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Design Example: Design of Stacked Multi-Storey Wood-Based Shear Walls Using a Mechanics-Based Approach

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Acknowledgements
This publication was developed by FPInnovations and the Canadian Wood Council based on design and construction practice and relevant research. This publication would not have been possible without the financial support of Forestry Innovation Investment of the Province of British Columbia.

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INTRODUCTION

Figure 1 shows a floor plan and elevation along with the preliminary shear wall locations for a six-storey wood-frame building. It is assumed some preliminary calculations have been provided to determine the approximate length of wall required to resist the lateral seismic loads.

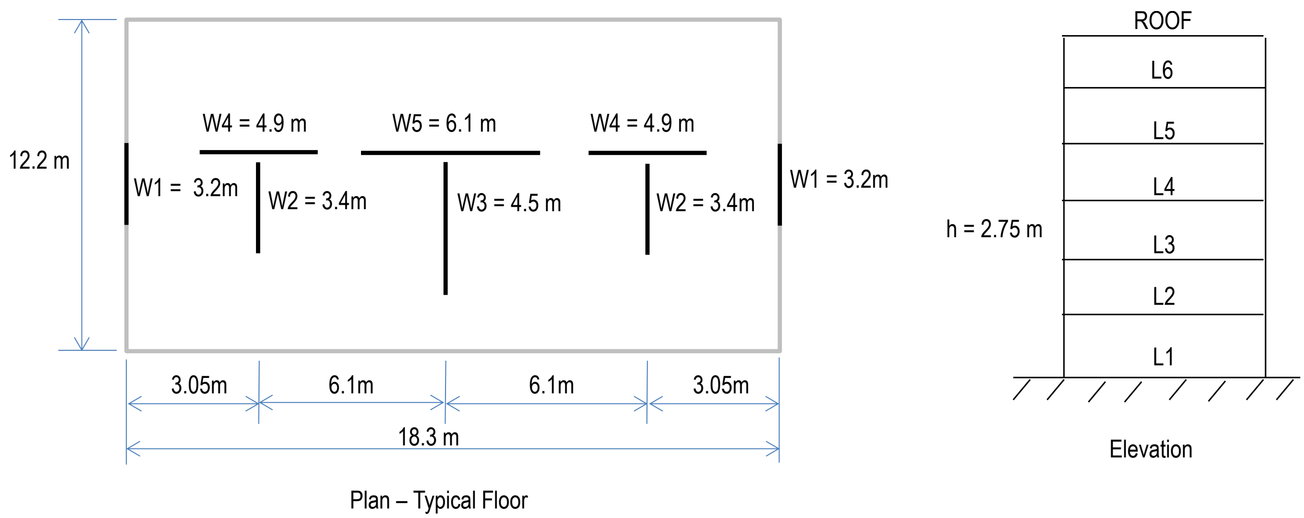

Figure 1. Plan and elevation view of the design building.

Assumptions

- Building area A = 12.2 m x 18.3 m
- Seismic mass W = 300 kN (roof) (Included dead load and 25% snow)
  W = 350 kN (per floor)
  ∑W = 300 + 350x5 = 2050 kN
- Floor to floor h = 2.75 m
- Building is regular – according to the 2012 BC Building Code
- Building is located in Vancouver
  - $S_a(0.2) = 0.94$
  - $S_a(0.5) = 0.64$
  - $S_a(1.0) = 0.33$
  - $S_a(2.0) = 0.17$
- Site Class C soils
- Importance factor = 1.0

Design

1. Based on the 2010 NBCC, the seismic loads can be determined using the equivalent static procedure as follows:

- Building height $h_n = 6 \times 2.75 = 16.5$ m
- Building period $T_a = 0.05 \cdot h_n^{0.75} = 0.05 \times 16.5^{0.75} = 0.409$ s
- $S(0.409) = \dfrac{0.94 - 0.64}{0.2 - 0.5} \times (0.409 - 0.5) + 0.64 = 0.731$
- Code formulas for base shear are as follows:

$$V_{\min} = \frac{S(4.0) M_v I_E W}{R_d R_o} = \frac{0.085 \times 1.0 \times 1.0 \times W}{1.7 \times 3.0} = 0.017W$$

$$V = \frac{S(T_a) M_v I_E W}{R_d R_o} = \frac{0.731 \times 1.0 \times 1.0 \times W}{1.7 \times 3.0} = 0.143W$$

$$V_{\max} = \frac{2/3 \times S(0.2) M_v I_E W}{R_d R_o} = \frac{2/3 \times 0.94 \times 1.0 \times 1.0 \times W}{1.7 \times 3.0} = 0.123W \longleftarrow$$ Governs design

2. Based on experience, it is likely that the building period *T*, is higher than the Code-based formula T_a. Therefore, the seismic forces can be recalculated assuming $T = 2 \times T_a$ which is the Code cut-off for determining base shears. Using $T = 2 \times T_a = 0.819$, the forces can be recalculated as follows:

- $S(0.819) = \dfrac{0.64 - 0.33}{0.5 - 1.0} \times (0.819 - 1.0) + 0.33 = 0.443$
- Code formulas for base shear are as follows:

$$V_{\min} = \frac{S(4.0) M_v I_E W}{R_d R_o} = \frac{0.085 \times 1.0 \times 1.0 \times W}{1.7 \times 3.0} = 0.017W$$

$$V = \frac{S(T) M_v I_E W}{R_d R_o} = \frac{0.443 \times 1.0 \times 1.0 \times W}{1.7 \times 3.0} = 0.087W \longleftarrow$$ Governs design

$$V_{\max} = \frac{2/3 \times S(0.2) M_v I_E W}{R_d R_o} = \frac{2/3 \times 0.94 \times 1.0 \times 1.0 \times W}{1.7 \times 3.0} = 0.123W$$

However, as a result of recent Code changes in the 2012 BC Building Code, the minimum base shear for wood-based shear walls needs to be increased by 20% to reduce the risk of sway-storey seismic behaviour when the building period is calculated by methods other than the Code-specified period. Therefore:

- V = 0.087 W × 1.2 = 0.104 W

  Which corresponds to T = 0.675 s on the design response spectrum.

Therefore: *V = 0.104 W = 0.104 × 2050 = 213 kN*

NS: Lw = 17.8 m, → $v = V / L_w = 12.0$ kN/m

EW: Lw = 15.9 m, → $v = V / L_w = 13.4$ kN/m

The initial distribution of shear forces over the height of the building can be determined in accordance with 4.1.8.11.6) of 2010 NBCC and is shown in Figure 2.

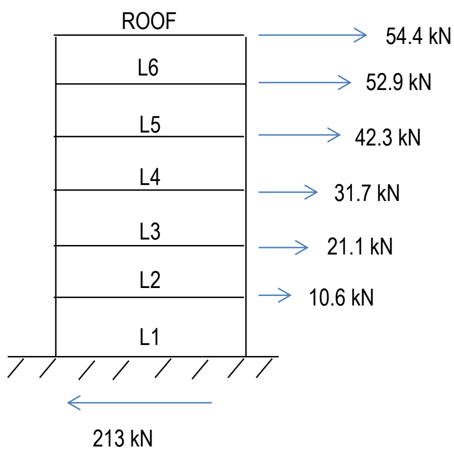

Figure 2. Initial vertical distribution of seismic forces

3. The initial distribution of forces to the various walls can be determined as follows.

Case 1 (flexible diaphragm)

- Assuming flexible diaphragms

$$V_{w1} = \frac{3.05/2}{(3.05 \times 2 + 6.1 \times 2)} \times V = 0.083\,\mathrm{V}$$

$$V_{w2} = \frac{(3.05/2 + 6.1/2)}{(3.05 \times 2 + 6.1 \times 2)} \times V = 0.250\,\mathrm{V}$$

$$V_{w3} = \frac{(6.1/2 + 6.1/2)}{(3.05 \times 2 + 6.1 \times 2)} \times V = 0.333\,\mathrm{V}$$

- As suggested by the Code commentary, for structures with flexible diaphragms accidental torsion should be taken into account by moving the centre of mass by 5% of the plan building dimension perpendicular to the seismic load, as shown in Figure 3. The largest of the seismic loads should be used for the design of each vertical element.

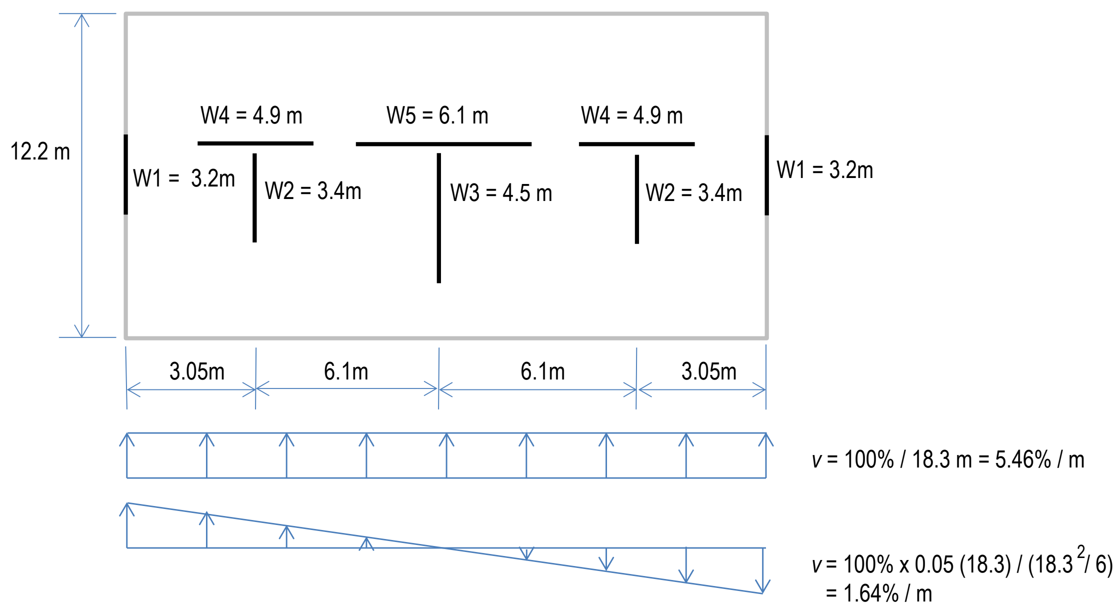

Figure 3. Force distribution of accidental torsion.

- Therefore, the resulting maximum wall forces are as follows:

$$V_{w1} = 0.083 + 0.0164 \times \left(1 + \frac{9.15 - 3.05/2}{9.15}\right) \times \frac{3.05}{2} \times \frac{1}{2} = 0.106\,\mathrm{V}$$

$$V_{w2} = 0.25 + 0.0164 \times \left(\frac{9.15 - 3.05/2}{9.15} + \frac{6.1/2}{9.15}\right) \times \frac{(3.05 + 6.1)}{2} \times \frac{1}{2} = 0.294\,\mathrm{V}$$

$$V_{w3} = 0.333\mathrm{V}$$

Case 2 (rigid diaphragm / wall stiffness proportional to length)

- Assuming the wall stiffness is proportional to the wall length:

$$V_{w1} = \frac{3.2}{(3.2\times 2+3.4\times 2+4.5)}\times V = 0.181\text{V}$$

$$V_{w2} = \frac{3.4}{(3.2\times 2+3.4\times 2+4.5)}\times V = 0.192\text{V}$$

$$V_{w3} = \frac{4.5}{(3.2\times 2+3.4\times 2+4.5)}\times V = 0.254\text{V}$$

Furthermore, the additional shear as a result of torsional moment due to the offset between the centre of rigidity and centre of mass (0.0 m in this example), in addition to the minimum prescribed Code eccentricity of 10% of the building dimension, can be determined as follows and is shown in Figure 4:

$$M_{tor} = (0.0 \pm 0.1\times 18.3)\times V = \pm 1.83\text{V}$$

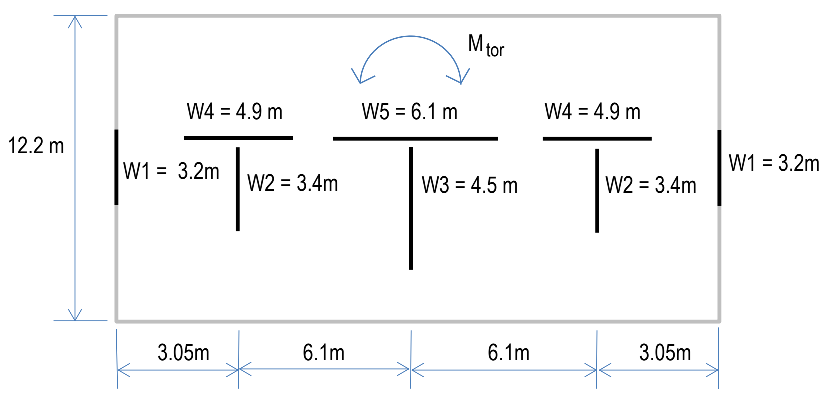

Figure 4. Torsional moment due to ecentricity and accidental torsion.

For a rigid diaphragm, the lateral force distributed to supporting shear wall *i* can be determined as follows:

$$V_i = \frac{F\times k_i}{\sum k} + \frac{M_{tor}\times k_i \times d_i}{J}$$

where

k = wall stiffness, N/mm

d = distance from the wall to the centre of rigidity (CoR), mm

M_{tor} = torsional moment

F = total lateral load on the supporting shear walls

$J = \sum kd_x^2 + \sum kd_y^2$

$$V_{w1} = 0.181 \pm \frac{1.83 \times 3.2 \times 9.15}{3.2 \times 9.15^2 \times 2 + 3.4 \times 6.1^2 \times 2} = 0.181 \pm 0.068\,\text{V}$$

$$V_{w2} = 0.192 \pm \frac{1.83 \times 3.4 \times 6.1}{3.2 \times 9.15^2 \times 2 + 3.4 \times 6.1^2 \times 2} = 0.192 \pm 0.048\,\text{V}$$

$$V_{w3} = 0.254\,\text{V}$$

Case 3 (envelope approach based on Cases 1 and 2)

- In the APEGBC Technical and Practice Bulletin (APEGBC 2011), it is recommended that the walls should be designed for the envelope forces if the force in any wall is different by more than 15% due to the change in flexible and rigid diaphragm assumptions.

Table 1 summarizes Cases 1 and 2, and the initial design force selected for Case 3.

Table 1. Distribution of seismic force to walls - (×V)

Wall	Case 1	Case 2	Case 3
W1	0.106	0.249	0.25
W2	0.294	0.240	0.29
W3	0.333	0.254	0.33
W4[1]	-	0.307	0.30
W5[1]	-	0.386	0.40

Note: Case 1 is not applicable as the walls align and carry 100% of the total load in the x direction.

4. Based on Case 3, determine the initial design force for each wall and perform an initial design for each wall in accordance with the design provisions of CSA O86 – 2009.

To illustrate this, Wall 1 will be used.

- Multiplying the inter-storey force V by 25% based on Case 3, the seismic forces on Wall 1 are shown in Figure 5:

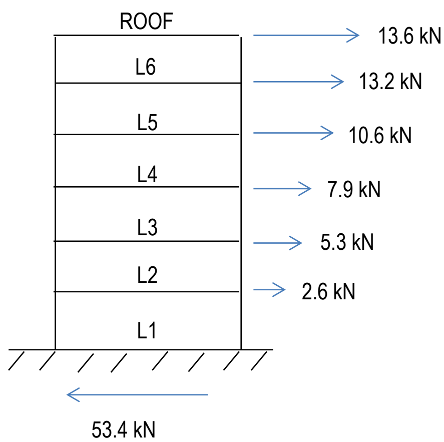

Figure 5. Initial design forces for Wall 1.

- The shear design and flexural design are provided in Tables 2 and 3. For Wall 1, a continuous anchor rod system is used as hold-down.

Table 2. Design of shear resistance of Wall 1

Level	F [2] (kN)	ΣF (kN)	v_e [3] (kN/m)	Shear wall [1]			v_r [4] kN/m	v_r/v_e
				Panel thickness (mm)	Nail diameter (mm)	Nail spacing (mm)		
Roof	13.6	13.6	4.25	12.5	3.25	150	9.14	2.15
5th	13.2	26.8	8.38	12.5	3.25	150	9.14	1.09
4th	10.6	37.4	11.69	12.5	3.25	100	13.27	1.14
3rd	7.9	45.3	14.16	12.5	3.66	100	16.10	1.14
2nd	5.3	50.6	15.81	12.5	3.66	100	16.10	1.02
1st	2.6	53.2	16.63	12.5	3.66	100	16.10	0.97

Note:

1. Wall 1 consists of SPF framing and OSB panels sheathed on both sides of the framing.
2. F is the seismic force applied to each level, determined in accordance with Clause 4.1.8.11.6) of the National Building Code of Canada.
3. v_e is the factored seismic force applied on the shear walls.
4. v_r is the factored shear resistance of the selected shear walls in accordance with CSA O86.

Check the ratio of second storey to first storey over-capacity coefficients:

$$0.9 < \frac{C_2}{C_1} = \frac{1.02}{0.97} = 1.05 < 1.2 \text{, okay}$$

Table 3. Design of boundary members of Wall 1

Level	w_d	w_l	F	$\Sigma w_d \times L_w$	$\Sigma w_l \times L_w$	ΣF	M_f	1.2 T_f	1.2 C_f	Steel rod	T_r	Stud	$C_{r\perp}$
	kN/m	kN/m	kN	kN	kN	kN	kN.m	kN	kN		kN		kN
Roof	1.12	2.44	13.6	3.6	7.8	13.6	37.5	15.1	21.8	SR9	142.0	6-2x6	179.0
5th	3	3	13.2	13.2	17.4	26.9	111.3	43.5	64.5	SR9	142.0	6-2x6	179.0
4th	3	3	10.6	22.8	27.0	37.5	214.3	85.3	120.7	SR9	142.0	6-2x6	179.0
3rd	3	3	7.9	32.4	36.6	45.4	339.2	137.1	187.0	SR9	142.0	6-2x6	179.0
2nd	3	3	5.3	42.0	46.2	50.7	478.6	195.7	260.0	HSR9	303.7	10-2x6	298.3
1st	3	3	2.6	51.6	55.8	53.4	625.4	257.7	336.3	HSR9	303.7	12-2x6	358.0

Note:

1. w_d and w_l is the specified dead and live load on the wall, respectively.
2. M_f is the bending moment at the bottom of the shear wall.
3. L_s is the length of the shear wall, which is 3.2 m.
4. L_c is the distance between the centers of steel rods, which is 2.6 m.
5. $T_f = M_f / L_c - \sum(w_d \times L_w) / 2$
6. $C_f = M_f / L_c + \sum((w_d + 0.5w_l) \times L_w) / 2$
7. T_r is the tensile capacity of steel rod.
8. $C_{r\perp}$ is the compression perpendicular capacity of the wood plates.
9. For M_f, J_x as per Clause 4.1.8.11.7) of NBCC 2010 could have been used to reduce the overturning moment but was not for the purpose of this example.

The properties of the steel rods are provided in Table 4.

Table 4. Properties of the steel rods

ATS	Size	A_g (in^2)	A_e (in^2)	$(0.4A_g + 0.6A_e)$ (in^2)	$(0.4A_g + 0.6A_e)$ (mm^2)	T_r (kN)	MOE (MPa)
SR9	1 1/8	0.994	0.763	0.855	551.9	142.0	200
HSR9	1 1/8	0.994	0.763	0.855	551.9	303.7	200

Note:
For the purpose of calculating steel rod elongation, a modified area is used to take into account the threaded and non-threaded portion of the steel rod. Assuming that 40% of the steel rod is not threaded and 60% of the steel rod is threaded, the modified area $A_t = 0.6\ A_g + 0.4\ A_e$

The Wall 2, Wall 3, Wall 4, and Wall 5 can be determined following the same method.

5. Determine the deflection and the period *T* of each stacked shear wall based on the wall properties determined in Step 4.

According to Clause 4.1.8.11(3)(d)(v) of the National Building Code of Canada, the deflection can be calculated based on the base shear with the period determined in accordance with a mechanics-based approach without the 2 × T_a upper limit. Using the seismic forces determined in Step 4 as initial input data, the deflection can be calculated. The inter-storey deflection of stacked multi-storey shear walls can be determined as follows (Newfield et al. 2013):

$$\Delta_i = \frac{V_i H_i^3}{3(EI)_i} + \frac{M_i H_i^2}{2(EI)_i} + \frac{V_i}{L_i}\frac{H_i}{B_{v,i}} + 0.0025 H_i e_{n,i} + \frac{H_i}{L_i} d_{a,i} + H_i\left(\sum_{j=1}^{i-1}\theta_j + \sum_{j=1}^{i-1}\alpha_j\right)$$

where

V_i	=	shear force at level *i*
	=	$\sum_{j=i}^{n} F_j$
M_i	=	moment at level *i*
	=	$\sum_{j=i+1}^{n} V_j H_j$, $M_n = 0$
H_i	=	height of the shearwall at level *i*
L_i	=	length of the shearwall at level *i*
$(EI)_i$	=	bending stiffness of the shearwall at level *i*
$e_{n,i}$	=	nail deformation at level *i*
$d_{a,i}$	=	sum of vertical deformation due to the bearing of wood plates in compression and elongation of the wall anchorage system in tension at level *i*
$\sum_{j=1}^{i-1}\theta_j$	=	cumulative rotation due to bending from the levels below
$\sum_{j=1}^{i-1}\alpha_j$	=	cumulative rotation due to wall anchorage system elongation from the levels below

For shear walls using continuous steel rods as hold-downs (Figure 5), the transformed moment of inertia should be used (Newfield et al. 2013). This can be determined as follows:

$$E = E_c$$

$$n = \frac{E_t}{E_c}$$

$$A_{t,tr} = A_t \cdot n$$

$$y_{tr} = \frac{A_c \cdot L_c}{A_{t,tr} + A_c}$$

$$I_{tr} = A_{t,tr} \cdot y_{tr}^2 + A_c \cdot (L_c - y_{tr})^2$$

Where I_{tr} is the transformed moment of inertia of the shear wall in terms of E_c for compression where the tension cord area is transformed by $n = E_t / E_c$.

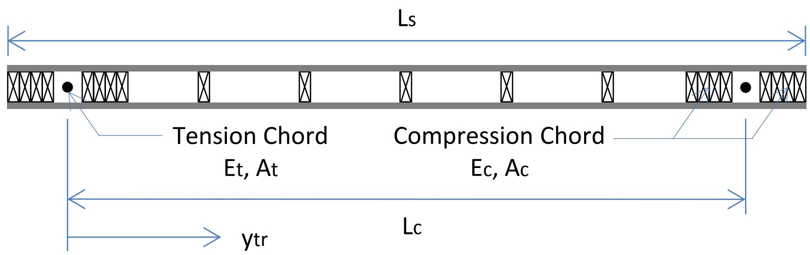

Figure 6. Shear wall section.

The transformed moment of inertia and natural axis are provided in Table 5.

Table 5. Moment of inertia and natural axis of Wall 1

Level	E_c	E_t	A_c	A_t	$A_{t,tr}$	y_{tr}	I_{tr}
	N/mm^2	N/mm^2	mm^2	mm^2	mm^2	mm	mm^4
Roof	9500	200000	31920	552	11618	1906	5.76E+10
5th	9500	200000	31920	552	11618	1906	5.76E+10
4th	9500	200000	31920	552	11618	1906	5.76E+10
3rd	9500	200000	31920	552	11618	1906	5.76E+10
2nd	9500	200000	53200	552	11618	2134	6.45E+10
1st	9500	200000	63840	552	11618	2200	6.64E+10

The deflection for each storey is summarized in Table 6.

Table 6. Deflection of Wall 1

Level	V	M	E	I_{tr}	B_v	v_e	e_n	d_a	$\Sigma d_a/L_s$	θ	$\Sigma\theta$	Δ_s
	N	N.mm	N/mm^2	mm^4	N/mm	N	mm	mm				mm
Roof	13621	0.00E+00	9500	5.76E+10	22000	319	0.165	0.2	2.63E-03	9.42E-05	6.83E-03	28.03
5th	26864	3.75E+07	9500	5.76E+10	22000	630	0.490	0.7	2.42E-03	3.74E-04	6.45E-03	29.99
4th	37459	1.11E+08	9500	5.76E+10	22000	585	0.422	1.3	2.03E-03	8.19E-04	5.63E-03	27.76
3rd	45405	2.14E+08	9500	5.76E+10	22000	709	0.443	2.0	1.41E-03	1.39E-03	4.24E-03	24.12
2nd	50702	3.39E+08	9500	6.45E+10	22000	792	0.559	2.1	7.71E-04	1.84E-03	2.40E-03	18.99
1st	53351	4.79E+08	9500	6.64E+10	22000	834	0.627	2.5	0.00E+00	2.40E-03	0.00E+00	11.97

Note:

Δ_s is the inter-storey drift.

$$\theta_i = \frac{M_i H_i}{(EI)_i} + \frac{V_i H_i^2}{2(EI)_i}$$

$d_{a,i} = \frac{T_{f,i}}{T_{r,i}} d_{\max} + \frac{C_{f,i}}{E_{c\perp,i} A_{c\perp,i}} L_{c\perp,i}$, in which the first term is the crushing of steel plate of the steel rod into wood plates on the tension side and the second term is the bearing deformation of the top and bottom wood plates of the wall on the compression side.

where

- $T_{f,i}$ - Tension load at level i
- $T_{r,i}$ - Steel rod capacity at level i
- $d_{max,i}$ - Steel plate crushing deformation at steel rod capacity at level i, which can be found from the product catalogues. In this design example, dmax is assumed as 1.0 mm for SR9 and 2.0 mm for HSR9.
- $C_{f,i}$ - Compression load at level i
- $E_{c\perp,i}$ - Modulus of elasticity perpendicular to grain of wood plates at level i. In this study, it is taken as 1/20 modulus of elasticity parallel to grain.
- $A_{c\perp,i}$ - Bearing area of the wood plates at level I. In this study, it is equal to the cross-section of end studs in compression.
- $L_{\perp,i}$ - Total bearing thickness of the wood plates at level i. A total of three wood plates (two top plate and one bottom plate) was assumed in this study.

According to FEMA 450 commentary (BSSC 2003), the period can be determined as follow:

$$T = 2\pi \sqrt{\frac{\sum_{i=1}^{n} w_i \Delta_i^2}{g \sum_{i=1}^{n} F_i \Delta_i}}$$

where

F_i = the seismic lateral force at level i

w_i = the seismic weight assigned to level i

Δ_i = the static lateral displacement at level i due to the force Fi computed on a linear elastic basis, and

g = the acceleration due to gravity

The period of Wall 1 is determined in Table 7.

Table 7. Fundamental period of Wall 1

Level	w_i	F_i	Δ_i	$w_i \cdot \Delta_i^2$	$F_i \cdot \Delta_i$
	kN	kN	mm	kN.mm^2	kN.mm
Roof	75	13.6	140.87	1488256	1919
5th	87.5	13.2	112.83	1113993	1494
4th	87.5	10.6	82.84	600520	878
3rd	87.5	7.9	55.08	265469	438
2nd	87.5	5.3	30.96	83862	164
1st	87.5	2.6	11.97	12535	32
Σ				3564635	4924

$$T = 2\pi\sqrt{\frac{3564635}{9800 \times 4924}} = 1.71 \text{ s}$$

As the nail deformation, e_n, is a nonlinear term which is dependent on the lateral force of the nail joint, the deflections need to be recalculated until the force on the nail joints converges.

Iteration round #1:

Using T = 1.71 s, the new base shear is obtained as follows:

$$S(1.71) = \frac{1.71 - 1.0}{2.0 - 1.0} \times (0.17 - 0.33) + 0.33 = 0.217$$

$$V = \frac{S(1.71) M_v I_E W}{R_d R_o} = \frac{0.217 \times 1.0 \times 1.0 \times W}{1.7 \times 3.0} = 0.043 \text{ W}$$

The seismic forces, deflection, and period are provided in Figure 7 and Tables 8 and 9.

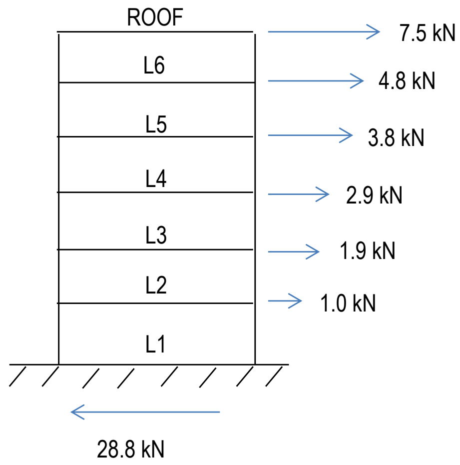

Figure 7. Vertical distribution of seismic forces (Iteration #1)

Table 8. Deflection of Wall 1 (Iteration #1)

Level	V	M	E	I_{tr}	Bv	v_e	e_n	d_a	$\Sigma\ d_a/L_s$	θ	$\Sigma\theta$	Δ_s
	N	N.mm	N/mm²	mm⁴	N/mm	N	mm	mm				mm
Roof	7505	0.00E+00	9500	5.76E+10	22000	176	0.088	0.1	1.19E-03	5.19E-05	3.04E-03	12.75
5th	12269	2.06E+07	9500	5.76E+10	22000	288	0.144	0.3	1.08E-03	1.89E-04	2.85E-03	12.89
4th	16080	5.44E+07	9500	5.76E+10	22000	251	0.126	0.6	8.94E-04	3.85E-04	2.47E-03	11.84
3rd	18939	9.86E+07	9500	5.76E+10	22000	296	0.118	0.9	6.07E-04	6.27E-04	1.84E-03	10.00
2nd	20844	1.51E+08	9500	6.45E+10	22000	326	0.135	0.9	3.27E-04	8.05E-04	1.04E-03	7.43
1st	21797	2.08E+08	9500	6.64E+10	22000	341	0.144	1.0	0.00E+00	1.04E-03	0.00E+00	4.23

Table 9. Fundamental period of Wall 1 (Iteration #1)

Level	w_i	F_i	Δ_i	$w_i \cdot \Delta_i^2$	$F_i \cdot \Delta_i$
	kN	kN	mm	kN.mm²	kN.mm
Roof	75	7.5	59.13	262270	444
5th	87.5	4.8	46.39	188298	221
4th	87.5	3.8	33.50	98210	128
3rd	87.5	2.9	21.66	41049	62
2nd	87.5	1.9	11.66	11894	22
1st	87.5	1.0	4.23	1564	4
Σ				603285	881

$$T = 2\pi\sqrt{\frac{603285}{9800 \times 881}} = 1.66\,\text{s}$$

Iteration round #2:

Using T = 1.66 s, the new base shear is obtained as follows:

$$S(1.66) = \frac{1.66 - 1.0}{2.0 - 1.0} \times (0.17 - 0.33) + 0.33 = 0.224$$

$$V = \frac{S(1.66) M_v I_E W}{R_d R_o} = \frac{0.224 \times 1.0 \times 1.0 \times W}{1.7 \times 3.0} = 0.044\,\text{W}$$

The seismic forces, deflection and period are provided in Figure 8 and Tables 10 and 11:

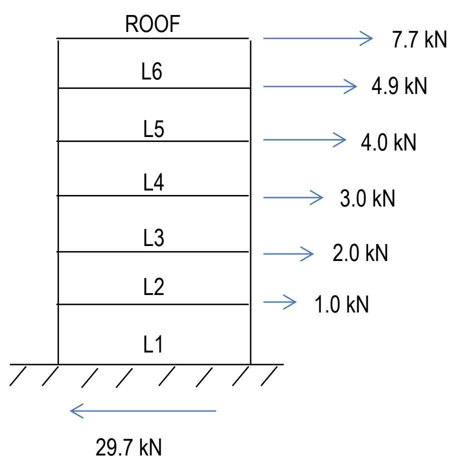

Figure 8. Vertical distribution of seismic forces (Iteration #2)

Table 10. Deflection of Wall 1 (Iteration #2)

Level	V	M	E	I_{tr}	B_v	v_e	e_n	d_a	$\Sigma d_a/Ls$	θ	$\Sigma\theta$	Δ_s
	N	N.mm	N/mm²	mm⁴	N/mm	N	mm	mm				mm
Roof	7707	0.00E+00	9500	5.76E+10	22000	181	0.090	0.1	1.23E-03	5.33E-05	3.14E-03	13.14
5th	12653	2.12E+07	9500	5.76E+10	22000	297	0.148	0.3	1.12E-03	1.94E-04	2.94E-03	13.29
4th	16609	5.60E+07	9500	5.76E+10	22000	260	0.130	0.6	9.23E-04	3.96E-04	2.55E-03	12.22
3rd	19576	1.02E+08	9500	5.76E+10	22000	306	0.124	0.9	6.27E-04	6.46E-04	1.90E-03	10.33
2nd	21555	1.55E+08	9500	6.45E+10	22000	337	0.142	0.9	3.38E-04	8.31E-04	1.07E-03	7.69
1st	22544	2.15E+08	9500	6.64E+10	22000	352	0.151	1.1	0.00E+00	1.07E-03	0.00E+00	4.38

Table 11. Fundamental period of Wall 1 (Iteration #2)					
Level	w_i	F_i	Δ_i	$w_i \cdot \Delta_i^2$	$F_i \cdot \Delta_i$
	kN	kN	mm	kN.mm²	kN.mm
Roof	75	7.7	61.07	279699	471
5th	87.5	4.9	47.93	200975	237
4th	87.5	4.0	34.63	104939	137
3rd	87.5	3.0	22.41	43938	66
2nd	87.5	2.0	12.08	12758	24
1st	87.5	1.0	4.38	1682	4
Σ				643990	939

$$T = 2\pi\sqrt{\frac{643990}{9800 \times 939}} = 1.66\,\text{s}$$

This is close enough to converge. In accordance with Clause 4.1.8.13.2) of the National Building Code of Canada 2010, lateral deflections shall be multiplied by $R_d R_o / I_E$ to give realistic values of anticipated deflections. Therefore, the deflections of the stacked multi-storey shear walls are as shown in Table 12:

Table 12. Inter-storey drift			
Level	**$\Delta_{elastic}$ (mm)**	**$\Delta \times R_d R_o$ (mm)**	**%**
Roof	13.14	67.0	2.4
5th	13.29	67.8	2.5
4th	12.22	62.3	2.3
3rd	10.33	52.7	1.9
2nd	7.69	39.2	1.4
1st	4.38	22.4	0.8

All the inter-storey drift ratios fall within the 2.5% limit specified in the National Building Code of Canada for normal type occupancy. It also suggests that using 2x the Code period is a reasonable assumption for the initial design. If the preliminary design could not meet the drift limit requirement using the base shear obtained based on the actual period, the shear walls should be re-designed until the drift limit requirement is satisfied.

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