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MULTI-STOREY LIGHT-FRAME WOOD SHEAR WALL LATERAL DEFLECTION: INVESTIGATION OF THE CUMULATIVE ROTATION EFFECT IN SEISMIC DESIGN

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ABSTRACT: Multi-storey light-frame wood shear wall systems are among the most prevalent type of construction, as they are cost-effective and can be built in a timely manner. In Canada, the CSA O86 "Engineering Design in Wood" standard recommends in its Annex a "multi-storey" approach for calculating lateral deflections of light-frame wood shear walls. This approach considers the cumulative rotations and is currently applied when designing mid-rise buildings using a mechanics-based period. While cumulative rotations are likely to occur to a certain extent in the elastic range, the predominant racking behavior of light-frame wood shear walls would prevent these cumulative rotations from continuing in the inelastic range. Furthermore, in moderate- to high-seismic zones, the 2020 National Building Code of Canada requires an additional 20% increase in the design base shear for mid-rise wood buildings when designed using a mechanics-based period approach. This paper investigates the impacts of these assumptions on the estimation of the lateral deflection for the seismic design of mid-rise light-frame wood buildings in Canada. A 2-segment, 6-storey light-frame wood shear wall was designed with and without accounting for these assumptions, and solutions are proposed to reduce undue conservatism. The resulting designs were evaluated using non-linear dynamic analyses. The results indicate that the proposed solutions in this paper lead to a safe and efficient design.

KEYWORDS: Earthquake, modelling, overturning, deflection, lateral loads, seismic design, shear wall

1 – INTRODUCTION

Mid-rise light-frame wood (LFW) buildings are chosen by designers for their cost efficiency and speed of construction, which are two key elements outlined in the Government of Canada's plan to reach their affordable housing objectives [1]. However, the National Building Code of Canada (NBC) 2020 [2] introduced higher seismic hazard values that led to greater design levels compared to previous editions, and this has impacted the material-efficiency, and therefore the feasibility, of LFW midrise buildings. The seismic hazard across the country was revised due to advancements in the ground motion models introduced with the 6th Generation Seismic Hazard Model of Canada [3]. While the new models aim to improve reliability when determining seismic hazard, it has become increasingly difficult to design 5- to 6-storey LFW buildings in high seismic zones using available analysis methods, in particular the Equivalent Static Force Procedure (ESFP).

The increased seismic hazard values in NBC 2020 have made it more challenging for mid-rise buildings to meet the storey drift requirements, which are set as a percentage of the storey height, in addition to stringent strength requirements. While the higher loads are affecting all building types, the existing seismic design requirements for LFW buildings contain multiple sources of undue conservatism, especially when determining the story drift under seismic loading.

For buildings located in moderate- to high-seismic zones and comprising more than 4 storeys of continuous wood construction, the NBC [2] requires a 20% increase in the base shear when a mechanics-based period is used in seismic design. As outlined in APEGBC [4], it was observed through research that designing a SFRS with 20% additional capacity [5] or increasing the minimum ESFP design loads by 20% would reduce the potential for weak-storey issues. Industry professionals concurred that increasing the design base shear by 20% when using a mechanics-based period was a simple solution that would limit the probability of weak-storey occurrence. While the

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intent may have been to increase the capacity of LFW shear walls, the increase also affects the calculation of lateral deflections for the storey drift requirements.

To determine the lateral deflection of shear walls in multi-storey buildings, the informative Annex A of the CSA O86 standard [6] recommends using a 4-term deflection equation with additional terms to account for the cumulative rotation of the shear wall segments. The effects of shear connections (discrete shear transfer elements in wall-to-wall and wall-to-foundation connections) and diaphragm stiffness are not considered in the deflection equations in Annex A of CSA O86. A recent study [7] on multi-storey shear walls cautioned that the cumulative rotations may result in non-conservative designs when resulting lateral deflections are used to determine the building's period for strength design, despite being too conservative for the inter-storey drift check. The study also identified that the out-of-plane stiffness of the floor diaphragm can greatly limit the cumulative rotations.

An additional source of conservatism arising from the inclusion of rotational effects relates to the application of force modification factors. When using linear analyses in seismic design, the base shear is reduced by a ductility-related force modification factor, R_d, and an overstrength-related force modification factor, R_o, to account for inelastic behaviour and overstrength of the SFRS, as per the NBC [2]. Since reduced design forces will result in reduced elastic lateral deflections, the NBC requires that the calculated deflections be multiplied by R_dR_o, to predict the realistic values of anticipated lateral deflection of the system. Past research observed that the rigid body rotation of shearwalls with hold-downs is significantly reduced compared to shearwalls without hold-downs. [8]. The study was conducted using discrete hold-downs, while this paper focused on continuous hold-downs to resist higher uplift in mid-rise buildings located in high-seismic zones. LFW shear walls are designed to ensure that energy is mainly dissipated through the sheathing-to-framing connections, which may limit the inelastic behaviour to occur through the racking movement. This suggests that cumulative rotations may be expected not to go beyond the elastic range. Since rotational effects in LFW shear walls are overestimated due to neglecting shear connections and diaphragm effects, as previously mentioned, and rotational behaviour is not expected to be inelastic, the appropriateness of scaling it by R_dR_o comes into question as it may not accurately represent the realistic lateral deflection of the system (see Fig. 1).

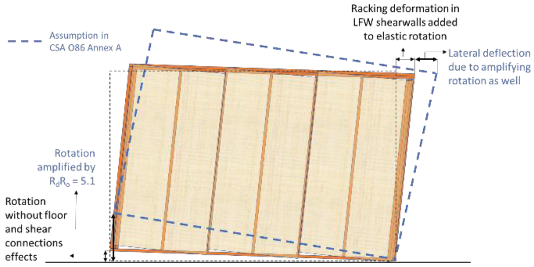

Figure 1. Schematic illustration of anticipated inelastic deflections in LFW shear walls

The conservativism inherent to the current seismic design of LFW shear walls makes it difficult for designers to comply with the required drift limits. Over-designing the system is among options used by designers to reduce lateral deflections, which may negatively impact its seismic performance. This approach can cause energy dissipation to occur on the lower storeys only, rather than over the full height of the building, since the impact of cumulative rotations is higher in the upper storeys. While it is generally understood that conservative designs lead to safer structures, studies [9, 10] have suggested that a strength profile proportional to the lateral force on each storey better dissipates energy across all storeys compared to a SFRS designed with a constant strength profile. The need to increase wall capacity to reduce lateral deflections often prevents designs from having a proportional strength profile in mid-rise buildings. The effects of storey-to-storey overcapacity ratios are also investigated in this study.

Since existing work that evaluates multi-storey, multi-segment LFW shear walls is limited, more research is needed to understand the behaviour of the system. This study investigates the validity of an assumption that limits the cumulative rotations to the elastic range in multi-storey LFW shear walls. This was achieved by performing non-linear time history analyses on a shear wall designed using this assumption and comparing results with forces and deflections predicted using ESFP. The impact of increasing the base shear by 20% when determining lateral deflections is also discussed in this paper. Due to the computationally intensive nature of non-linear dynamic analyses, this study was limited to a 2-segment, 6-storey hypothetical building.

To investigate sources of undue conservatism, a Python program capable of performing the seismic design of a midrise LFW building, using a mechanics-based period, was developed. The Python program was established following NBC and CSA O86 provisions but can be adapted to implement new rationalized design assumptions. The design assumptions helped to successfully design a 6-storey LFW building in Vancouver, B.C., considered as a high seismic location. No collapse was observed in the non-linear dynamic analysis.

2 – BACKGROUND

2.1 Codes and Standards provisions

The increased seismic hazard introduced in NBC 2020 compared to NBC 2015 is due to advancements in the models used to determine expected ground motions [3]. For typical LFW midrise shear walls, the new seismic hazard values in NBC 2020 result in a 19% increase in design level for short periods and 22% increase in design level for long periods in Vancouver (City Hall) Site Class C, compared to NBC 2015 (Fig. 2).

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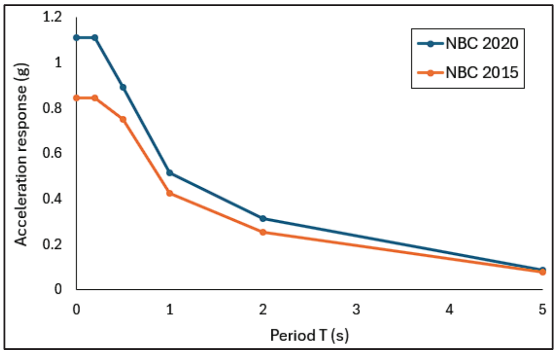

Figure 2. NBC 2020 and NBC 2015 design spectrum in Vancouver (City Hall), B.C, for Site Class C

The NBC [2] prescribes an empirical equation to determine the fundamental lateral period, Ta, for wood-based shear wall systems using the height, h_n (in m), of the building (1).

$$Ta = 0.05(h_n)^{3/4} \tag{1}$$

In addition, the NBC permits the use of mechanics-based methods to determine the building's fundamental period, such as the Rayleigh method [11], or period resulting from modal analysis. For LFW shear walls, mechanics-based methods often generate larger periods than the NBC empirical equation, therefore reducing the seismic design forces. To not underestimate design forces, mechanics-based periods are capped at an upper limit of 2xTa for strength design and 2 s when determining deflections.

In CSA O86, the lateral deflection equation for LFW shear walls is composed of terms representing 4 distinct components: bending deflection, shear deflection, nail slip, and hold-down elongation [6]. A more detailed deflection equation for application in multi-storey shear walls, and accounting for cumulative rotations, is added to Annex A of the standard. Cumulative rotations, α (2) and θ (3), are respectively associated with the elongation of the hold-down system and the bending component.

$$\alpha = (d_a)_j / L_s \tag{2}$$

$$\theta = M_jH_j/(EI)_j + V_jH_j^2/(2EI)_j \tag{3}$$

Where j is the storey number, d_a is the total vertical elongation of the hold-down, L_s is the length of the shear wall segment, M is the overturning moment at the top of storey, H is interstorey height, EI is the shear wall bending rigidity, and V is the applied shear force.

To quantify the lateral deflection related to the α and θ, Δ_α and Δ_θ respectively, (4) and (5) are used.

$$\Delta_\alpha = \tan(\alpha) \times H_j \tag{4}$$

$$\Delta_\theta = \tan(\theta) \times H_j \tag{5}$$

In order to predict lateral deflection in the inelastic range, the NBC requires the elastic deflection, which includes the cumulative rotation when determined using Annex A, to be multiplied by R_dR_o (5.1 for light-frame wood shear walls).

Furthermore, CSA O86 requires the overcapacity ratio of the two first storeys, C_2/C_1, to be between 0.9 and 1.2 to prevent a weak-storey mechanism from occurring under dynamic seismic loading. The overcapacity of a given storey is determined in (6).

$$C_j = V_{rj} / V_{fj} \tag{6}$$

Where V_{rj} and V_{fj} are respectively the factored resistance and the design shear force of all shear walls at storey j.

The CSA O86 commentary [12] recommends extending the application of the overcapacity ratio criterion to subsequent storeys for the same reasons, and this is especially valid for 6-storey buildings. Similar recommendations are made by [9, 10], where a strength profile proportional to the lateral force on each storey (i.e., an overcapacity ratio, C_{j+1}/C_j, around 1.0 for all adjacent levels) resulted in better energy dissipation throughout all storeys.

3 – PROJECT DESCRIPTION

The Python program developed internally allows for implementation of different design assumptions within the framework of the ESFP and provides full control of factors and variables considered in design. Assumptions relevant to this study include isolating the effects of cumulative rotations for predicting lateral deflections in seismic design. For this purpose, two sets of results will be presented: first, amplifying the total elastic deflections (i.e., including the cumulative rotations) by R_dR_o, and second, excluding the cumulative rotations from this amplification. With the lack of research quantifying cumulative rotations in the elastic and inelastic ranges, the assumption that rotations need not be scaled by R_d or R_o is considered here as a lower-bound solution.

In this study, the practicability of designing shear walls with a 20% overcapacity, as recommended in [5], is also investigated. This assumption will be compared with the current design process in the *Results* section.

Prior to using the Python program to assess the impact of different design assumptions, its output was validated through a third-party peer-review. The third-party engineering firm modelled a hypothetical 6-storey LFW building using Response Spectrum Analysis (RSA). The results output by the Python program, including the period, forces, and elastic deflection values, were compared with those from the RSA, showing good agreement.

Following the validation of the Python program output, the design of the shear walls was deemed appropriate for evaluation under simulated earthquakes using non-linear dynamic analysis. This allowed for the examination of the assumptions proposed in this paper. While the program was developed and validated for the elastic design of a full building, the present study was conducted assuming an archetype 2-segment 6-storey LFW shearwall. This archetype was deemed sufficient for the current purposes mainly due to the computationally intensive nature of full-building non-linear dynamic analyses. A full building analysis where tortional effects can be captured will be examined in future work.

The 2 segments studied, denoted as SG1 and SG2, have respective lengths of 4.8 m and 5.1 m with a height of 2.78 m. As the rotation component is dependent on the wall length, i.e., minimal for long walls and more prominent for short walls [13], an average length of around 5 m was used in this study. The segment lengths were selected to ensure that no segment is governed solely by bending behaviour (i.e., too short) or shear behaviour (i.e., too long). In addition, the segment lengths were chosen to be similar but not identical to investigate the effect of stiffness-based force distribution. This can be further illustrated by using identical wall construction details for a given storey, as this will result in slightly different overcapacity between segments. The construction details for each segment are given in Table 1. Note that continuous hold-down rods were used in this study because they are commonly employed for seismic design in mid-rise LFW buildings.

Table 1: Wall configuration for SG1 and SG2

Lev-el	Sheathing			Nails		Hold-down rod[a] (ATS)
	Thick-ness (mm)	*Type[b]*	*# of lay-ers*	*Spac-ing (mm)*	*Dia-meter (mm)*	
1	12.5	DFP	2	100	3.33	HSR8
2	18.5	DFP	2	125	3.33	HSR7
3	9.5	DFP	2	100	3.25	HSR7
4	18.5	DFP	1	75	3.25	HSR7
5	15.5	DFP	1	100	3.33	HSR7
6	12.5	DFP	1	125	2.84	HSR7

a) Rod cross-sections are 307.4 mm^2 for HSR7 and 402.3 mm^2 for HSR8

b) DFP type refers to Douglas-fir plywood

Sheathing thickness and nail spacing were chosen such that the amount of overcapacity was limited to around 20% in each segment. This was done to prevent additional over-design beyond this ratio, as discussed in the *Results* section.

The results for the archetype SFRS under different design assumptions are compared with those of a non-linear dynamic analysis. The non-linear model was developed in OpenSees and uses available test data to represent the deformation behaviour of the shear wall segments and hold-downs. The model is used to estimate the structural response of the SFRS in the inelastic range [14] when subjected to 11 different ground motions. It generates demand parameters such as storey drift, accelerations, yielding of components, and force demands [15]. These demand parameters were used to evaluate the collapse risk of the archetype SFRS. The non-linear dynamic analysis results are provided herein to observe and discuss conservatism in current seismic design requirements.

4 – DESIGN PROCESS

4.1 ESFP using mechanics-based period

The design process in this study consists of two distinct iterative procedures to determine seismic shear forces and lateral deflections: strength design, in which 2xTa (given in eq. 1) was used, and drift check, for which the actual period of the building was used.

For strength design, 2xTa was used as this is the longest period allowed by the NBC, and therefore the design force represents the lower bound permitted by the Code when wall details are chosen to minimize overdesign. The building period was then used to determine the spectral acceleration from the design spectrum presented in Fig. 2, and the ESFP described in the NBC was employed to determine the design base shear (V). The calculated base shear is distributed at each level proportionally to their height and weight, giving the force at storey j (F_j). Starting from the upper level, these forces are cumulated to determine the shear force to be resisted by the shear wall segments at each storey (V_j), which is then distributed to each segment (V_{sg}) based on their stiffness. To determine the lateral deflection, the 4-term deflection equation is applied (Δ_{SG}). Since stiffness is derived using V_{sg}/Δ_{SG}, and Δ_{SG} is a function of V_{sg}, an iterative process is needed until the deflections of all segments are within the specified tolerance. The tolerance set for convergence of the deflection of segments was ±0.6 mm.

For the drift check, the same process described above for strength design was employed. The deflection obtained from that iteration is used to calculate the building fundamental period, using the Rayleigh method, presented in (5).

$$T\ (s) = 2\pi \sqrt{\frac{\sum_{j=1}^{n} w_j \delta_j^2}{g \sum_{j=1}^{n} F_j \delta_j}} \qquad (5)$$

Where j is the storey, w_j is the storey weight, δ_j is the cumulative deflection, g is gravity, and F_j is the lateral force at each level.

Since the base shear is a function of the building period, the Rayleigh period (T) is then used to calculate a new design base shear. The iterative process to equalize the deflection of segments within a shearline is repeated to calculate an updated period, and so on. Once the period from two consecutive iterations of the overall process is within the chosen tolerance (±0.01 seconds), the iterations are complete, and the final T is applied to deflection check. Note that if the calculated period is shorter than 2xTa, this final period must be used in the strength design. However, this was not the case in this study.

4.2 Non-linear model

A non-linear model of the 2-segment 6-storey LFW shearwall was developed using OpenSees. The segments were modelled at each storey using rigid elements at boundaries that are connected using pin connections, and a non-linear shear hinge in the middle, as shown in Fig. 3.

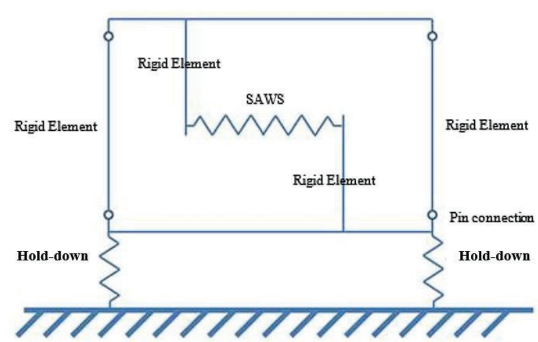

Figure 3. OpenSees model of the LFW shear wall

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To model the hold-downs, a zero-length element was added at each end of the shear wall. The hold-downs were modelled using elastic elements, for which the stiffness in both tension and compression were based on the wall details; they account for tension stiffness of the hold-down and compression stiffness at end-posts (including compressive stiffness parallel and perpendicular to grain), respectively. The properties of the shear hinge were modeled using the SAWS element [16] in OpenSees. The SAWS model was calibrated using experimental data from shear wall tests conducted by FPInnovations [17], as shown in Fig. 4.

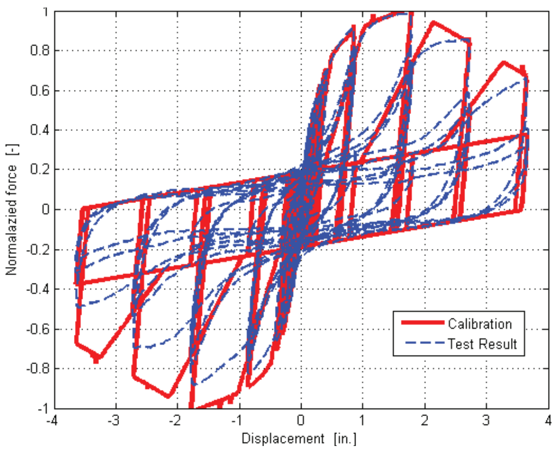

Figure 4. Comparison of the hysteresis model from the analytical and experimental data

The peak capacity of the SAWS model was based on each segment's factored resistance, with an additional over-strength factor of 1.6 [17]. The nodal mass and nodal gravity loads were determined using seismic weight at each level and gravity load tributary to each segment, respectively (see Table 2).

Table 2: Nodal Mass and Nodal Gravity Load of shearwall segments

Level	Nodal Mass (kg)	Nodal Gravity Load (kN)	
		SG1	SG2
1	18625	16.64	17.87
2	18625	16.84	18.08
3	18625	17.11	18.36
4	18625	17.50	18.77
5	17618	17.57	18.82
6	18021	15.47	16.69

The shear wall model was 2-dimensional, with walls along the same line tied together using equal degree-of-freedom constraints to simulate the drag strut effect. Gravity loads were assigned to the model using lump forces at the nodes on the wall at each storey. Rayleigh damping of 2.5% was assigned to the 1st and 3rd mode. The P-Delta geometric transformation was used to model the nonlinear geometry transformation in this study.

Eleven ground motions were selected and scaled to match the target spectrum of the prototype site in Vancouver using NBC 2020 [2] for a period between 0.15 s to 1.0 s. The fundamental period resulting from modal analysis was found to be 0.78s. The ground motions were applied to the shear wall model in the in-plane direction. Fig. 5 shows the mean of the scaled response spectra (blue-dashed line) against the target spectrum (black line).

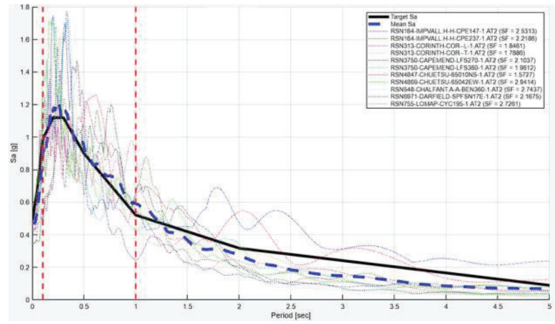

Figure 5. Scaled response spectra

In the interest of brevity, only the RSN313_CORINTH_COR--L ground motion was used for comparison in this paper.

4.3 Investigated design assumptions

Multiple design assumptions were considered when comparing the design results to the non-linear dynamic analysis to investigate their conservatism. The assumptions relevant to this study are as follows:

1) The wall configuration ensured around 20% overcapacity relative to shear forces applied on each segment, as recommended by [5].

2) The predicted lateral deflections were determined without amplifying the cumulative rotations (i.e., R_dR_o factors not applied cumulative rotations) to better represent the realistic racking behaviour of LFW shear walls in the design process.

3) An overcapacity ratio of approximately 1.0 between adjacent storeys (C_{i+1}/C_i) was maintained to ensure uniform energy dissipation throughout the height of the building.

4) The period used for strength design was 2xTa (i.e., 0.83s) while the calculated period was used for drift design (T = 1.03s is lower than 2s).

5 – RESULTS & DISCUSSION

5.1 Predicted lateral deflection

The Python program was used to determine the building period, and subsequently the forces distributed to segment 1 (SG1) and segment 2 (SG2) at each level. Since both segments had a similar length and the same wall details, the shear forces are comparable, as shown in Table 3. The results from Table 3 were generated with a period capped at 2xTa, using the design assumptions described in the previous section.

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Table 3: Shear force (vsg = Vsg/Ls), capacity (vrsg) and overcapacity of shear wall segments

Level	SG1			SG2		
	v_{sg} (kN/m)	vr_{sg} (kN/m)	Over-capacity	v_{sg} (kN/m)	vr_{sg} (kN/m)	Over-capaciy
1	13.57	16.61	1.22	13.92	16.61	1.19
2	12.99	16.00	1.23	13.24	16.00	1.21
3	11.71	14.52	1.24	11.99	14.52	1.21
4	9.91	12.08	1.22	10.01	12.08	1.21
5	7.42	9.11	1.23	7.46	9.11	1.22
6	4.45	5.49	1.23	4.46	5.49	1.23

Table 4 displays the deflection without rotation (Δ), the cumulative lateral deflection from α and θ rotations (Δ_α and Δ_θ, respectively), and the total deflection including cumulative rotations (Δ_{Tot}).

The predicted lateral deflection is also presented in Table 4; the penultimate and ultimate columns respectively display:

- Lateral deflections determined without amplifying the cumulative rotations by R_dR_o, and;
- Total lateral deflections (i.e., including cumulative rotations) amplified by R_dR_o

As observed, the former is well below the maximum Code drift limit for the building studied (i.e., within 2.5% storey height), while the latter slightly exceed this limit at the upper levels.

Table 4: Elastic and predicted lateral deflection of SG1

Level	Elastic lateral deflection (mm)				Predicted lateral deflection (mm)	
	Δ	Δ_α	Δ_θ	Δ_{Tot}	$(\Delta * R_dR_o) + (\Delta_\alpha + \Delta_\theta)$	$\Delta_{Tot} * R_dR_o$
1	6.75	0.00	0.00	6.75	34.43	34.43
2	7.42	1.77	0.67	9.86	40.28	50.30
3	6.24	3.50	1.32	11.06	36.64	56.41
4	6.90	4.82	1.94	13.66	41.95	69.67
5	5.76	5.62	2.33	13.71	37.33	69.92
6	5.58	5.98	2.53	14.09	36.97	71.86

When cumulative rotations are amplified by R_dR_o, the upper storeys are most affected, as there is between 66% to 94% greater predicted lateral deflections at levels 4 to 6. These results show the considerable contribution of cumulative rotations when they are assumed to exist in the inelastic range. While this paper does not question the existence of cumulative rotations, it is discussed that they may not be expected to go beyond the elastic range. The Annex approach assumes rocking behaviour in shear wall segments, while the proposed assumptions mainly account for the racking behaviour that is characteristic of LFW shear walls. The nailed connections are intended to yield and enable energy dissipation mainly in racking movement [18]. This energy is dissipated through internal friction, yielding of ductile connections, and irreversible deformations within the wall assembly [19].

5.2 Inelastic deflection from non-linear dynamic analysis

In Figure 6 and Table 5, the maximum lateral deflection in the non-linear analysis, measured at the peak force experienced by the shear wall segment, is compared to the predicted lateral deflections discussed in the previous section.

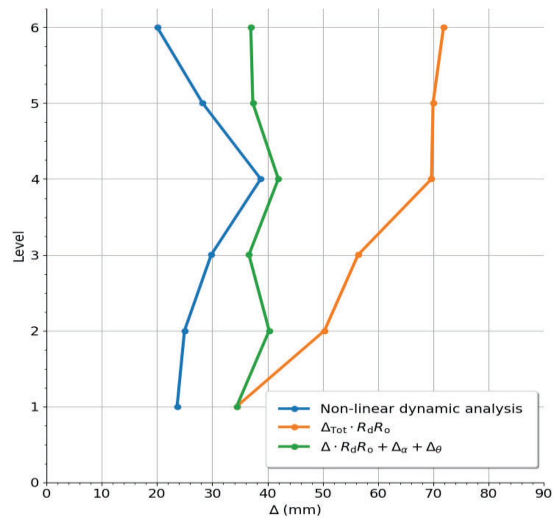

Figure 6. Cumulative lateral deflection comparison (Corinth EQ)

Table 5: Difference between predicted lateral deflection and non-linear dynamic analysis

Level	Non-linear dynamic analysis deflection	$(\Delta * R_dR_o) + (\Delta_\alpha + \Delta_\theta)$	$\Delta_{Tot} * R_dR_o$
		Difference vs non-linear analysis	
1	24 mm	+43%	+43%
2	25 mm	+61%	+101%
3	30 mm	+22%	+88%
4	39 mm	+8%	+79%
5	28 mm	+33%	+150%
6	20 mm	+85%	+259%

As shown, amplifying the cumulative rotations to obtain predicted lateral deflections considerably overestimates the inelastic deflections relative to the non-linear dynamic analysis. Since the energy is mainly dissipated through the racking behaviour of LFW shearwalls, the deflections estimated in the non-linear model are dominated by the nail-slip behaviour [20]. This is especially the case once a segment is acting within the inelastic range. The shear wall sheathing-to-framing connections will yield, and the shear wall will experience minor further rotation; however, the rotations may still behave within the elastic range and the rotational deformation will be small in comparison to the inelastic racking behaviour. The deflection calculation approach in Annex A assumes rigid body rocking movement as the dominant kinematic mode when determining the cumulative rotations. Furthermore,

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multiplying the resulting total lateral deflection by R_dR_o effectively means that such a rigid-body movement continues into the inelastic range.

Additionally, other system effects, such as diaphragm out-of-plane stiffness, shear connections, presence of hold-downs, and gravity loads will reduce uplift and further promote the racking movement. As their effects were not well-understood, their contributions were not considered in CSA O86 Annex A cumulative rotation equations [12].

Although the diaphragm's out-of-plane stiffness was not included in the non-linear analysis nor considered in the CSA O86 Annex deflection equations, it will have an influence on the rotation components of LFW shear wall deflection. If the diaphragm is assumed to be infinitely flexible in the out-of-plane direction, it can be expected to deform and allow rotation at the storey interface. However, if the diaphragm is assumed to be rigid, the cumulative rotations will be restricted to an extent that they may be considered negligible [21]. As further observed in recent research [22], the semi-rigid behaviour of the diaphragm, in addition to gravity loads, transverse walls, and other system effects, may result in minimal rotations.

The present results indicate better agreement with the non-linear analysis, in terms of deflection values and trend, when the cumulative rotations are not assumed to continue in the inelastic range. Additionally, the predicated lateral deflections using the proposed assumptions remained conservative when compared to the non-linear dynamic analysis on all levels. No collapse was identified in the non-linear dynamic analysis for all 11 ground motions when tested using the construction details based on the proposed assumptions, and the deflections were within acceptable limits at all storeys. Furthermore, the continuous hold-downs in the model did not experience forces exceeding their design capacity.

5.3 Design of shear walls with 20% overcapacity versus 20% increase in the base shear

Since the previous sections focused on the predicted lateral deflections, the 20% increase in the base shear required by the NBC [2] for using a mechanics-based period was not considered. Such an increase directly impacts the lateral deflection calculation, as will be discussed in this section. Therefore, the approach of using 100% of the base shear corresponding to the mechanics-based period and designing walls with 20% overcapacity was preferred to investigate the design assumptions of Section 4.3.

An additional investigation was conducted to compare the influence of increasing the base shear by 20% when performing both strength and drift design, and the approach using the assumptions mentioned in the previous section. Consequently, two initial cases were developed:

Case 1 – No increase in the base shear and ensure 20% overcapacity in the design of shear wall segments.

Case 2a – Include a 20% increase in the base shear applied to deflection calculation without attempting to comply with the drift limit requirement. This case was used as a reference point.

When minimizing overdesign of shear wall segments (beyond the intended capacity or overcapacity above), both cases resulted in the same wall details, i.e., same wall capacities shown in Table 3. Note that for both cases, the R_dR_o factors have been applied on total deflection (i.e., including cumulative rotations) to assess the impact of increasing base shear in drift check.

The predicted deflections for Case 1 and Case 2a are shown in Fig. 7. For the 2-segment multi-storey shear wall investigated in this study, designing the building with a 20% increase in the base shear resulted in 31% to 36% storey drift increase at all levels, compared to Case 1. However, when seeking compliance with the maximum drift requirement (2.5% of the storey height), one option is to increase the capacity of shear walls. As evidenced by Case 2b, the capacity of shear walls was considerably increased at all levels to achieve lateral deflections within the NBC drift limits. Increasing the hold-down rod size alone was not sufficient with such a high lateral demand.

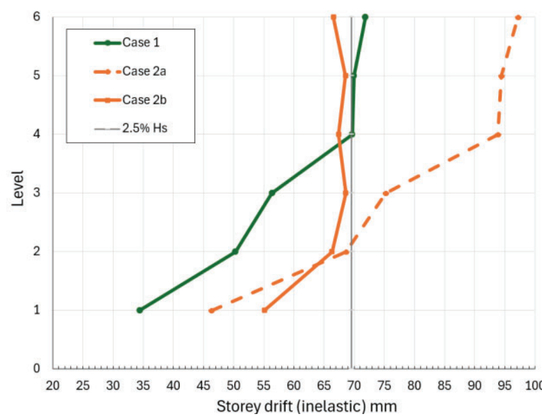

Figure 7. Predicted lateral deflections for Case 1, Case 2a and Case 2b

The shear forces, capacity and overcapacity ratios for Case 2b are shown in Table 6.

Table 6 – Case 2b shear forces ($v_{sg} = V_{sg}/L_s$) resulting from base shear increased by 20%, capacity (vr_{sg}) and overcapacity at each storey

Level	SG1[a]			SG2[a]		
	v_{sg} (kN/m)	vr_{sg} (kN/m)	Over-capaciy	v_{sg} (kN/m)	vr_{sg} (kN/m)	Over-capaciy
1	16.30	19.81	1.22	16.68	19.81	1.19
2	15.66	18.22	1.16	15.81	18.22	1.15
3	14.16	19.81	1.40	14.29	19.81	1.39
4	11.90	19.81	1.66	12.01	19.81	1.65
5	8.91	13.42	1.51	8.95	13.42	1.50
6	5.34	9.79	1.83	5.36	9.79	1.83

a) 2 x HSR10 rods were used on all levels, for which the total cross-section is 1286.5 mm^2

As shown in Table 6, meeting the deflection requirements resulted in up to 83% overcapacity, which was more prominent at upper storeys. Since the purpose of the 20% increase in the base shear is to avoid weak-storey occurrences, its effect on deflection calculation may be inadvertent. Variations in the overdesign of shear walls at adjacent storeys may also affect the efficiency of the SFRS in energy dissipation. The deflection profile throughout the height for Case 2b is shown in Fig. 7, where the deflection in upper levels was controlled

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through design to remain within the maximum limit. It is worth noting that the overcapacity ratio requirement in CSA O86 (C_{j+1}/C_j between 0.9 and 1.2) was satisfied for all levels. The non-linear behaviour of Case 2b is shown in Fig. 8 through the hysteresis curves resulting for the ground motion RSN313_CORINTH_COR--L.

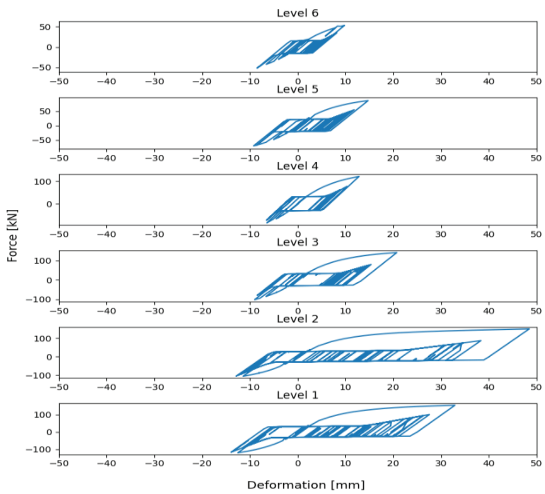

Figure 8. Case 2b hysteresis curve for the RSN313_CORINTH_COR--L ground motion

It can be observed that energy dissipation is predominantly concentrated on lower levels, with larger lateral deflection in the 2nd storey where the overcapacity is below the adjacent storeys (Table 6). Although the building showed no signs of collapse, it cannot be confirmed whether this would be the case for other ground motions, as the remaining ground motions were not analysed for Case 2b. If the design level is exceeded, a collapse is more likely to happen with such a distribution of energy dissipation. This would defeat the initial purpose behind increasing the base shear by 20% if applied to both strength and deflection design.

The hysteresis curves for Case 2b also emphasizes the importance of a parabolic strength profile along the height of the building, which is more difficult to achieve when the lateral deflection calculations are highly conservative, as shown in Fig. 9.

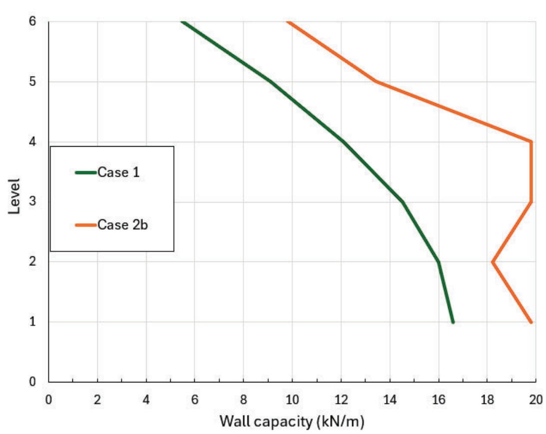

Figure 9 – Wall capacity comparison between Case 1 and Case 2b

In Case 2b, shear wall capacities at Levels 3 to 6 increased by 36% to 78% compared to Case 1. Hysteresis curves in all storeys for Case 1 3are presented in Fig. 10.

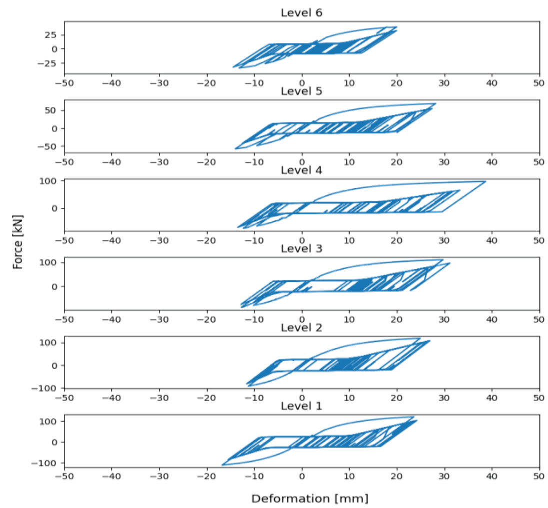

Figure 10. Case 1 hysteresis curve for the RSN313_CORINTH_COR--L ground motion

For shear walls having a strength profile proportional to the lateral force at all storeys (Case 1), the hysteresis curve demonstrates a uniform energy distribution along the height of the building. This suggests that a more efficient design was achieved, since all storeys contribute to the energy dissipation, limiting the probability of a weak-storey mechanism.

An additional point of interest is that applying a 20% increase to the base shear in strength design only would have resulted in similar wall details to Case 1. Further increasing the overcapacity is indeed beneficial to prevent collapse, provided that it results in a capacity profile proportional to applied forces. However, by applying the 20% increase in base shear to deflection calculation, there may be too little focus on preventing such capacity profiles that could cause a weak-storey mechanism. Since the non-linear dynamic analysis showed no collapse for all 11 ground motions and similar uniform energy dissipation was observed in other ground motions, Case 1 design may be considered more appropriate to improve efficiency. More research on more archetypes is needed to confirm the observations of this paper, and will be included in future work.

The results suggest that over-conservatism in deflection calculation may have detrimental effects on strength design and negatively impact the efficiency of seismic design in LFW midrise buildings, particularly in locations where seismic loading is so high that deflection limits become hard to meet.

The requirement to increase the base shear by 20% was not as detrimental when it was first recommended in 2009 [4] and prior to the first increase in seismic hazard values in NBC 2015. With an additional increase in NBC 2020, the approach used to determine lateral deflection has become more critical, and the current results further emphasize the importance of avoiding negative impacts on strength design while striving to meet the deflection limits.

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6 – CONCLUSION

Midrise LFW buildings are a good solution for the housing crisis in Canada. However, the increased seismic hazard values in NBC 2020 presented challenges to the feasibility and efficiency of midrise LFW buildings in moderate- to high-seismic zones, mainly due to overly conservative lateral deflections. This study investigated sources of undue conservatism when determining lateral deflections (e.g. effects of including cumulative rotations). For this purpose, a Python program was developed and peer-reviewed, which was used to design a 2-segment 6-storey LFW shear wall and to determine lateral deflections at each storey according to the following rationalized assumptions:

- Shear walls designed with 20% overcapacity relative to the seismic shear forces;
- Predicted lateral deflection determined without amplifying cumulative rotations by R_dR_o, and;
- Overcapacity ratios ensured to be around 1.0 between all adjacent storeys.

The design was evaluated in a non-linear time history dynamic analysis that used available test data to represent the anticipated behaviour of the shear wall segments under 11 ground motions scaled to the design level in the studied location. In this paper, results of only one ground motion were presented. The non-linear analysis results were compared with predicted values, and design assumptions of this study were discussed.

An additional investigation was carried out related to the influence of increasing the base shear by 20% when performing the drift design compared to designing shear walls with 20% overcapacity.

The following are the key observations of this study:

1. When cumulative rotations are assumed to go beyond the elastic range (i.e., amplified by R_dR_o factors) the lateral deflections are 25% to 94% greater in Level 2 to Level 6 of the archetype shear wall compared to rotations assumed to be limited to the elastic range (see Fig. 6).

2. There was a better agreement, in values and trend, with the lateral deflections from the non-linear dynamic analysis when the rotation components were assumed to remain in the elastic range (i.e., not amplified by R_dR_o factors). Moreover, the calculated lateral deflections using this assumption were larger than those from the non-linear dynamic analysis (see Fig.6).

3. For the same wall details, a 20% increase in the base shear led to a 31% to 36% increase in lateral deflections compared to a design where the base shear was not modified (see Fig. 7). This shows how challenging it is for designers to meet drift requirements.

4. By overdesigning shear walls at the upper storeys to reduce predicted lateral deflections as a solution to meet the drift requirements (in addition to larger hold-down sizes), energy dissipation became concentrated at the lower storeys (see Fig. 8). A parabolic strength profile corresponding to over-capacity ratios around 1.0 between adjacent storeys showed better energy dissipation throughout the building height (see Fig. 10).

The rationalized assumptions considered in this study to reduce undue conservatism in the seismic drift design resulted in a more efficient design, with lateral deflections comparable to those obtained from the non-linear dynamic analysis. For all 11 ground motions used in the non-linear dynamic analysis, the LFW shear walls showed no sign of collapse.

An evaluation under non-linear dynamic analysis of the current design procedure (i.e., applying a 20% increase in the base shear on deflection calculation and amplifying the cumulative rotations in the calculated lateral deflections by R_dR_o) was performed. Results showed a concentration of energy dissipation in the lower storeys, although the high degree of overdesign present in the current design procedure showed no-collapse under the 11 ground motions scaled to design level.

Future research will include designing a full midrise LFW building, assuming both rigid and flexible diaphragm assumptions. The 3-dimensional design will be validated with non-linear dynamic analyses and used to further evaluate the assumptions discussed herein. The effects of cumulative rotation in multi-segment multi-storey shear walls need to be quantified and investigated experimentally to confirm values predicted in this study.

Limitations of this investigative study include the low variability in the segment length. Furthermore, the accuracy of the non-linear dynamic analysis results depends on the accuracy of the model.

7 – ACKNOWLEDGEMENT

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