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Evaluation of the Seismic Overstrength-Related Force Modification Factor of Light-Frame Wood Shearwall Buildings using the Performance-Based Unified Procedure

Esmaeil Morshedi[1*], Ghasan Doudak[2], Farrokh Fazileh[3] and Reza Fathi-Fazl[3]

[1] Graduate Research Assistant, Department of Civil Engineering, University of Ottawa, Ottawa, Canada
[2] Professor, Department of Civil Engineering, University of Ottawa, Ottawa, Canada
[3] Senior Research Officer, National Research Council Canada, Ottawa, Canada
[*]esmaeil.morshedi@uottawa.ca (Corresponding Author)

ABSTRACT

In this paper, the seismic overstrength-related force modification factor (R_o) of light-frame wood shearwall buildings is studied using the recently developed Performance-Based Unified (PBU) procedure by the National Research Council Canada (NRC). Several parameters including seismicity level, number of storeys and building heights, building irregularities, design assumptions, presence of gravity loads, shearwall aspect ratios were studied. A total of seven (7) performance groups (PGs) comprised of sixteen (16) 2D archetypes were developed. Two distinct locations, Vancouver and Montreal, representing different levels of seismicity were selected. The number of storeys for Vancouver archetypes was chosen to be 3 and 6 such that for the 6-storey archetype the height limit of 20 m for light-frame wood shearwalls is met according to the National Building Code (NBC) requirement. The Montreal archetypes had 3, 5, and 8 storeys resulting in the 8-storey building reaching the code limit of 30 m. The R_o factors are calculated using the methodology outlined in the PBU procedure. In this procedure, R_o is determined using nonlinear static (pushover) analysis. The numerical models of the designed archetypes were developed in the OpenSees platform. The nonlinear force-displacement parameters of the shearwalls were defined using experimental data available in the literature. Using the results from pushover analysis it was found that the R_o value for light-frame wood shearwalls in the NBC represents a conservative estimate.

Keywords: Overstrength-related force modification factor, light-frame wood shear walls, National Building Code of Canada, seismic force modification factors

INTRODUCTION

Seismic force modification factors are crucial to the seismic design of buildings. These factors are used to determine the design lateral forces based on the Equivalent Static Force Procedure (ESFP), as outlined in the National Building Code of Canada (NBC) 2020 [1]. The code provides the ductility-related force modification factor (R_d) and the overstrength-related force modification factor (R_o) for different structural systems. The former considers the ductility of the seismic force-resisting system (SFRS) and reflects its ability to dissipate energy through inelastic deformations. The latter, which is the focus of this paper, accounts for the reserve strength of the SFRS. This reserve strength originates from several sources including the overdesign of the structural members due to design and practical limitations regarding the discrete dimensions available, the difference between actual properties of the construction materials and those used in the design process, the degree of strain hardening of the construction materials and the redundancy of the SFRS [2,3]. The R_o value for light-frame wood shearwalls of 1.7, as provided in the NBC [1], is empirical and largely based on engineering judgment and experience gained from the performance of structures in the past earthquakes.

Most of the research regarding the seismic response modification factors in the literature concern with other SFRSs, such as steel and concrete buildings. Uang [2] suggested a framework for calculating the response modification factor, R, and the displacement amplification factor, C_d, used in the National Earthquake Hazards Reduction Program (NEHRP) provisions. These factors are functions of structural ductility and structural overstrength. The structural overstrength was defined as the ratio of the peak resistance to the resistance corresponding to the first significant yield in the structure according to the pushover curve derived from nonlinear static analysis. Miranda [4] and Miranda and Bertero [5] reported that the strength, or force reduction factor is a function of the ductility demand, soil condition and the period of the building. Whittaker *et al.* [6] formulated the response modification factor, R, as a function of the structural overstrength, structural ductility and damping in the structure. It was mentioned that the reported values for structural overstrength in the literature have significant scatter, and there is a need for a more systematic approach to calculate the structural overstrength [6]. Mitchell *et al*. [3] proposed a framework to calculate the structural overstrength, as presented in Eq. (*1*).

$$R_o = R_{size} R_{\phi} R_{yield} R_{sh} R_{mech} \tag{1}$$

Where R_{size} is due to overdesign of the structural members. R_{φ} represents the difference between nominal yield strength (V_n) and factored design strength (V_d), R_{yield} is the difference between the expected yield strength (V_e) and nominal yield strength, R_{sh} is the overstrength due to strain hardening of the material, and R_{mech} is due to failure mechanism of the structural system. Mitchell *et al*. [3] proposed different values for these parameters and R_o values for different SFRSs. For nailed light-frame wood shearwalls with wood structural panels, the proposed values for R_{size}, R_{φ}, R_{yield}, R_{sh} and R_{mech} are suggested as 1.15, 1.43, 1.0, 1.05, 1.0, respectively [3]. The calculated R_o value is hence 1.73. It should be noted that the R_{φ} value equal to 1.43 represents the resistance factor, φ, equal to 0.7. The φ factor was revised in the CSA O86-19 [7] to a value of 0.8 because of the change in the design method, while the R_o value in the NBC remained unchanged. Also, other numbers are empirical and conservative, as stated by the authors in Mitchell *et al*. [3]. For example, assuming a value of 1.0 for R_{mech} means that no redundancy is assumed in light-frame wood shearwalls, which clearly is a very conservative assumption. Also, a value of 1.0 for R_{yield} to account for the expected strength of the construction material should be investigated further considering a large ensemble experimental data. However, it should be noted that when the structural overstrength calculation is based on nonlinear pushover analysis, separating different components of the overstrength, as suggested by Mitchell *et al*. [3], may not be possible. Hence, a systematic procedure may be required to derive the overstrength factor, as also mentioned by Whittaker *et al*. [6].

The current seismic force modification factors for light-frame wood shearwalls are largely based on the observed performance of such buildings during past earthquakes and outcomes of experimental data and analytical studies in the literature [8–12]. Rainer and Karacabeyli [8] reported that most one and two-storey buildings were capable of meeting the life-safety objective of building codes. Ceccotti and Karacabeyli [9] studied the validity of the seismic design parameters, based on the 1995 NBCC through nonlinear dynamic analysis of a four-storey light-frame wood shearwall building. The authors concluded that a value of 3 for the *R* factor would result in an acceptable seismic performance of these types of structural systems. Ceccotti and Sandhaas [13] proposed a methodology for the calculation of the behaviour factor (*q*) as per the European Standard (EC8) [14] using nonlinear time history analysis. In this procedure, the *q* factor is calculated by dividing the Peak Ground Acceleration near the collapse state of the building ($PGA_{near\text{-}collapse}$) to the design PGA_{design}, e.g., $q = PGA_{near\text{-}collapse} / PGA_{design}$) [14]. Estrella *et al*. [10] studied the seismic performance factors of light-frame wood shearwalls designed according to the Chilean Seismic Design Code NCh433 [15]. The authors reported that the current response modification factor, *R*, equal to 5.5 leads to a satisfactory seismic performance of light-frame wood shearwalls against collapse.

A systematic framework has been established by FEMA P-695 [16] for the quantification of seismic performance of SFRSs, where both R_d and R_o can be calculated explicitly. In this procedure, nonlinear pushover analysis is used to determine the R_o factor. More recently, the National Research Council Canada (NRC) developed a Performance-Based Unified (PBU) procedure [17] to systematically determine the seismic force modification factors of SFRSs. The methodology is consistent with the FEMA P-695 approach with some differences in the performance evaluation stage. This methodology also uses nonlinear pushover analysis as a preliminary method for the evaluation of structural overstrength, R_o, and the period-based structural ductility, μ_T. In this procedure, the R_o factor is defined as the ratio of the maximum base shear resistance, V_{max}, to the design base shear (demand), *V*.

In order to establish a systematic evaluation of the seismic performance of light-frame wood shearwalls, this paper presents an evaluation of the R_o factor for light-frame wood shearwalls according to the PBU procedure [17]. Seven performance groups (PGs) including sixteen archetypes are selected. The number of storeys ranges from three to eight and different conditions are considered, including gravity load, weak-story and gravity induced lateral demand (GILD) irregularities, rigid versus flexible diaphragm design philosophy, and the evaluation of the 20% increase in design base shear of buildings with five storeys and more according to the NBC. Two different locations, Vancouver and Montreal, representing regions with high and moderate seismicity are chosen.

BUILDING ARCHETYPES AND DESIGN

Table 1 presents the seven selected PGs and building archetypes for this study. The table also provides the number of archetypes in each PG, number of storeys and the nomenclature used for individual building archetypes. This includes the seismicity level through the city name (VAN or MONT), followed by the number of storeys (e.g., 3, 5, 8), presence of gravity loads (G), diaphragm flexibility (i.e., F, R) and the type of irregularity considered (e.g., weak, GILD).

Table 1: Performance groups and building archetypes information

Performance Group	Number of Archetypes	Seismicity Level	Gravity load	Number of Storeys	Remarks	Building Name	Archetype Name
PG 1	2	High (Vancouver)	No	3	Basic Archetype	Vancouver 3	VAN3
			No	6	Basic Archetype	Vancouver 6	VAN6
PG 2	3	Moderate (Montreal)	No	3	Basic Archetype	Montreal 3	MONT3
			No	5	Basic Archetype	Montreal 5	MONT5
			No	8	Basic Archetype	Montreal 8	MONT8
PG 3	2	High (Vancouver)	Yes	3	-	Vancouver 3	VAN3G
			Yes	6	-	Vancouver 6	VAN6G
PG 4	3	Moderate (Montreal)	Yes	3	-	Montreal 3	MONT3G
			Yes	5	-	Montreal 5	MONT5G
			Yes	8	-	Montreal 8	MONT8G
PG 5	2	High	No	6	Irregular Type 6	Vancouver 6	VAN6Weak
		Moderate	No	3	Irregular Type 9	Montreal 3	MONT3GILD
PG 6	2	High	No	6	W/O 1.2 factor	Vancouver 6	VAN6W/O1.2
		Moderate	No	8	W/O 1.2 factor	Montreal 8	MONT8W/O1.2
PG 7	2	High (Vancouver)	No	6	Rigid Diaphragm	Vancouver 6	VAN6R
			No	6	Flexible Diaphragm	Vancouver 6	VAN6F

The plan dimensions for all building archetypes are 18.80 m and 18.0 m in the E-W and N-S directions, respectively, and are considered to be the same for all archetypes. For the Vancouver site, two buildings, one three-storey (VAN3) and one six-storey (VAN6), were selected to represent different building heights. The storey height for the VAN3 building was 3.056m and the total height of the building was 9.168m. The storey height for the VAN6 building was selected to be 3.312m resulting in a total building height of 19.872m, which is close to the code limit of 20 m [1]. For the Montreal location three-, five- and eight-storey buildings (MONT3, MONT5 and MONT8) were selected. The storey heights are 3.056 m, 3.312 m and 3.650 m, with total building heights of 9.168 m, 16.56 m and 29.20 m, respectively. The total height of the eight-storey building in Montreal was selected such that the height limit of the NBC 2020 of 30.0 m is reached [1].

The dead load (DL) is assumed to include the weight of all structural, non-structural, and other permanent facilities (e.g., partition walls, finishes, insulation, plumbing etc.). The snow load (SL) for Montreal is 2.48 kPa, while the corresponding value for the Vancouver location is 1.64 kPa [1]. This results in the seismic weights of the different building archetypes in this study as summarized in Table 2.

Table 2:Seismic weights assigned to floors and roof (kN/m^2)

Archetype	Floor	Roof
VAN3	1.12	1.35
VAN6	1.16	1.4
MONT3	1.10	1.55
MONT5	1.12	1.57
MONT8	1.13	1.6

The longitude and latitude coordinates for the Vancouver and Montreal sites are 49.256-123.0 and 45.542-73.655, respectively. Building archetypes are assumed to be built on a site with soil type “C” for both locations. In addition, the importance of the buildings is assumed to be normal. Table 3 summarized the seismic design accelerations, *Sa(T)*, for locations under consideration.

Table 3: Seismic design acceleration information

	Periods (sec)						
Location	0.0	0.2	0.5	1.0	2.0	5.0	10
Vancouver	1.05	1.05	0.838	0.487	0.296	0.085	0.036
Montreal	0.751	0.751	0.543	0.293	0.133	0.035	0.011

The seismic analysis was based on the ESFP for base shear calculation and the distribution of seismic forces along the height of the building archetypes found in the NBC [1]. The distribution of seismic forces between different shearwalls was carried out based on the worst case for rigid and flexible diaphragm analyses. The force distribution between different shearwall segments within a shearline was based on the shearwall stiffness. The shearwalls consisted of Douglas fir plywood attached on one side of blocked S-P-F wall studs of No.1/No.2 grade. This represents common construction practice in light-frame wood shearwall buildings in Canada. To resist the overturning moment and control storey drift under seismic forces, continuous rod tie-down systems from the Simpson Strong-tie® Catalogue [18] were used at the ends of each shearwall segments. The continuous rod tie-downs extend from the foundation to the top of the building and are tied at each level. Shrinkage compensators are assumed at each level of the building in order to control the tie-down displacements induced by wood shrinkage as a result of moisture reduction in the indoor environment after construction.

Table 4 provides the designed archetypes and their corresponding design periods. Also, the periods of the archetypes are calculated using Rayleigh method [19] based on the 4-term deflection equation provided in the CSA O86-19 [7]. For the performance evaluation of buildings in this study, a shearline representative of the whole building is chosen and analyzed.

Table 4: Design period of the archetypes

Archetype	**height (m)**	**Design Period (sec)**		**Calculated Period-Rayleigh method**	
		N-S Dir.	**E-W Dir.**	**N-S Dir.**	**E-W Dir.**
VAN3	9.17	0.53	0.53	0.57	0.59
VAN3G	9.17	0.53	0.53	0.53	0.59
VAN6	19.87	0.82	0.83	0.82	0.83
VAN6G	19.87	0.70	0.74	0.70	0.74
VAN6W/O1.2	19.87	0.94	0.94	0.95	0.97
VAN6Weak	19.87	0.81	0.83	0.81	0.83
MONT3	9.17	0.53	0.53	0.73	0.78
MONT3G	9.17	0.53	0.53	0.74	0.84
MONT5	16.56	0.82	0.82	1.12	1.12
MONT5G	16.56	0.82	0.82	1.00	1.19
MONT8	29.20	1.26	1.26	1.58	1.74
MONT8G	29.20	1.26	1.26	1.50	1.73
MONT8W/O1.2	29.20	1.26	1.26	1.75	1.98

NUMERICAL MODELING OF THE BUILDING ARCHETYPES

The numerical models of the archetypes are developed based on established modeling procedures for light-frame wood shearwalls subjected to lateral seismic loads, using the OpenSees platform [20], as illustrated in Figure 1. In this configuration, each shearwall segment is modeled using rigid elements (e.g., elasticBeamColumn of the OpenSees library) connected to each other through pin connections. This modeling technique is similar to that of the CUREE Caltech project [19]. The shear behavior is implemented using a shear spring (zero-length element) located at the mid-height of the shearwall segment. Tie-down rods, representing the hold-downs, are modeled to incorporate the cumulative effects of displacements along the height of the buildings. This is done by modeling zero-length spring elements at the base of each storey.

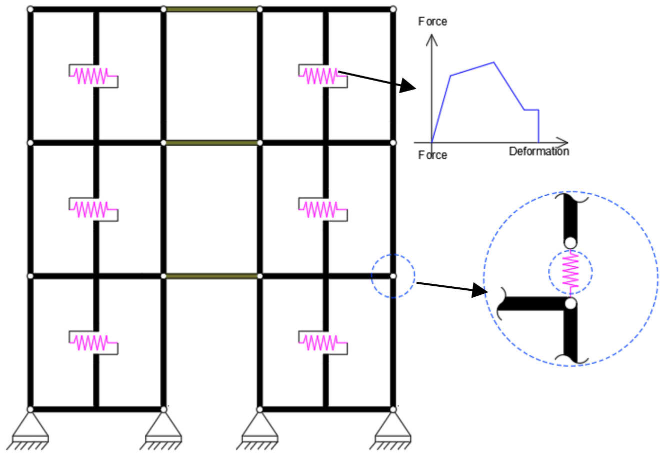


Figure 1: OpenSees model of a light-frame wood shearwall

For the nonlinear pushover analysis, the nonlinear force-displacement (F-D) behavior of the shearwalls is simulated through a multi-linear curve. The backbone curve of the Pinching4 material behaviour found in the OpenSees [20] library is used. The coordinates of the F-D curve of the Pinching4 material model are shown in Figure 2.

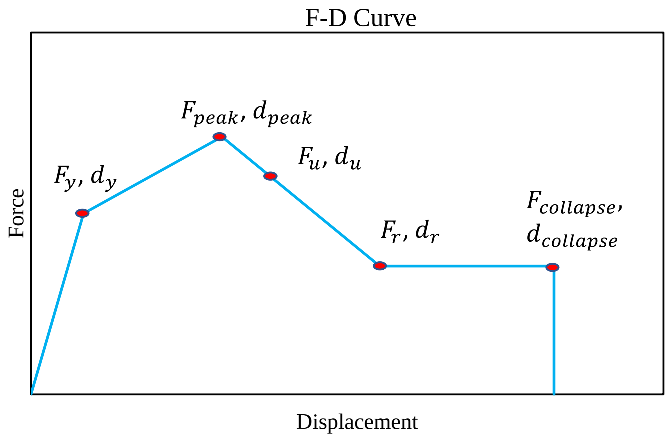


Figure 2: Coordinates of a F-D curve

From Figure 2, F_y and d_y represent the yield force and yield displacement of the shear wall, respectively. F_{peak} and d_{peak} are the maximum resistance of the shear wall and its corresponding displacement, respectively. F_{ult} and d_{ult} are the points corresponding to a 20% decrease in peak resistance. F_r and d_r are residual force and displacements, respectively. $F_{collapse}$ and $d_{collapse}$ are points that represent the last point of the F-D curve, and they correspond to the collapse of the shear wall. NIST GCR 17-917-45 [21] recommends $d_{collapse}$ to be 1.5 d_{ult}, and as such, this value is adopted in the current study. The coordinates of the F-D curve were determined using regression formulas developed [22] based on available test results on full scale shear walls [23]. More information regarding the derivation of the regression formulas can be found in [22]. It is noted that since the regression formulas are derived based on the experimental data, they include the inherent variability found in the test data.

The behavior of the rod in tie-down system is modeled using elastic material behaviour with different stiffnesses in tension and compression (see Figure 3). Maintaining elastic behaviour in the hold-down follows the requirements outlined in the CSA O86-19 [7].

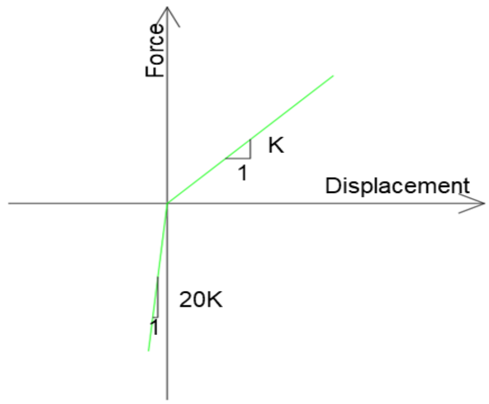


Figure 3: Elastic material used for modeling rod tie-downs

In Figure 3, the stiffness of the tie-down, *K*, is equal to *EA/h*. *E* is the elastic modulus of the steel rod material, taken equal to 200,000 MPa. *A* is the ATS rod cross-sectional area, and *h* is the length of the ATS rod, which is equal to the storey height. A high stiffness value, equal to 20 times the tension stiffness (20**K*), is considered for the ATS rod springs in compression, simulating the contact behavior of the vertical end-studs and the shearwall bottom plate.

PUSHOVER ANALYSIS

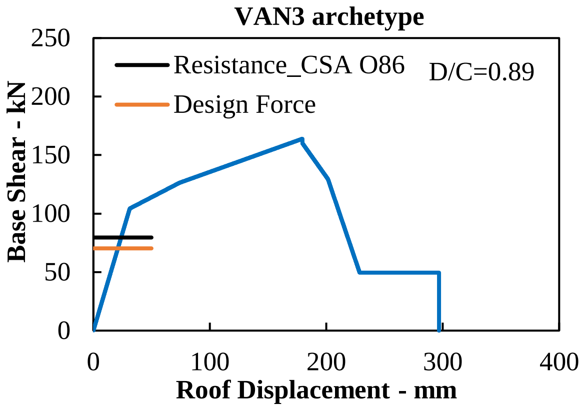
The pushover analysis for all building archetypes is performed in OpenSees, and the base-shear versus roof-displacement curves are developed. Each archetype is loaded with increasing lateral force until the failure point. The lateral load pattern over the height of the building in the pushover analysis corresponds to the lateral load distribution in the design step. Figure 4 (a)-(f) illustrate the base-shear versus roof-displacement curves for some of the archetypes in this study obtained from the pushover analysis. These figures also include the design base shear as well as the shearwall capacity according to the CSA O86-19 [7]. In addition, the demand to capacity ratio (*D/C*) of the bottom storey is shown on each graph. Table 5 shows the *D/C* ratios from the design stage of all archetypes for all storeys. The bold numbers show the collapsed level in pushover analysis.

Table 5: Demand-to-capacity ratio values for Vancouver and Montreal archetypes

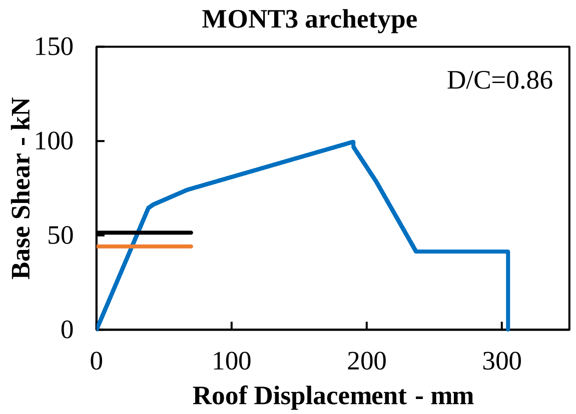
					D/C Ratio			
Storey	**VAN3**	**VAN3G**	**VAN6**	**VAN6G**	**VAN6W/O1.2**	**VAN6Weak**	**VAN6R**	**VAN6F**
6			0.48	0.41	0.36	0.4	0.47	0.66
5			0.76	0.67	0.58	0.65	0.74	0.77
4			0.81	0.84	0.75	0.83	0.91	0.83
3	0.95	0.71	0.77	0.93	0.87	0.8	0.96	0.79
2	**0.99**	0.86	0.85	0.96	0.95	0.8	**1.00**	0.86
1	0.95	**0.95**	**0.88**	**0.99**	**0.99**	**0.93**	0.94	**0.90**

Storey	**MONT3**	**MONT3G**	**MONT3GILD**	**MONT5**	**MONT5G**	**MONT8**	**MONT8G**	**MONT8W/O1.2**
8						0.62	0.75	0.59
7						0.90	0.92	0.86
6						0.96	0.94	0.91
5				0.86	0.89	0.91	0.87	0.87
4				0.88	0.90	0.80	**0.87**	0.80
3	0.95	0.83	0.87	0.88	0.88	0.86	0.78	0.86
2	**0.94**	0.82	0.81	0.98	0.98	0.91	0.82	0.91
1	0.86	**0.95**	**0.95**	**0.91**	**0.92**	**0.93**	0.84	**0.93**

As can be seen from Table 5, the *D/C* ratios for all archetypes are reasonably close to unity, indicating a relatively optimized design of the archetypes in this study. This was meant to result in a relatively uniform inelastic shearwall deformations, and hence energy dissipation, along the height of buildings. Although it is generally observed that the failure occurs in the first and second storeys of each building archetype, it can be observed that an attempt to achieve optimized design may sometimes cause failure of other higher storeys. Where the yielding is initiated may also be affected by the variability in the actual behaviour of shearwalls, and hence the preference to base the derived regression formulas on actual test results.



(a)



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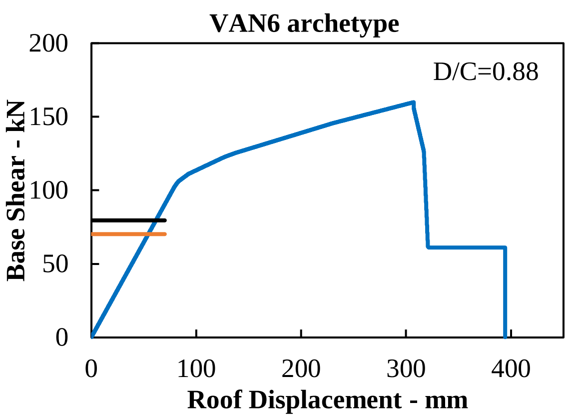

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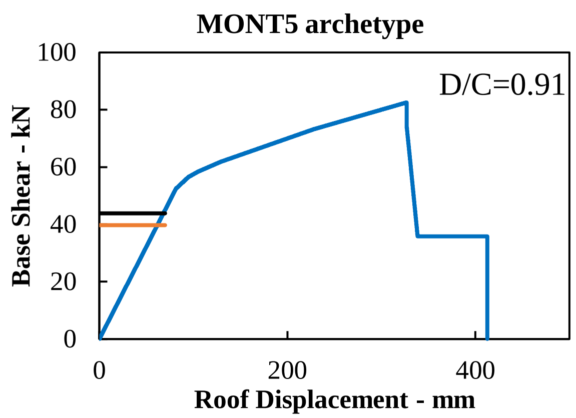

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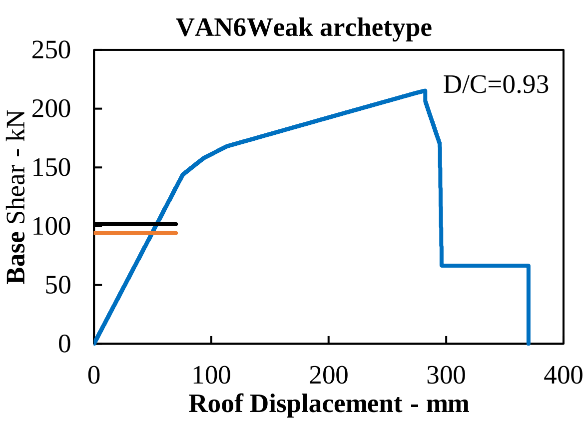

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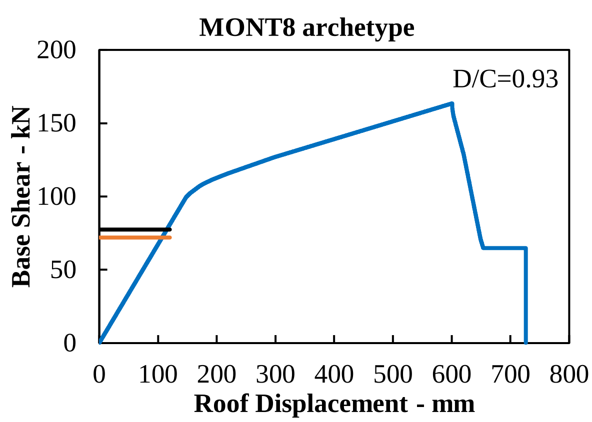

(f)

Figure 4: Base-shear versus roof-displacement (pushover) curves: (a) VAN3 archetype, (b) MONT3 archetype, (c) VAN6 archetype, (d) MONT5 archetype, (e) VAN6Weak archetype, (f) MONT8 archetype

PERFORMANCE EVALUATION OF THE BUILDING ARCHETYPES

The overstrength-related force modification factor (R_o) of the investigated archetypes is calculated according to the PBU procedure [17], as the ratio of the maximum base shear, i.e., peak resistance (V_{peak}), to the design base shear (V). Accordingly, the R_o values for all archetypes are calculated and are summarized in Table 6.

Table 6: The overstrength-related force modification factor (R_o) values

Performance Group	Number of Archetypes	Number of Storeys	Archetype	R_o
PG 1	2	3	VAN3	2.26
		6	VAN6	2.29
			Average	**2.28**
PG 2	3	3	MONT3	2.31
		5	MONT5	2.13
		8	MONT8	2.10
			Average	**2.18**
PG 3	2	3	VAN3G	2.26
		6	VAN6G	2.16
			Average	**2.21**
PG 4	3	3	MONT3G	2.31
		5	MONT5G	2.13
		8	MONT8G	2.18
			Average	**2.21**
PG 5	2	6	VAN6Weak	2.23
		3	MONT3GILD	2.08
			Average	**2.16**
PG 6	2	6	VAN6W/O1.2	2.36
		8	MONT8W/O1.2	2.18
			Average	**2.27**
PG 7	2	6	VAN6R	2.28
		6	VAN6F	2.32
			Average	**2.30**

Based on the PBU procedure [17], the value of R_o was selected to be based on the minimum of the average values of all PGs resulting in a value of 2.2. it is noted that this value is greater than 1.7 in the NBC [1] for light-frame wood shearwalls. It can generally be observed that the R_o value tends to decrease as the height of the archetypes increases. This is due to the fact that for multi-storey buildings, and despite effort in design, uniform inelastic deformations along the height of the building may not be ensured. Furthermore, irregularities in shorter buildings may also cause concentration of plastic deformations in a specific storey, and as a result, this could affect both the ductility and overstrength of the structure. For example, it can be observed that the three-storey Montreal archetype with GILD irregularity has the lowest R_o value. It is important to note, however, that interpretation of the effect of irregularities on the seismic performance of a structural system is incomplete unless other 3D effects are considered. Regarding the weak-storey irregularity, as expected, the collapse occurred at the bottom storey where the shearwall had lower capacity than the top storey, although the capacity at the bottom storey is greater than the demand. The R_o value seems to be insensitive to parameters such as gravity load, rigid versus flexible diaphragm assumption and increase in the base shear for buildings with five storeys and more.

The period-based ductility, μ_T, of an archetype can be extracted from the results of pushover analysis. This factor represents an approximate measure of the structural ductility and provides an indication of the ductility-related force modification factor, R_d [2]. According to the PBU procedure [17], the μ_T factor is defined by Eq. (2).

$$\mu_T = \frac{\delta_u}{\delta_{y,eff}} \tag{2}$$

Where δ_u is the ultimate displacement, and $\delta_{y,eff}$ is the effective yield displacement at the roof level. The δ_u factor depends on the performance level of interest, i.e., Immediate Occupancy (IO), Life Safety (LS), and Collapse Prevention (CP)), with roof displacements assumed to be corresponding to the inter-storey drift ratio (*IDR*) levels of 1%, 2.5%, and 4.5%, respectively. The effective yield displacement, $\delta_{y,eff}$, is calculated using Eq. (3), based on the PBU procedure [17].

$$\delta_{y,eff} = C_0 \frac{V_{max}}{W}\left(\frac{g}{4\pi^2}\right) \times \max\,(T, T_1)^2 \tag{3}$$

Where C_0 is a coefficient that relates the fundamental period of an equivalent SDOF system to the roof displacement of an MDOF system [16,19] and can be obtained using Eq. (*4*). V_{max} and W are the maximum base shear of the archetype according to the pushover curve and total weight of the archetype, respectively. g is the constant of gravity. Periods T and T_1 are the design period and the period obtained from modal analysis, respectively.

$$C_0 = \varphi_{1,r} \frac{\sum_1^N m_x \varphi_{1,x}}{\sum_1^N m_x \varphi_{1,x}^2} \tag{4}$$

Where m_x is the mass at level x, $\varphi_{1,x}$ is the eigenvector of level x in the first mode, and $\varphi_{1,r}$ is the eigenvector at roof level in the first mode of the archetype.

Table 7 summarizes the μ_T values for all archetypes at different performance levels. It can generally be observed that μ_T decreases as the height or the period of the archetypes increases. The rate of this decrease in the μ_T is not consistent for all performance levels, and it depends on the level of inelastic deformations in the SFRS. It also appears that the presence of gravity load reduces the structural ductility, and this reduction becomes more pronounced when the intensity of the gravity loads increases. The reduction seems further worsened when the structure enters into post-peak behaviour.

Table 7: Period-based ductility values of the archetypes

Performance Group	Archetype	T_a (sec)	T_1 (sec)	C_0	V_{max} (kN)	W (kN)	$\delta_{y\text{-}eff}$ (mm)	δ_{IO} (mm)	δ_{LS} (mm)	δ_{CP} (mm)	$\mu_{T\text{-}IO}$	$\mu_{T\text{-}LS}$	$\mu_{T\text{-}CP}$
PG 1	VAN3	0.53	0.58	1.24	163.89	404.88	42.34	75.73	179.44	228.66	1.79	4.24	5.40
	VAN6	0.82	0.89	1.31	159.98	408.49	100.23	129.73	307.41	321.69	1.29	3.07	3.21
PG 2	MONT3	0.53	0.84	1.23	99.64	425.12	50.82	80.21	190.08	236.46	1.58	3.74	4.65
	MONT5	0.82	1.26	1.26	82.55	377.39	108.70	137.89	326.75	338.57	1.27	3.01	3.12
	MONT8	1.26	1.96	1.31	163.55	1072.52	191.08	253.56	600.85	649.38	1.33	3.15	3.40
PG 3	VAN3G	0.53	0.62	1.26	121.45	303.76	48.15	63.23	149.83	199.37	1.31	3.11	4.14
	VAN6G	0.70	0.83	1.36	139.28	329.00	98.33	125.49	297.36	310.40	1.28	3.02	3.16
PG 4	MONT3G	0.53	0.78	1.23	85.86	317.17	50.45	64.36	152.50	198.32	1.28	3.02	3.93
	MONT5G	0.82	1.24	1.28	109.88	515.55	104.58	132.21	313.29	316.97	1.26	3.00	3.03
	MONT8G	1.26	2.05	1.32	128.17	804.38	219.64	265.09	628.17	636.79	1.21	2.86	2.90
PG 5	VAN6Weak	0.83	0.85	1.34	215.33	609.12	85.65	119.14	282.31	296.40	1.39	3.30	3.46
	MONT3GILD	0.53	0.83	1.24	108.82	493.55	46.60	66.40	157.34	203.46	1.43	3.38	4.37
PG 6	VAN6W/O1.2	0.94	0.90	1.34	157.55	609.12	76.34	95.53	226.38	262.92	1.25	2.97	3.44
	MONT8W/O1.2	1.26	2.14	1.31	122.65	1076.16	169.99	219.80	520.86	520.92	1.29	3.06	3.07
PG 7	VAN6R	0.94	0.93	1.36	169.87	490.00	103.99	127.23	301.49	320.49	1.22	2.90	3.08
	VAN6F	0.94	0.94	1.34	129.64	408.23	93.38	125.70	297.86	317.60	1.35	3.19	3.40

T_a: Design period; T_1: First-mode period from Eigenvalue analysis

CONCLUSIONS

The objective of this research was to investigate the seismic performance of light-frame wood shearwalls using the PBU procedure [16]. This paper summarized the results of the overstrength factor (R_o) and period-based ductility (μ_T) of light-frame wood shearwalls. Sixteen archetypes for two distinct locations, representing regions with high and moderate seismic intensities, were designed according to NBC 2020 and CSA O86-19 requirements, and evaluated using nonlinear pushover analysis. Based on the results of this study, the minimum R_o value was found to be 2.2 which is greater than the corresponding R_o value for light-frame wood shearwalls found in the NBC, R_o =1.7. There is a tendency for the R_o factor to decrease with increasing the height of the building. Also, for the archetypes in this study, it seems the R_o factor is independent of the level of seismicity. Generally, the presence of irregularity decreased the value of the overstrength factor, with the lowest factor found to be corresponding to the archetype with GILD irregularity. The R_o value seems to be insensitive to other parameters such as gravity load, rigid versus flexible diaphragm design philosophy and increase of 20% in the base shear of buildings for five storeys and more. The *D/C* ratio played a significant role in the collapse mechanism as the storey with closest *D/C* ratio to unity was more susceptible to enter into inelastic behaviour and possibly yield to building collapse. Failures were also observed to occur at storeys other than the bottom storey resulting in a lower structural overstrength. The period-based ductility (μ_T) decreased as the period of the building increased, which was more pronounced when the buildings underwent significant inelastic deformations. The presence of the gravity load reduced the period-based ductility, and this reduction was observed to be greater when the structure entered into post-peak behaviour with negative stiffness.

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