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Contribution of Plasterboard Finishes to Structural Performance of Multi-storey Light Wood Frame Buildings

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Abstract

The main objective of this paper is to investigate the contribution of plasterboard finishes made of gypsum wall board (GWB) to the structural performance of multi-storey light wood frame building (LWFB) subjected to earthquake load. Four- to six-storey buildings were analysed in this study. Computer software, SAPWood, developed to analyze LWFB subjected to actual earthquake motions was used. Two cases were considered in the analyses. The first one was a reference case where all shear walls are fabricated with wood-based sheathing panels only. The second case was buildings with walls fabricated with wood-based sheathing panels plus GWB. All shear wall hysteretic properties for both cases (with and without GWB) and inter-storey (hold-down) connections were derived from detailed numerical modeling of wall sub-systems available in the SAPWood database. The buildings were subjected to a major earthquake ground motion excitation, and the ground motion was scaled until failure in the components (walls or hold-down connections) or excessive inter-storey drift was reached. Main outputs that were used as comparison between the two cases included natural period, maximum storey shear force and drift, and individual wall responses (force and deformation). Specific attention was paid to how the applied forces are distributed between the different types of wall panels i.e. wood-based and gypsum-based.

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Keywords: multi-storey wood building; shear wall; gypsum wall board; earthquake load, drift.

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1. INTRODUCTION

Light wood frame buildings (LWFB) represent more than 90% of the residential construction in North America, mostly in the form of single detached family houses and low-rise multi-storey apartments up to four-storey high. However, recently the province of British Columbia in Canada changed its building code regulations to allow 5- and 6-storey LWFB to be constructed. The design of these taller LWFB presents some challenges to engineers in term of strength and serviceability limit states. Either as single or multi-storey, LWFB are normally constructed from several diaphragms such as walls, floors, and roofs; interconnected by nails, metal plates, anchor bolts, and other proprietary fasteners to form a light plated structure that is efficient in resisting gravity and lateral loads.

Wall components in LWFB that are designed to carry lateral loads are practically called shear walls. Exterior wood shear walls are normally composed of wood studs sheathed on one side with structural wood-based panels such as Oriented Strand Board (OSB) or plywood, and on the other side with finish materials such as gypsum wall board (GWB). Nails are often used to fasten the wood-based panels to the wood-frame, while nails or screws are used to connect the drywall to the wood stud frame. Metal connectors such as hold downs can be added to the wall ends to resist overturning forces generated from lateral loads. The strength and stiffness of shear walls normally depend on nail-slip characteristic between the sheathing panels and frame, anchorage deformation, bending deformation of the lumber studs and shear deformation of sheathing panels.

Although typically not considered as the primary component to resist lateral loads, GWB can appreciably affect the overall structural performance of buildings (Uang and Gatto, 2003). Moreover, from an economic point of view, most of the cost of damage repairs after an earthquake event has been spent on the drywall replacement. The main objective of this study is to investigate the structural responses of multi-storey LWFB when GWB is considered as part of shear wall components that resist lateral loads due to earthquakes. The main focus was on LWFB taller than three storeys, since there have been many test data and analytical studies on GWB effects for residential buildings of one to three storeys height. The main vehicle in this study was numerical modeling of multi-storey LWFB subjected to earthquake motions using software called SAPWood developed by Pei and van de Lindt (2007). All shear wall hysteretic properties for both cases (OSB only and OSB+GWB) were derived from separate analyses using detailed numerical models of wall sub-systems available in that software.

2. STRUCTURAL (SEISMIC) ANALYSES OF LWFB

Four-, five- and six-storey buildings with the typical floor layout shown in Figure 1 were analysed in this study. The six-storey structure is a designed building taken from Association of Professional Engineers and Geoscientists of British Columbia (APEGBC) bulletin of design guidance and construction practice (APEGBC, 2009). Because of symmetry, only half of the building layout is shown. The locations of shear walls are also shown in the layout. The shear wall height for each building was assumed to be the same, 2.77m; and their lengths are given in Table 1. In total, there are 34 shear walls, 18 in the X-direction, and 16 in the Y-direction. Two analysis cases were performed: buildings with the shear walls consisting of wood based sheathing panels made of OSB on one side and GWB panels on the other side of the wood frame; and buildings with only OSB on one side of the frame as benchmark.

The wood wall frame used consisted of 2x6 Douglas-Fir lumber with edge nailing patterns shown in Table 2 for each wall at each storey of the buildings. It should be noted that a denser edge-nailing pattern was applied at the lower storeys of the building to resist the larger base shear forces due to earthquake load. The intermediate (field) nail spacing used for the 12 mm OSB panels was the same for all storeys, 300 mm on centres (o.c.). All GWB panels used were 12 mm thick (1/2 inch) and connected to the wood stud frame using screws at 300 mm o.c. Steel hold-downs (E=200 GPa) were utilized at the wall ends, with 25.4 mm-diameter (1 inch) bolts applied at 5th to 6th storeys, and 32 mm-diameter bolts for 1st to 4th storey levels. Non-structural walls were not incorporated in this study, but their weight contribution was included in the structural analysis. Total estimated weight for each floor was 129.6 kN, and for the roof was 97.9 kN.

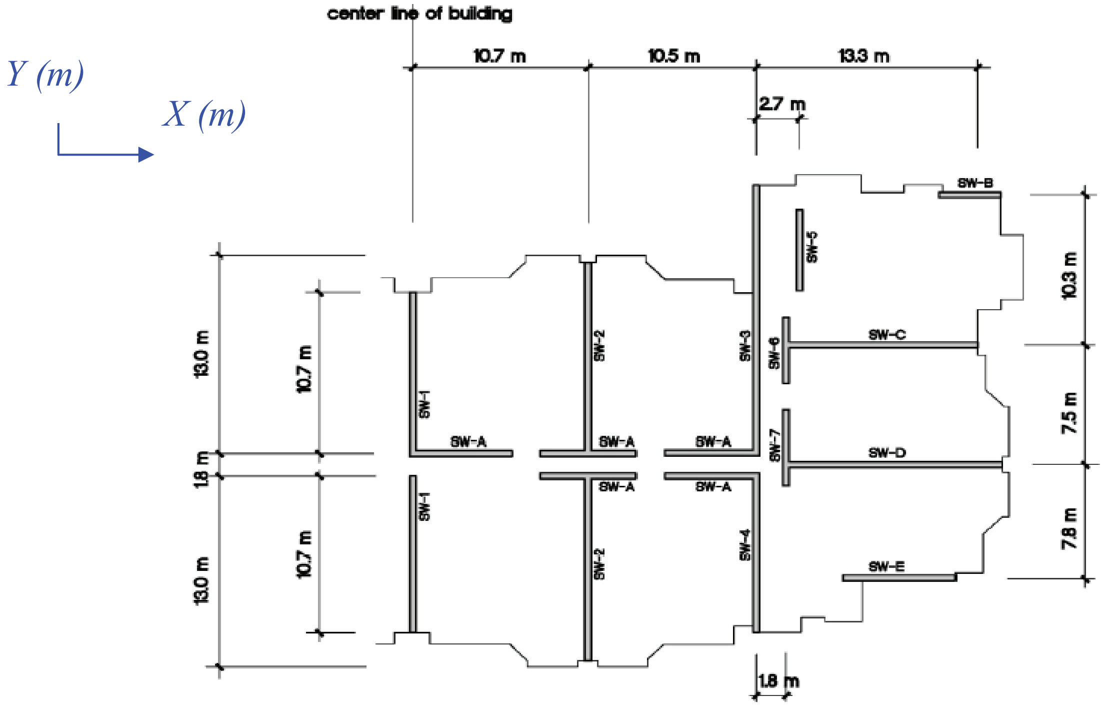


Figure 1: Typical (half) building layout (APEGBC, 2009)

Major numerical input for the structural (seismic) analysis using SAPWood software includes diaphragm coordinates, shear wall geometry (locations, lengths, and height), shear wall properties, hold-down properties, masses for the diaphragms and walls, and time-domain earthquake ground motions in the orthogonal directions. Figure 2 illustrates the modeling philosophy used in the SAPWood program. In essence, the actual three-dimensional building is degenerated into a two-dimensional planar model composed of 'zero-height' shear wall elements connecting the floor or roof diaphragms together or to the foundation. The model assumes that all diaphragms in this building can be idealized as rigid. Irregular shape of diaphragms can be incorporated in the model by inputting their respected (corner) coordinates. In SAPWood, each diaphragm movement was represented by 3 degrees-of-freedom, i.e. two translations and one rotation about vertical axis. Each shear wall spring element can be calibrated to reflect the hysteretic response under lateral cyclic load including strength and stiffness degradation, and pinching effect. For the bi-linear spring elements, the axial stiffness value of wood stud in compression was 14.0 kN/mm, and that of the hold-down in tension was 14.0 kN/mm and 10.7 kN/mm for 1st - 3rd storeys and 4th - 6th storeys respectively.

Table 1: Shear wall lengths

Direction	Shear wall mark (see Fig. 2)	Length (m)	Number of shear walls
X	SW-A	6.07	10
	SW-B	3.66	2
	SW-C	10.97	2
	SW-D	12.19	2
	SW-E	7.62	2
Y	SW-1	6.10	2
	SW-2	12.95	4
	SW-3	16.92	2
	SW-4	10.67	2
	SW-5	6.1	2
	SW-6	4.27	2
	SW-7	4.88	2

Table 2: Nail diameter (D)* and edge nail spacing (S) for the shear walls

Storey	4-storey building		5-storey building		6-storey building	
	D (mm)	S (mm)	D (mm)	S (mm)	D (mm)	S (mm)
1	3.76	50.8	3.76	50.8	3.76	50.8
2	3.40	50.8	3.76	50.8	3.76	50.8
3	3.40	76.2	3.40	50.8	3.40	50.8
4	3.40	101.6	3.40	76.2	3.40	50.8
5	-	-	3.40	101.6	3.40	76.2
6	-	-	-	-	3.40	101.6

Note: *Common nail

There are two ways to obtain shear wall hysteretic springs: by conducting wall tests or numerical modeling. Since in this study there are various shear wall lengths in the buildings, the numerical model option available in SAPWood database was used to obtain the hysteretic wall parameters. Data input required in addition to the shear wall geometry (sheathing panel thickness, length and height) is nail connection properties (nail diameters, and edge and intermediate nail spacing). Figure 3 illustrates a typical shear wall response under cyclic load and curve fitting procedure to extract hysteretic response parameters. Figure 3b shows an illustrative example of extracting the response into 10 hysteretic parameters for numerical modeling input. Table 3 gives selected response parameters (initial stiffness K_0, resistance force parameter F_0, pinching residual resistance force F_1, slip at maximum restoring force X_u, and stiffness factors r_1-r_4) for selected shear walls both for OSB and GWB panels. Note that in the case of OSB plus GWB shear walls, the locations of OSB and GWB panels were virtually made the same in the modeling input geometry. The load sharing characteristic is then dependent on their respective hysteretic responses.

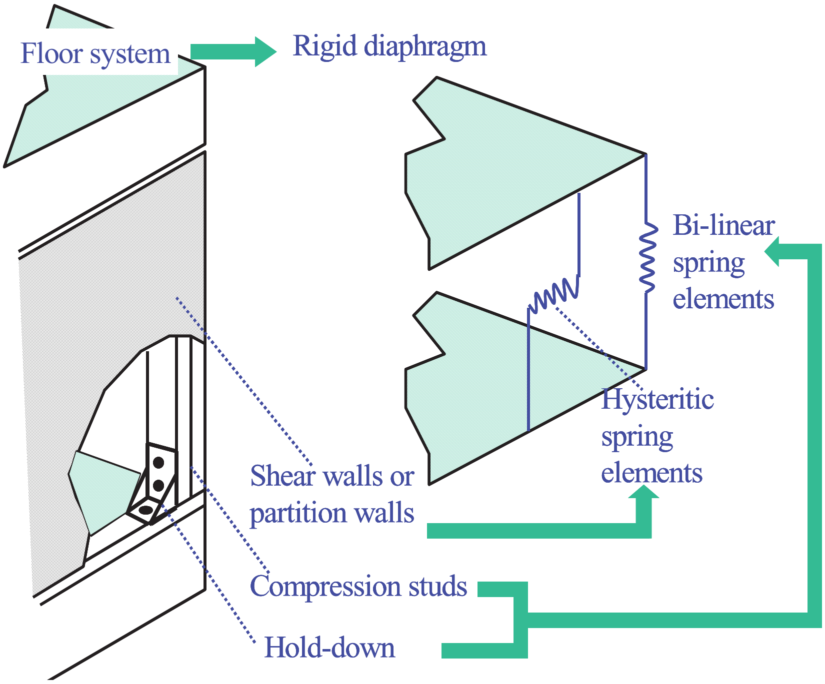
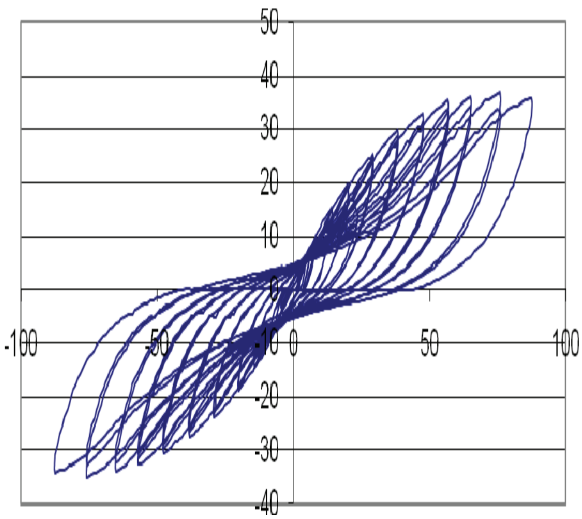
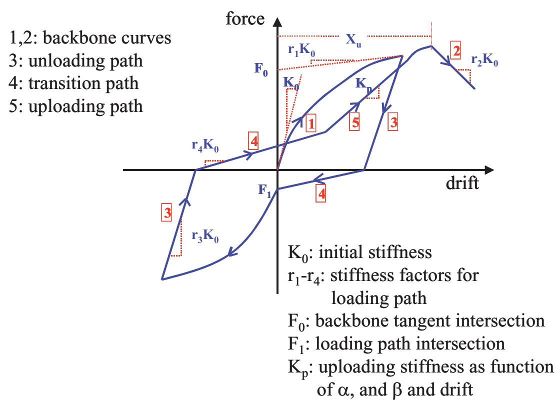


Figure 2: SAPWood model (Pei and van de Lindt, 2007)



(a) Typical load-displacement response



(b) curve fitting using 10 hysteretic parameters

Figure 3: Shear wall behaviour under cyclic loading (Folz and Filiatraut, 2004)

Table 3: Hysteretic parameters for selected shear walls (as generated from SAPWood)

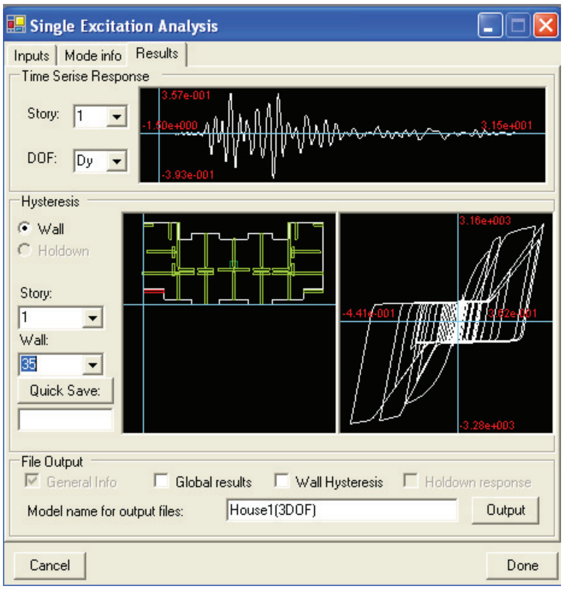
Shear wall panels	Storey *)	Shear wall mark*)	K_0 (kN/mm)	F_0 (kN)	F_1 (kN)	X_u (mm)	r_1	r_2	r_3	r_4
OSB only	1	SW-D	60.3	359.6	55.5	21.0	0.01	-0.13	1.00	0.0004
	1	SW-3	86.8	494.2	76.5	21.0	0.01	-0.11	1.00	0.0004
	3	SW-D	24.0	230.7	36.8	50.8	0.01	-0.05	1.00	0.0004
	3	SW-3	33.6	323.6	51.1	50.8	0.01	-0.05	1.00	0.0004
	6	SW-D	24.0	255.6	39.5	48.9	0.01	-0.05	1.00	0.0004
	6	SW-3	17.1	184.8	28.5	53.3	0.01	-0.05	1.00	0.0004
GWB only	1	SW-D	11.6	17.8	4.0	27.0	0.029	-0.017	1.00	0.005
	1	SW-3	16.0	24.7	5.5	27.0	0.029	-0.017	1.00	0.005
	3	SW-D	11.6	17.8	4.0	27.0	0.029	-0.017	1.00	0.005
	3	SW-3	16.0	24.7	5.5	27.0	0.029	-0.017	1.00	0.005
	6	SW-D	11.6	17.8	4.0	27.0	0.029	-0.017	1.00	0.005
	6	SW-3	16.0	24.7	5.5	27.0	0.029	-0.017	1.00	0.005

Note: *) see Table 1 for the shear wall geometry information and Table 2 for the nailing pattern.

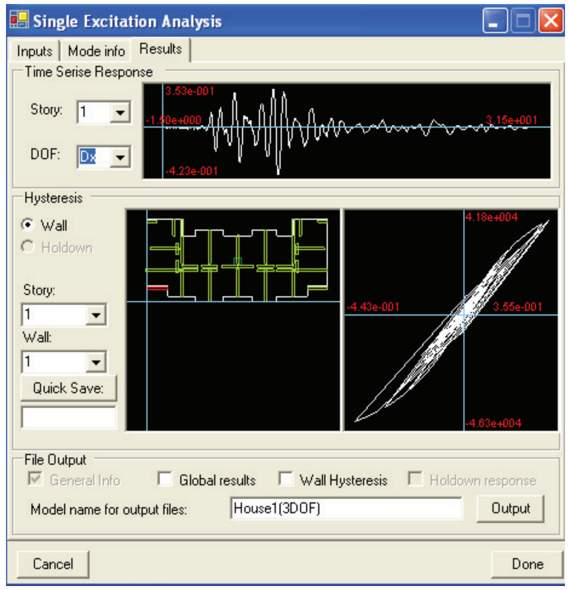
3. ANALYSIS RESULTS AND DISCUSSION

Time domain analysis was performed using an actual earthquake record that was applied in the orthogonal directions of the buildings. No coupling behaviour between these orthogonal directions was incorporated in the analysis. A 30 second-duration of the Northridge earthquake excitation with the peak ground acceleration (PGA) = 0.16g was applied to the buildings (PEER, 2000). Only one earthquake record was run in this study, and this PGA was scaled up progressively until failure or excessive drift was observed. In SAPWood, failure can be assumed to occur in the structural system when the calculation results in numerical instability (i.e., demonstrated by unusually large, unrealistic natural period or excessive drifts) generated at the shear wall components. The largest inter-storey drift developed can be used as a first indicator to locate failure initiation. Similarly, when several wall hysteretic responses are drifting too far from the original input values (e.g. peak load is exceeded), then failure can be assumed to have taken place.

There was no vertical acceleration applied. A time step of 0.005 sec was used to ensure convergence in the numerical analysis, and 0.2 sec time interval was used for calculating the maximum average acceleration at each diaphragm. There was no concentrated mass applied in the models, and the masses were assumed uniformly distributed over the floor and roof diaphragms. A damping ratio of 2% was assumed in the structural system analysis. Figure 4 illustrates an example of output analysis showing drift history on the first floor and load-displacement responses of OSB (left-figure) and GWB (right-figure) faces located at the same shear wall location subjected to this earthquake excitation. From this specific example, it can be observed that the GWB wall has almost reached its fully hysteretic response capacity, while for the OSB face fully hysteretic response accompanied with large energy dissipation has not been reached. This could indicate that damage (or failure) is generated first in the GWB component.



Building layout



Building layout

Figure 4: Examples of analysis output

3.1. Natural period

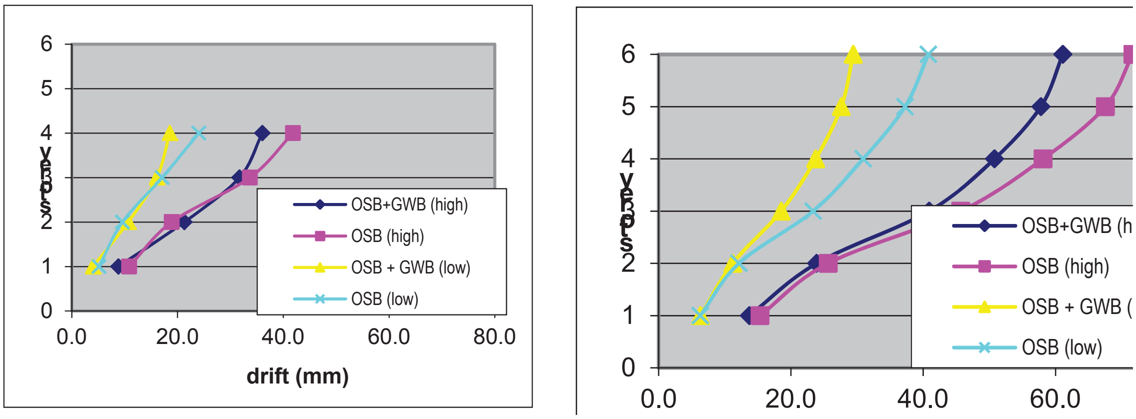
Natural period of the structure under two conditions were calculated: ‘initial’ and ‘current’. The initial natural period corresponds to the initial stage (initial ground shaking applied), while the current value is after ground shaking fully applied. Table 4 summarizes the results for the three types of buildings analysed, with the earthquake record scaled to 0.3g. As expected, the current natural period shows larger values than the initial ones indicating decreased stiffness in the structural systems. Inclusion of GWB in the analysis resulted in lower natural periods than those with OSB only, indicating that there is an increase in the stiffness relative to the mass of the structures. It should be noted that using the empirical equation given in the National Building Code of Canada (NRCC 2005) for light wood-frame buildings, $T=0.05 \times h^{0.75}$ where h is the building height in m, the calculated natural period is substantially lower than those shown in Table 4 (0.32s, 0.38s and 0.44s for the 4-, 5- and 6-storey building respectively).

Table 4: Fundamental period (seconds)

Shear wall sheathing	4-storey building		5-storey building		6-storey building	
	Initial	Current	Initial	Current	Initial	Current
OSB	0.50	0.62	0.57	0.75	0.69	1.01
OSB+GWB	0.43	0.46	0.50	0.58	0.60	0.67

3.2. Storey drift

Figure 5 summarizes the maximum drifts for the 4- and 6-storey buildings under low-scaled Northridge earthquake motion (PGA=0.16g) and high-scaled motions (scaled to 0.3g) that resulted in failure. For the 6-storey buildings (both cases), the failure was initiated in the 3rd storey indicated by the largest drift developed and several load-displacement responses of the walls that drifted too far from their input (properties) values (e.g. post-peak load was reached). For the 4 and 5-storey buildings, the failure was initiated at the 2nd storey for the same reason. In general it can be noted that by incorporating GWB in the shear walls of the buildings, the drifts are reduced by 23-30% (low-scaled motion) and 14-25% (high-scaled motion).



(a) 4-storey building (b) 6-storey building

Figure 5: Drifts at the low and high-scale earthquake motions ("high"= high-scaled motion PGA= 0.3g), "low" = low-scale motion, PGA=0.16g)

3.3. Storey shear force distribution

The maximum shear forces generated in the OSB and GWB faces within the shear walls can be extracted from the output generated by SAPWood. Table 5 summarizes the percent storey forces carried by GWB for the three buildings both at the low and high-scaled earthquake motions. Also shown in the table are the percent values of initial GWB stiffness over the stiffness of OSB+GWB wall for each storey. In the first storey, where the shear storey forces are the largest, less than 10 % of the applied force is carried by GWB, while at the highest storey, almost 30% of the storey force is distributed to GWB for the low-scaled earthquake motion, and 20 % for the high-scaled motion. Therefore, the storey forces distributed to GWB are reduced moving from the low to high earthquake load conditions. This is largely because GWB wall has lower post-elastic deformation capacity compared to the OSB wall resulting in larger forces distributed to the OSB face.

Table 5: Percent storey forces carried by GWB *)

Storey	4-storey building			5-storey building			6-storey building		
	low	high	K_{GWB}/K_{tot} **	low	high	K_{GWB}/K_{tot} **	low	high	K_{GWB}/K_{tot} **
1	9	6	16	8	6	16	8	6	16
2	13	9	28	9	8	16	8	7	16
3	18	11	45	13	10	28	11	9	28
4	29	20	54	17	12	45	14	10	28
5	-	-	-	27	21	54	19	14	45
6	-	-	-	-	-	-	27	23	54
Notes: *) It was assumed that the remainder was carried by OSB; ** $K_{tot}=K_{GWB}+K_{OSB}$									

With respect to structural design code, Canadian practice allows the use of GWB-sheathed shear walls in platform-frame wood construction to resist shear due to lateral forces based on given specified strength (CSA, 2009). Currently the Canadian timber design code, CSA O86-09, includes provisions regarding the maximum percent storey shear distributions to GWB components in buildings up to four-storey in height. The factored shear resistance provided by the GWB-sheathed walls shall be less than the specified percentage of storey shear forces. According to Table 9.5.4 of CSAO86-09, as much as 80%, 60%, 40%, and 40% of the respective 4th, 3rd, 2nd, and 1st storey shear forces can be carried by walls with GWB. Due to the relative stiffness of OSB and GWB walls, as is shown in Table 5, it appears that the percent force absorbed by GWB is smaller than that given in the CSA O86-09. However, it should be noted that the analysed buildings were designed with seismic modification factor of $R_d*R_o=3*1.7 = 5.1$, which essentially ignores the structural contribution of GWB. For GWB to be considered as part of lateral load carrying system, $R_d*R_o=2*1.7=3.4$ shall be used, thereby increasing the design load by 33%. (Note: R_d= ductility factor, and R_o=over-strength factor, by which the specified earthquake load is divided.). Based on this preliminary study, it appears that the allowable percent shear force that is resisted by GWB should be calculated based on the relative stiffness between GWB-sheathed and wood panel-sheathed walls, and not the storey level as stipulated in CSA O86-09. Work is currently underway to derive the percent values of GWB contribution based on its ductility characteristics and relative stiffness.

4. CONCLUSIONS

It can be concluded that GWB used in combination with wood-based structural sheathing (e.g. OSB) in shear walls affects the structural performance of multi-storey LWFB subjected to dynamic earthquake load. Incorporating GWB in the analysis leads to stiffer structures and smaller drifts and natural periods compared to cases where only wood-based panels are used. Therefore, provision for including GWB contribution in estimating natural period (e.g. for base shear calculation) is needed. The percent force resisted by GWB is a direct result of the relative stiffness of the OSB- and GWB-sheathed wall, and appears independent of storey height. Provisions to suggest percent values are needed based on the ductility and relative stiffness of GWB and OSB walls. Future work will include investigating various GWB wall stiffnesses by changing the fastening schedule.

5. ACKNOWLEDGMENTS

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