEEP curves as described in the next section.

SEISMIC FORCE MODIFICATION FACTORS (R_dR_o)

According to the NBCC 2015 equivalent static force procedure, the elastic seismic load, V_e, shall be reduced by a force modification factor equal to the multiplication of the ductility (R_d) and over-strength (R_o) in order to calculate the design seismic load, V, as shown in the following Equations.

$$V_e = S(T_a)M_vI_eW \tag{1}$$

$$V = \frac{V_e}{R_dR_o} \tag{2}$$

where $S(T_a)$ is the design spectral acceleration with the probability of exceedance of 2% in 50 years (2,475 years return) at the specified period; M_v is the factor accounting for the effect of the higher modes; W is the seismic weight; and I_e is the importance factor.

The ductility-related force modification factor, R_d, has a direct correlation with structural ductility. Newmark and Hall established a relationship (Equation 4) between the ductility ratio, μ, and the force modification factor [6]. The ductility ratio, μ, can be obtained from Equation 3. For short-period structures (T = 0.1s to 0.5s), Equation 4 is suggested to be used. Given that none of the buildings selected for this study had a period longer than 0.3s, the R_d values were determined through Equation 4 and are listed in Table 4. By using an average value of μ=2.08 from test results, R_d was estimated as 1.75 using Equation 4.

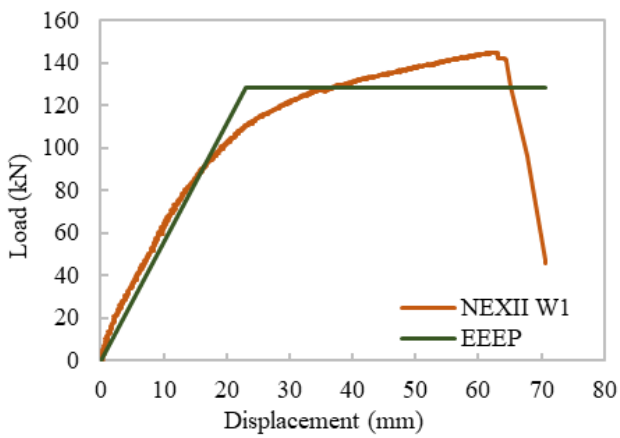
$$\mu = \frac{\Delta_u}{\Delta_y} \tag{3}$$

$$R_d = \sqrt{2\mu - 1} \tag{4}$$

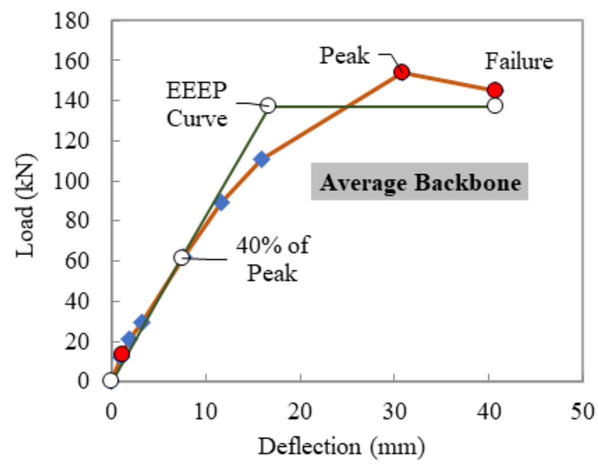
According to NBCC 2015, the overstrength-related force modification factor (R_o) is the accumulation of probable strength, which includes all potential factors contributing to the wall strength. The following Equation suggested by Mitchel et al. [7], was adopted to include the various parameters contributing to the overstrength-related force modification factor, R_o:

$$R_o = R_{size} R_{\varphi} R_{yield} R_{sh} R_{mech} \tag{5}$$

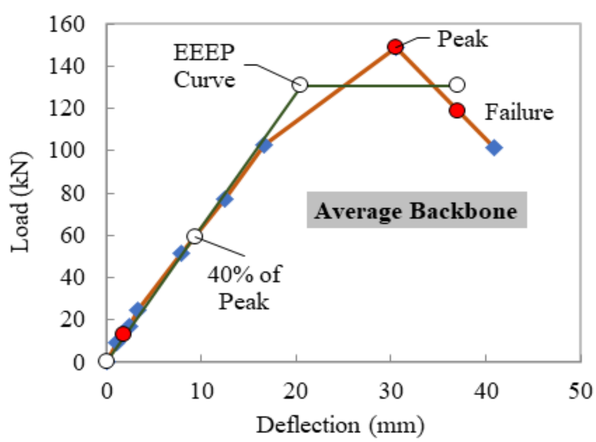
where, R_{size} is the factor that accounts for rounding of the dimensions, R_{φ} is the ratio between nominal and factored material resistances (= 1/φ), R_{yield} is the ratio of actual yield strength to the guaranteed yield strength, R_{sh} is overstrength due to strain hardening, and R_{mech} is the overstrength arising from mobilizing the full capacity of the structure until a failure mechanism is formed. Based on the recommendations of Structural Insulated Panel Association (SIPA), 2019 [8] and ICC-ES AC04 [9], a factor of safety of 3.0 is adopted in the design of the Nexii panels which results in an R_{φ} of 3.0. Conservatively, the rest of the factors (R_{size}, R_{yield}, R_{sh}, and R_{mech}) were set to unity in this study. As a result, the R_o value for wall panels was estimated as 3.0.



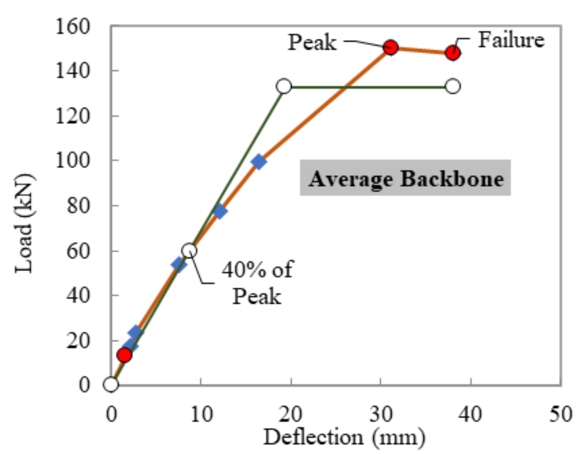
(a) W1: Monotonic test



(b) W2-1: Cyclic reverse test



(c) W2-2: Cyclic reverse test



(d) W2-3: Cyclic reverse test

Figure 5. Idealized responses of the tested panels under monotonic and cyclic reverse loading.

SELECTION AND SCALING OF GROUND MOTION RECORDS

FEMA P-695 suggests employing ground motions developed for Site Class D to estimate the $R_d R_o$ values for the selected archetypes [2]. The ground motions used in the analysis meet the criteria in NBCC Commentary J [10]. The fundamental period of both CRU1 and CRU2, archetypes was around 0.25 seconds. Figure 6 illustrates the period range T_R for CRU1 and CRU2 that was selected in accordance with the recommendations of the NBCC Structural Commentary J for the purpose of scaling the ground motion records. The UHS in Figure 6 was obtained based on the Seismic hazard deaggregations for a probability of 2% in 50 years for Vancouver, BC.

In the southwest part of British Columbia, where seismic hazard results from a combination of shallow crustal, in-slab, and subduction earthquakes, it is imperative to consider all three sources of earthquakes.

A minimum of five ground motions associated with each seismic source, i.e., crustal, in-slab, and subduction were selected. These records were chosen using Goda and Atkinson's seismic hazard model for western Canada [12] and matched to the UHS for Vancouver site Class D [13].

NON-LINEAR TIME HISTORY ANALYSIS (NTHA)

Following the NRCC-CONST-56478E guideline [3], the equivalent static force procedure was used to initially design the CRU1 and CRU2 archetypes. The CRU1 and CRU2 archetypes were comprised of four and three panels, respectively, based on the initial design. Several NTHAs were conducted to determine the number of panels that meet the maximum drift criteria of 1.5% obtained from the tests. This led to 78mm and 91mm ultimate drifts for CRU1 and CRU2, respectively. NTHA was used to estimate the R_d value for each archetype listed in Table 1 using Equations 3 and 4. For this purpose, all archetypes were modeled as 2D frames and analyzed in SeismoStruct software under the selected scaled ground motions (Figure 7). The dimensions of the frames for the CRU1 and CRU2 models in SeismoStruct are shown in Figure 8. The seismic weight applied on each node was 5.25 tonnes and 7.45 tonnes for CRU1 and CRU2, respectively. To simulate the wall panel in-plane behaviour, a non-linear link element was used to simulate the idealized load-displacement response of the panels (Figure 5). The final R_d values resulting from NTHA of each archetype are shown in Table 4.

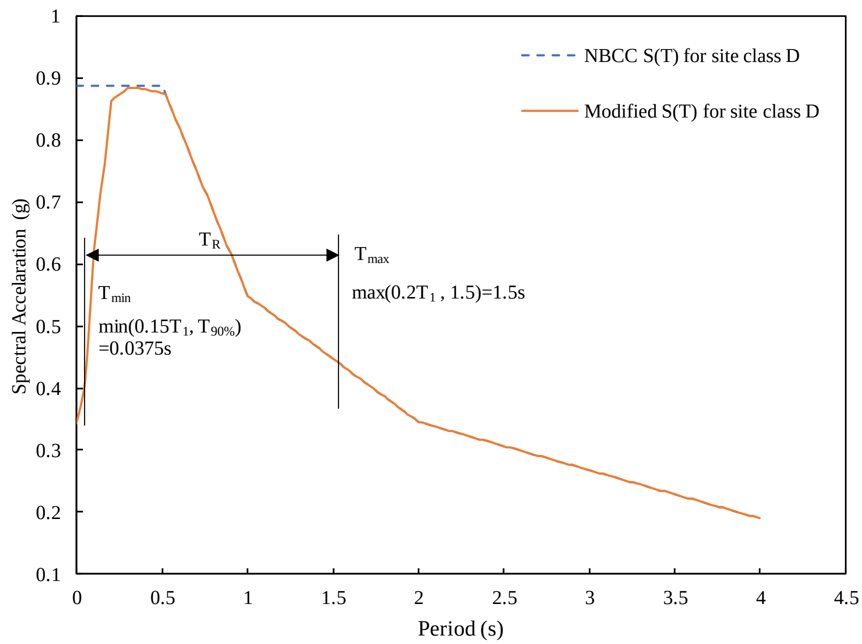


Figure 6. Target Spectrum (UHS 2015 for Vancouver, site class D)

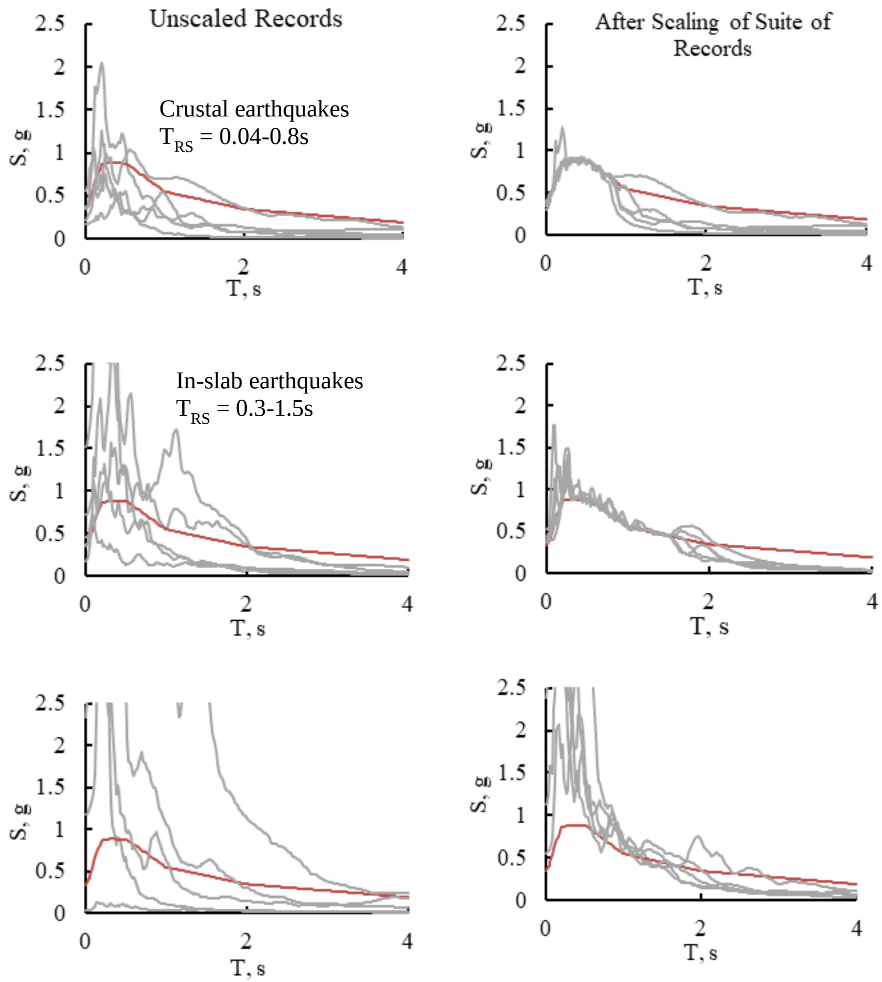
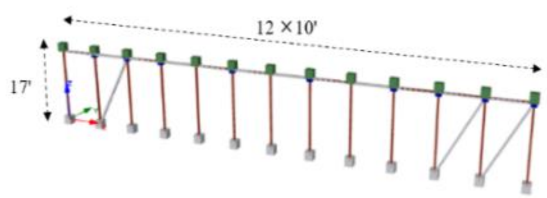
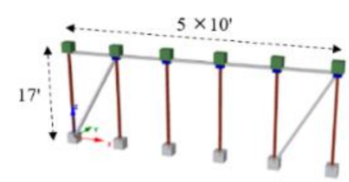


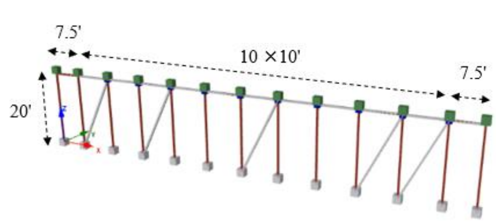
Figure 7. Selection and scaling of ground motion records for a Class D site in Vancouver ($0.0375<T_R<1.5$).



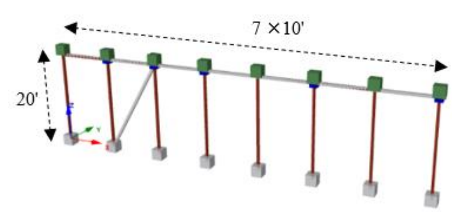
(a) CRU1-Longitudinal direction



(b) CRU1-Transverse direction



(c) CRU2-Longitudinal direction



(d) CRU2-Transverse direction

Figure 8. SeismoStruct models for CRU1 and CRU2 Archetypes.

Table 4. R_d values from the NTHA of CRU1 and CRU2

CRU1-Longitudinal	ID	R_d	Δ_y (mm)	Δ_u (mm)	CRU1-Transverse	ID	R_d	Δ_y (mm)	Δ_u (mm)	$\Delta_{allow.}$ (mm)
Shallow Crustal	1	1.35	51.2	72	Shallow Crustal	16	1.18	51.2	61	77.7
	2	1.18	51.2	61		17	1.19	51.2	62	77.7
	3	1.27	51.2	67		18	1.22	51.2	64	77.7
	4	1.27	51.2	67		19	1.30	51.2	69	77.7
	5	1.19	51.2	62		20	1.18	51.2	61	77.7
In-slab	6	1.30	51.2	69	In-slab	21	1.32	51.2	70	77.7
	7	1.39	51.2	75		22	1.27	51.2	67	77.7
	8	1.22	51.2	64		23	1.19	51.2	62	77.7
	9	1.21	51.2	63		24	1.40	51.2	76	77.7
	10	1.38	51.2	74		25	1.14	51.2	59	77.7
Subduction	11	1.22	51.2	64	Subduction	26	1.18	51.2	61	77.7
	12	1.19	51.2	62		27	1.22	51.2	64	77.7
	13	1.39	51.2	75		28	1.33	51.2	71	77.7
	14	1.27	51.2	67		29	1.33	51.2	71	77.7
	15	1.38	51.2	74		30	1.26	51.2	66	77.7
CRU2 - Longitudinal					CRU2-Transverse					
Shallow Crustal	31	1.32	51.2	70	Shallow Crustal	46	1.44	51.2	79	91.4
	32	1.38	51.2	74		47	1.33	51.2	71	91.4
	33	1.21	51.2	63		48	1.16	51.2	60	91.4
	34	1.29	51.2	68		49	1.30	51.2	69	91.4
	35	1.18	51.2	61		50	1.42	51.2	77	91.4
In-slab	36	1.42	51.2	77	In-slab	51	1.36	51.2	73	91.4
	37	1.18	51.2	61		52	1.47	51.2	81	91.4
	38	1.33	51.2	71		53	1.29	51.2	68	91.4
	39	1.38	51.2	74		54	1.30	51.2	69	91.4
	40	1.29	51.2	68		55	1.39	51.2	75	91.4
Subduction	41	1.22	51.2	64	Subduction	56	1.56	51.2	88	91.4
	42	1.32	51.2	70		57	1.32	51.2	70	91.4
	43	1.24	51.2	65		58	1.39	51.2	75	91.4
	44	1.30	51.2	69		59	1.29	51.2	68	91.4
	45	1.33	51.2	71		60	1.35	51.2	72	91.4

Figure 9 shows the normal distribution of R_d resulting from the NTHA cases conducted on both CRU1 and CRU2 archetypes. From the curve, the estimated standard deviation is 0.09, which shows that the obtained R_d values are generally consistent. Hence, a ductility-related force modification factor, R_d, of 1.3 is recommended to be used in the seismic design of the NEXII wall panels based on this preliminary study.

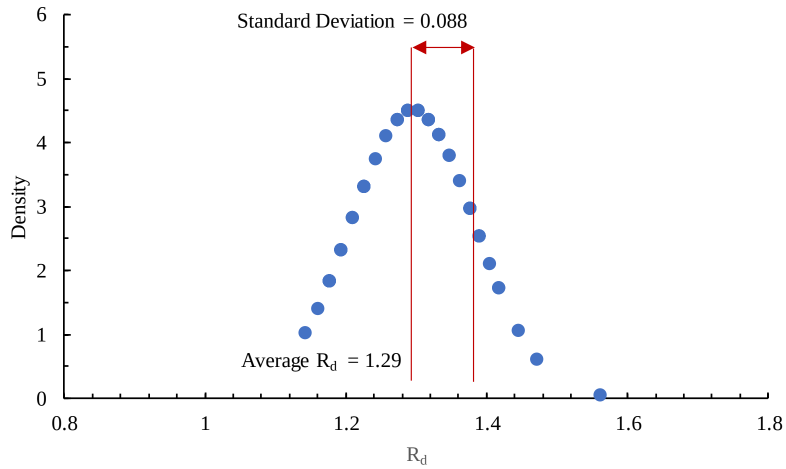


Figure 9. Normal distribution of the R_d values resulted from the NTHA.

CONCLUSION

The primary purpose of this study was to understand the in-plane behaviour of a novel composite wall panel developed and manufactured by Nexii and to determine the panel strength and ductility-related factors (R_dR_o) to be used in the seismic design of the panels when acting as the building SFRS. Following the experimental program conducted on the panels, the elastic and ultimate strengths and deformation limits were defined. Idealized load-displacement curves were obtained and used to determine the seismic force reduction factors (R_dR_o). The maximum permitted drift of the panels (Δ_u) was measured from the tests as 1.5%. A total of 15 ground motions were selected and scaled to the UHS of Vancouver, BC. The selected ground motion consisted of different sources of earthquakes, including shallow crustal, in-slab, and subduction sources. A total of 60 NTHAs were carried out in SeismoStruct software for two commercial retail unit buildings as the archetypes. Upon comprehensive evaluation of the data from the experiments and the NTHA, ductility and over-strength related force modification factors ($R_{d,}$ and R_o) of 1.3 and 3.0, respectively, are recommended to be used for the seismic design of the panels based on this preliminary study. In future revisions to the panel design by adjusting the proportion of the panel components, it is intended to alter the failure mode to a more ductile mode that allows achieving a greater R_d value.

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