greentech engineering

Chapter #6 DESIGN LOADS

Fruitland Vertical Farm and Marketplace

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Team 12

Chapter Lead(s):

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PROJECT OVERVIEW

The City of Hamilton has retained GreenTech Engineering (GreenTech) to complete the design and consultation for the Fruitland Vertical Farm and Marketplace located at the intersection of North Service Road and Fruitland Road in Stoney Creek, Ontario. The City of Hamilton's 2031 Master Plan (2015) identifies the need for sustainable infrastructure, with the goal of implementing innovative solutions for the problems threatening today's society. To fulfill this need, the City has chosen to implement a vertical farm in a community slated for urban development in the coming years.

The objective of the Fruitland Vertical Farm and Marketplace is to provide an alternate means of food production in a population-dense environment. The proposed undertaking will seek to act as a "sustainable landmark" within the City of Hamilton by implementing sustainable structural, stormwater, transportation, and geotechnical practices throughout its design and construction.

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6.1 DESIGN LOAD CONSIDERATIONS

Determining the design loads for the building is crucial for ensuring that the appropriate structural members are being used. Throughout the building's service life, various loads will be applied and thus it is important that the greatest loads within statistical reason are applied with the appropriate load combinations. For this reason, Greentech Engineering takes a conservative approach when calculating the design loads to ensure the building's structural members are resilient for present and possible future use of the building.

The following gravity and lateral loads are based off many factors such as the context of the building's location, environmental factors, occupational use and items in the building. After the design loads are calculated a structural analysis will be able to inform Greentech Engineering and other stakeholders if a change in the architectural and structural layout is required. These loads were calculated in compliance with NBCC 2015, Division 4, Part 4 (Structural Design).

6.2 GRAVITY LOADS

6.2.1 Dead Load

Dead load is defined as a permanent load due to the weight of building components, including weight of the members, weight of all materials of construction added into the building, weight of partition and permanent equipment, and weight of vertical load due to earth, plants and trees.

The resulted dead loads for Fruitland vertical farm and market place are summarized in Table 1, and detailed calculated can be found in Appendix A.

Level	Partition (kPa)	Floor Slab (kPa)	Fire protection (kPa)	Rack (kPa)	Structural framing (kPa)	Duct, pipe and wiring (kPa)	Cladding (kN/m)	Ceiling (kPa)
Roof	-	0.3	0.07	-	0.5	0.25	-	-
4th	1	1.84	0.07	0.2	0.5	0.25	4.8	-
3rd	1	1.84	0.07	0.2	0.5	0.25	4.8	-
2nd	1	1.84	0.07	0.2	0.5	0.25	4.8	0.2

Table 1: Summary of Dead Load

6.2.2 Live Load

Live load, according to NBCC 2015, is a variable load due to intended use and any occupancy (including loads due to cranes and the pressure liquid in containers). In practice, the greatest live load at each level is used to design for structural members and connections

Table 2 and 3 shows the possible live loads for each level and the specified live load will be used for design.

Level	Live Load (kPa)					
	Balcony	Restaurant (dining area)	Office Area (upper floor)	Service room	Equipment Room	Pumping room
Roof	-	-	-	-	-	-
4th	-	-	2.4	3.6	3.6	3.6
3rd	-	-	2.4	3.6	3.6	3.6
2nd	4.8	4.8	2.4	3.6	3.6	3.6

Table 2: Summary of Live Load 1

Level	Live Load (kPa)					
	Mechanical Equipment	Washroom	Roof	Kitchen	Retail and wholesale (rack)	Design Live Load
Roof	-	-	1	-	-	1
4th	3.6	2.4	-	-	4.8	4.8
3rd	3.6	2.4	-	-	4.8	4.8
2nd	3.6	2.4	-	4.8	4.8	4.8

Table 3: Summary of Live Load 2

6.2.3 Snow Load

Typically snow loads are the main design load for roofs and thus is the greatest concern for the structural designer when choosing appropriate sections. It is greater than the roof live load because the NBCC suggests the use of 1 kPa for live load whereas the 1 in 50-year ground snow load in the Hamilton area is 1.1 kPa and this is before the application of other coefficients.

As can be seen in Figure 1 below, Fruitland Vertical Farm and Marketplace consists of three different roof sections. The largest portion is labelled Roof 1 and it is a curved roof that is over the main vertical farm portion. Roof 2 is a flat roof that is over the main building but the office portion of the building. A flat portion of the roof was required for roof top units to be placed and maintained throughout the year which would not have been possible on the curved portion of the roof. Roof 3 is the roof that extends over the marketplace which is also a flat roof. The calculations for the snow load on each roof surface can be seen in Appendix A.

Roof 2 and Roof 3 both experience snow load drift which is responsible for the accumulation of snow which piles up when a difference in height is present between two neighboring elevations. The extents of the snow load distribution can be seen in Figure 1 below and the calculations for the drift load can be seen in Appendix A. Figure 2 depicts how this load would be distributed.

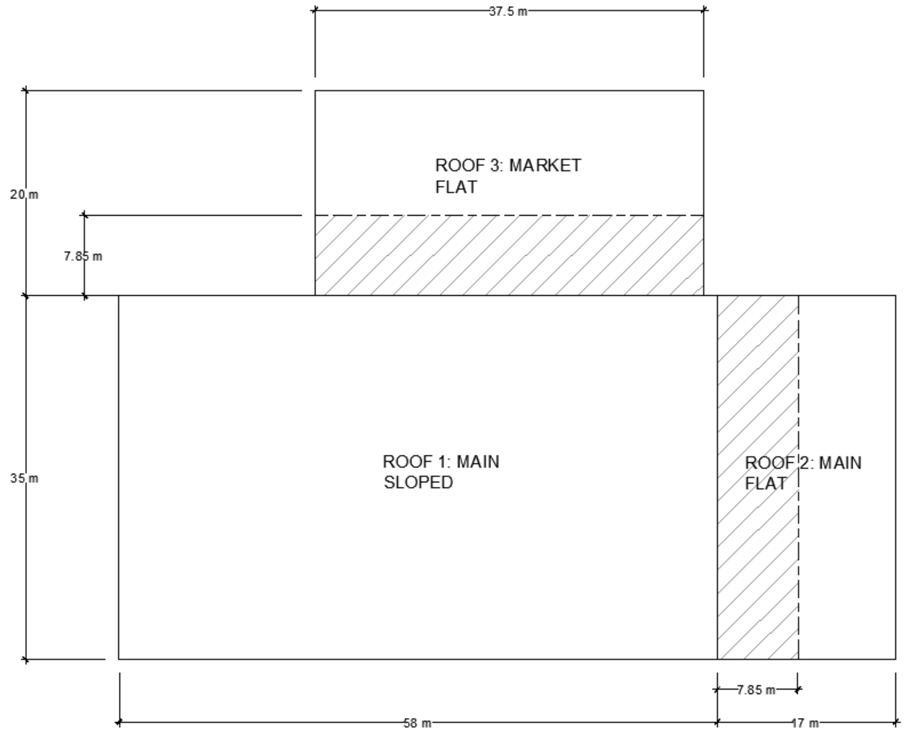

Figure 1: Roof labels along with drift load extents

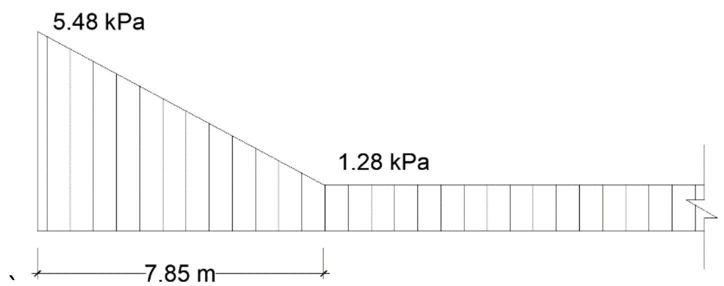

Figure 2: Snow Load Accumulation Distribution

As seen in Figure 2, at the boundary between the lower roof and higher elevation, the snow load is 5.48 kPa. Over 7.85 m this snow load decreases linearly to 1.28 kPa at which it remains. This accumulation occurs only on Roof 2 and 3 and a simple uniform distribution occurs over the entirety of the main roof. All unhatched portions of Figure 1 have the same uniform load of 1.28

kPa. It is expected that the increase in snow load at the hatched portions in Figure 1 will result in either greater member sizes or smaller spacing in roof joists.

6.2.4 Wind Load

The last gravity load which will be considered for this development is the case of wind uplift or downward wind pressure upon the roof. In many gabled roof cases, non-lateral wind load does play an important part for secondary members when considering uplift. The roof in consideration for Fruitland Vertical Farm and Marketplace is slightly slanted and thus the uplift in this situation may not be crucial. Regardless, uplift is calculated for the surface of each roof using Figure 4.1.7.6.-C in the NBCC. Figure 3 below shows the uplift and downward wind pressure that is exerted on each roof. Downward wind pressure rarely governs the gravity design and usually it is not a load that requires consideration for the design – this is also the case for this development.

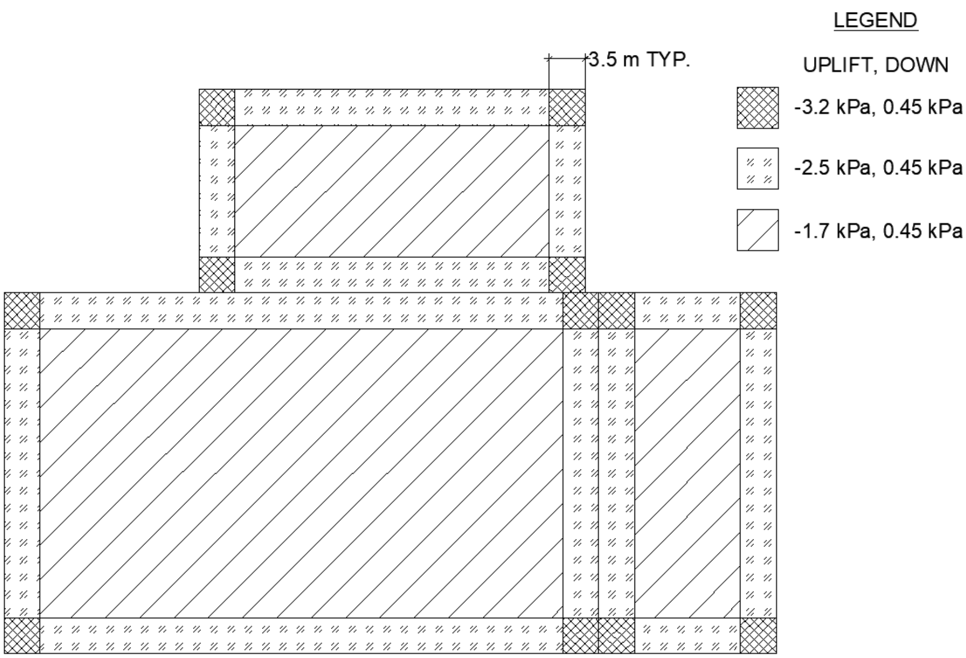

Figure 3: Roof Wind Pressure Distribution

6.3 LATERAL LOADS

6.3.1 Seismic Load

Seismic load is deemed to be rare and unexpected loading compared to other frequent sustained loads acting on a structure, such as dead load or live load (Xue, 2012). Seismic load only acts in a short period of time; however, it can cause huge damage due to unexpected ground motion behavior.

The primary objective of seismic provision stated in NBCC 2015 is to provide a sufficient design to match with limit state design philosophy. It is strictly defined in NBCC 2015 that the acceptable seismic hazard is at 20% probability of being exceeded in 50 years (Codes, 2015). According to section 4.1.8.7 in NBCC 2015, the analysis for seismic design can be carried out by Dynamic Analysis Procedure or Equivalent Static Force Procedure.

In NBCC 2015, the equivalent static force procedure is applied for structures that meet any of the following criteria:

a. In case where $I_E F_a S_a(0.2)$ is less than 0.5
b. Regular structures that are less than 60 m in height and have a fundamental lateral period, T_a, less than 2 seconds in each of two orthogonal directions
c. Structures with structural irregularity of Type 1, 2, 3, 4, 5, 6 or 8, that less than 20 m height and have a fundamental lateral period, Ta, less than 0.5 seconds in each of two orthogonal directions

Foundation

The foundation systems were not analyzed in this study. However, according to previous studies, Fruitland vertical farm is founded on stiff soil, thus, site classification is "C" (Ontario Association of Architects, 2016).

Design Response Spectrum

A response spectra is obtained by calculating the response of many single-degree-of-freedom (SDOF) systems to a specified excitation with various damping ratio (Xue, 2012). Response spectrum is the plot of the peak responses with different period.

Design response spectrum combined the spectra of several earthquakes occurred in the same region, thus, it represents the characteristics of ground motion in that area. According to NBCC 2015, for a specific vibration mode and damping ratio, the base shear of a seismic force-resisting system is proportional to its spectral acceleration at the corresponding natural vibration period of the structure.

Figure 4 shows the design response spectrums for the building.

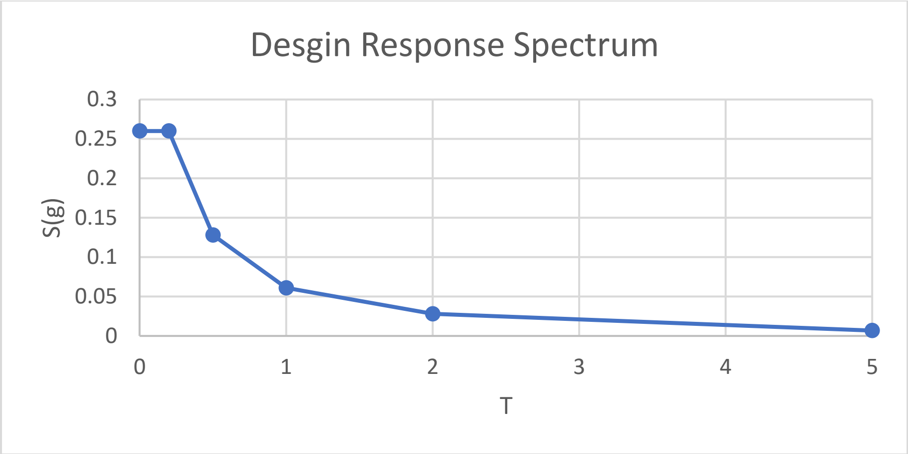

Figure 4: Design Response Spectrum

Ductility-Related Force Modification Factor (R_d)

In NBCC 2015, ductility-related force modification factor (R_d) is accounted for to determine the required full-yield strength of seismic force-resisting system (Xue, 2012). It reflects the capability of a structure to dissipate the input energy causing by an earthquake through its inelastic behavior. Therefore, for material which can perform inelastic deformation, R_d is usually equal to 1.0 or higher. The greater R_d value means the higher ductility of the structure.

For moderately ductile concentrically braced frame, R_d is taken as 3.0

Overstrength-Related Force Modification Factor (R_o)

Overstrength-related force modification factor (R_o) accounts for the dependable overstrength portion in a structure designed according to the provision. According to Xue (2012), additional overstrength of the structure is introduced by choosing larger section than needed, which usually happens in practical design process. Therefore, in order to have a more accurate estimate, R_o is accounted for.

For moderately ductile concentrically braced frame, R_o is taken as 1.3

Table 4.1.8.9.
SFRS Ductility-Related Force Modification Factors, R_d, Overstrength-Related Force Modification Factors, R_o, and General Restrictions(1)
Forming Part of Sentences 4.1.8.9.(1) and (5)

Type of SFRS	R_d	R_o	Restrictions(2)				
			Cases Where $I_EF_aS_a(0.2)$				Cases Where $I_EF_vS_a(1.0)$
			< 0.2	≥ 0.2 to < 0.35	≥ 0.35 to ≤ 0.75	> 0.75	> 0.3
Steel Structures Designed and Detailed According to CSA S16(3)(4)							
Ductile moment-resisting frames	5.0	1.5	NL	NL	NL	NL	NL
Moderately ductile moment-resisting frames	3.5	1.5	NL	NL	NL	NL	NL
Limited ductility moment-resisting frames	2.0	1.3	NL	NL	60	30	30
Moderately ductile concentrically braced frames							
Tension-compression braces	3.0	1.3	NL	NL	40	40	40
Tension only braces	3.0	1.3	NL	NL	20	20	20
Limited ductility concentrically braced frames							
Tension-compression braces	2.0	1.3	NL	NL	60	60	60
Tension only braces	2.0	1.3	NL	NL	40	40	40
Ductile buckling-restrained braced frames	4.0	1.2	NL	NL	40	40	40

Figure 5: Value for Rd and Ro

Table 4 shows the result of seismic load acting on each level and the base shear value

Floor	Lateral Force (kN)
2nd	281.1
3rd	432.2
4th	642.8
Roof	332.6
Base Shear	1688.6

Table 4: Seismic Load Result

Refer to Appendix A for detailed calculation.

6.3.2 Wind Load

When considering which load case governs for the lateral design of the building, a comparison between the seismic and wind loads need to be conducted to see which case governs. Fruitland Vertical Farm and Market Place is located within the Hamilton region and thus can typically be governed by wind or seismic loads. Whether or not the wind loads, or seismic loads govern depend on the size and shape of the building and also on the dead load. For the size of the building which is under 20 meters and is still considered a low rise building which still could be laterally governed by wind or seismic. The wide base of the building does indicate that a wind load may govern. Regardless, a seismic and wind load analysis was conducted to ensure that the appropriate and most stringent load case is used to design the lateral system. The calculations for uplift and also the lateral wind loads can be found in Appendix A.

Due to the shape of the building different than the regular rectangular footprint, which is outlined in Figure 4.1.7.6.-A in the NBCC 2015, conservative assumptions were made to analyze the wind load pattern on the building. The conservative assumption that was used in Fruitland's case was to imagine a rectangular prism surrounding the entirety of the building which would take on more wind load as it covered a greater area. For a visual explanation of this assumption one can visit Appendix A. Lateral load distribution can also be seen in Figure 6 and Figure 7 which extends from the base to the full height.

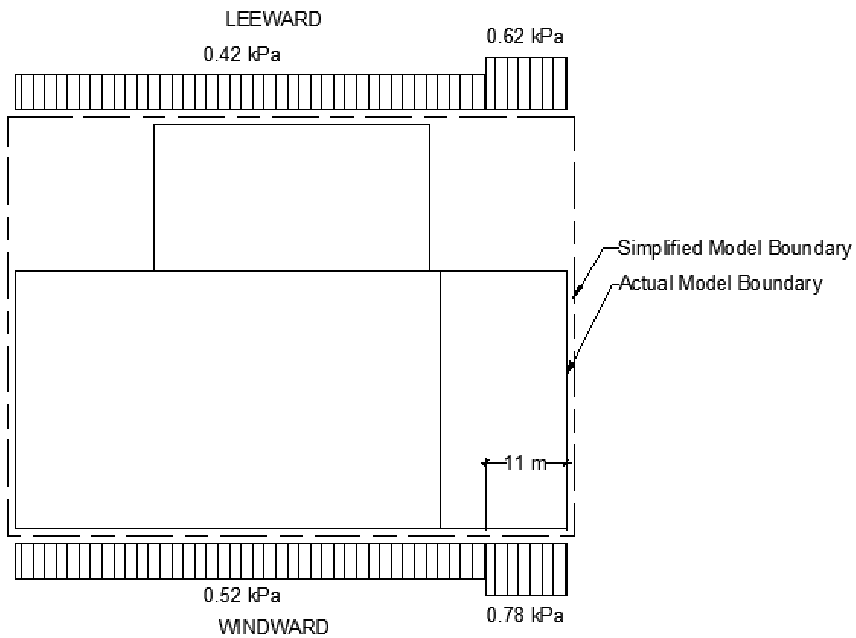

Figure 6: South to North Wind Load Distribution

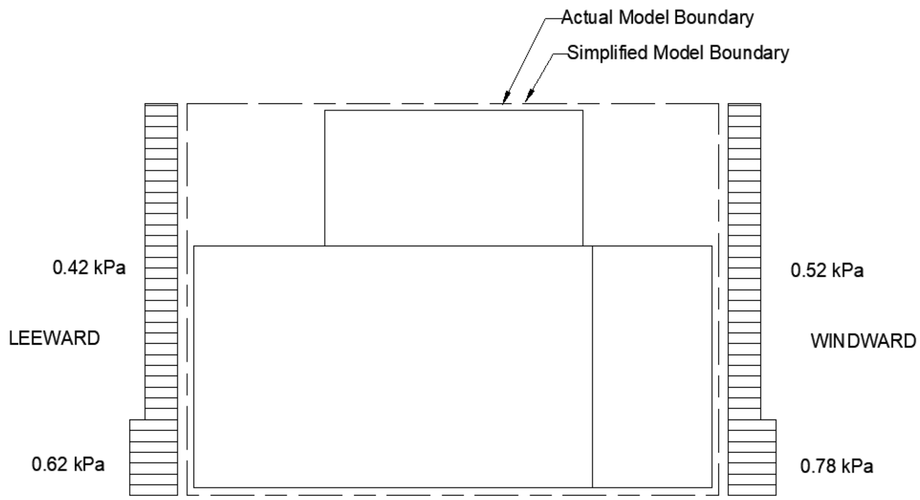

Figure 7: East to West Wind Load Distribution

Note, the values in wind distributions in Figure 6 and 7 represent loads from wind going from the windward to leeward side and the values are additive.

The total load applied on each level can be seen in Table 5 below for both the South to North distribution as well as the East to West.

	South to North Distribution		East to West Distribution	
Diaphragm/Floor	**Load (kN)**	**Factored Load (kN)**	**Load (kN)**	**Factored Load (kN)**
Roof	169	237	227	317
4	282	395	378	529
3	226	316	302	423
2	226	316	302	423

Table 5: Total Load Distribution Per Level

When comparing the values in this table to the factored loads calculated per level in the seismic chapter it is evident that the wind loads govern the lateral design over the seismic loads.

6.3.3 Notional Load

According to CSA S16-14, the additional translation load effects produced by notional load, equals to 0.5% of total factored gravity loads, to be added to the lateral loads for each load combination. Notional load must be calculated separately for each storey and shall be applied in both orthogonal directions independently when analyze three-dimensional loading effect.

Table 6 shows the notional loads due to dead, live and snow load at each level.

Level	Notional Dead Load (kN)	Notional Live Load (kN)	Notional Snow Load (kN)
Roof	16.46	13.13	16.80
4th	58.57	63.00	-
3rd	56.81	67.69	9.09
2nd	77.24	85.50	-

Table 6: Summary of Notional Loads

REFERENCES

Codes, C. C. on B. and F. (2015). *National Building Code of Canada 2015*. National Research Council Canada.

Ontario Association of Architects. (2016). *Seismic Hazard Index Calculation Examples*. Retrieved from https://oaa.on.ca/oaamedia/documents/Attach2 to PT.35 Seismic Calculations (July 13, 2016).pdf

Xue, Y. (2012). *Capacity design optimization of steel building frameworks using nonlinear time-history analysis*.

APPENDIX A

McMaster University ENGINEERING	Name: Appendix A	ID #: Group 12	
	Title: Dead Load and Live Load	Date:	Page: 1

Dead Load and Live Load

Dead Load

Roof:

.Fire protection = 0.07 KPa . Glass roof = 0.3 KPa S16-14, p7-69

.Structural framing = 0.5 KPa

.Duct, pipe, wiring (including lighting system) = 0.25KPa.

∴ Total Dead Load Roof = 1.12 KPa.

4^{th} Floor

. Partition = 1 KPa S16-14, p7-69

. Floor slab = 1.84 KPa

. Fire protection = 0.07 KPa.

. Structural framing = 0.5 KPa.

. Duct, pipe, wiring (including lighting system) = 0.2 KPa

. Cladding:

$h_4 = 6m$ thickness = 0.05 m.

$h_3 = 4m$ density of glass = 1615 kg/m^3

$g = 9.81\ m/s^2$

$$\text{cladding weight} = \frac{h_4+h_3}{2} \times \text{thickness} \times \text{density} \times g.$$

$$= \frac{6m+4m}{2} \times 0.05\ m \times 1615\ kg/m^3 \times 9.81\ m/s^2$$

$$= 3961 (N/m)$$

Cladding weight = 4(kN/m)

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	Title: Dead Load and Live Load	Date:	Page: 2

. Weight of rack:

Total weight of water $= 3.5\ m^3 \times 10\ KN/m^3 = 35\ kN$

Width of 1 rack $= 1.525\ m$.

length of 1 rack $= 5.6\ m$

Total number of rack $= 84$.

$\Rightarrow$ Weight of water on floor $= \dfrac{35\ kN}{84 \times 1.525\ m \times 5.6\ m} = 0.04879\ KPa$.

Average vegie weight $= 9.81\ N$

vegie /rack $= 22$

Total weight of vegie /rack $= 9.81 \times 22 = 215.82(N) = 0.22\ kN$

$\Rightarrow$ Weight of vegie on floor $= \dfrac{0.22\ kN}{1.525\ m \times 5.6\ m} = 0.025\ KPa$.

Weight of shelf $= 100\ kg \times 9.81\ m/s^2 = 981\ N = 0.981\ kN$

$\Rightarrow$ Weight of shelf on floor $= \dfrac{0.981\ kN}{1.525\ m \times 5.6\ m} = 0.11487\ kPa$

Total weight of rack $= 0.04879 + 0.025 + 0.114875$

$= 0.1889\ kPa$.

$\Rightarrow$ Total weight of rack $= 0.2\ kPa$

3rd floor:

. Partition $= 1\ KPa$ S16-14, p7-69.

. Floor slab $= 1.84\ kPa$

. Fire protection $= 0.07\ KPa$.

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	Title: Dead Load and Live Load	Date:	Page: 3

. Weight of rack = 0.2 kPa.

. structural framing = 0.5 KPa

. Duct, pipe, wiring = 0.2 kPa

. Cladding = $\frac{4m + 4m}{2} \times 0.05m \times 1615\ kg/m^3 \times 9.81\ m/s^2$

$= 3168\ (N/m)$

$\Rightarrow$ Cladding = 3.2 (KN/m)

2nd floor:

. Partition = 1KPa — S16-14, pg 7-69

. Floor slab = 1.84 KPa

. Fire protection = 0.07 kPa

. Weight of rack = 0.2 kPa

. structural framing = 0.5 kPa

. Duct, pipe, wiring = 0.2 kPa

. Cladding = 3.2 (kN/m)

. Suspended ceiling = 0.2 kPa

Live Load:

Roof:

. Live Load roof = 1 KPa — NBCC Table 4.1.5.3

4th, 3rd floor:

. Office Area = 2.4 kPa — NBCC Table 4.1.5.3

. Service Room = 3.6 kPa

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	Title: Dead Load and Live Load	Date:	Page: 4

- Pumping room = 3.6 KPa.
- Washroom = 2.4 KPa
- Mechanical equipment = 3.6 KPa
- Retail and whole sale = 4.8 kPa (Assume for racks)

⇒ Max Live Load = 4.8 KPa.

2nd floor:

- Office Area: 2.4 KPa — NBCC Table 4.1.5.3
- Service room = 3.6 KPa.
- Pumping room = 3.6 kPa
- Washroom = 2.4 KPa
- Mechanical equipment = 3.6 KPa
- Retail and whole sale = 4.8 KPa (Assume for racks)
- Restaurant = 4.8 KPa
- Kitchen = 4.8 kPa

⇒ Max Live Load = 4.8 kPa

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	Title: Snow Load	Date:	Page: 1

SNOW LOAD

$S = I_s[S_s(C_b C_w C_s C_a) + S_r]$ §4.1.6.2

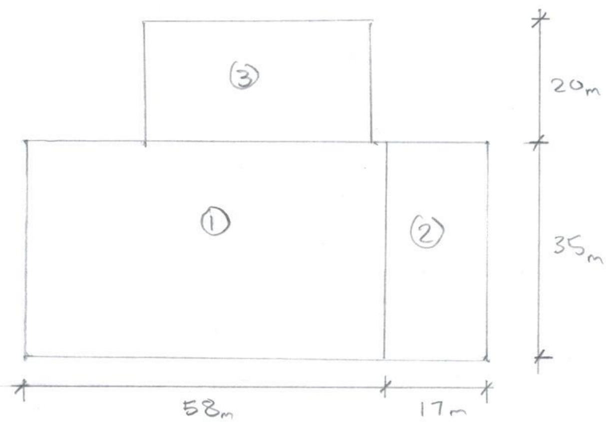
$I_s(ULS) = 1$ (Normal)
$I_s(SLS) = 0.9$ (Serviceability) TABLE 4.1.6.2A

$S_s = 1.1$ kPa (1 in 50 yr ground snow load) Table C-2
$S_r = 0.4$ kPa

$C_w = 1$ (Urban) §4.1.6.2(3)
$C_s = 1$ (slope $<30°$ for all roofs) §4.1.6.2(5)

C_a and C_b will depend on which roof and case we look at.

- Overall there are 3 roofs to look at



- Roof 1
  $w = 35m$ $l = 58m$
  $l_c = 2w - w^2/l = 2 \cdot 35 - 35^2/58 = 48.9m$ §4.1.6.2(2)
  $70/C_w^2 = 70/1^2 = 70$
  $l_c < \frac{70}{C_w^2}$ so $C_b = 0.8$

  $C_a = 1$ (uniform snow load case) §4.1.6.2(5)

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· Roof 1 cont'd

$S = I_s[S_s(C_b C_w C_s C_a) + S_r]$ — 4.1.6.2

$S_{ULS} = (1)[1.1(0.8 \times 1 \times 1 \times 1) + 0.4]$
$= 1.28\,kPa$

$S_{SLS} = 0.9 \times 1.28$
$= 1.152\,kPa$

· Roof 2 (Main building - lower roof)

$l = 35m$, $w = 17m$

$l_c = 2w - w^2/l = 2 \times 17 - 17^2/35 = 25.7$ — 4.1.6.2(2)

$\frac{70}{C_w^2} = \frac{70}{1} = 70$

$l_c < 70/C_w^2$ so $C_b = 0.8$

↳ Drift

CASE 1 (Upper roof snow comes onto lower roof) — Fig 4.1.6.5B

$\beta = 1$ $\quad h_p = 0$

$w_s = 35m$ $\quad l_s = 58m$ (upper roof dimensions)

$l_{cs} = 2w_s - w_s^2/l_s = 48.9$

$\gamma = MIN(0.43 \times S_s + 2.2, 4) = 2.673$ — 4.1.6.13

$h = 2m$ (height difference) — 4.1.6.5

$C_{a0_1} = \beta \gamma h / C_b S_s = 1 \times 2.673 \times 2 / 0.8 \times 1.1 = 6.075$

$F = 0.35\beta\sqrt{\frac{\gamma(l_{cs} - 5h_p')}{S_s}} + C_b$

$= 0.35 \times (1)\sqrt{\frac{2.673(48.9 - 0)}{1.1}} + 0.8$

$= 4.62$

$C_{a0_2} = F/C_b = 4.62/0.8 = 5.77$

$C_{a0} = Min(C_{a0_1}, C_{a0_2}) = 5.77$

CASE 2 (Lower roof is snow area source for drift)

$\beta = 0.67$

$w_s = 17m$ $\quad l_s = 35m$

$l_{cs} = 2 \times 17 - 17^2/35 = 25.74m$

$C_{a0_1} = \beta \gamma h / C_b S_s = 0.67 \times 2.673 \times 2 / 0.8 \times 1.1 = 4.07$

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·Roof 2 cont'd

$F = 0.35 \times 0.67 \times \sqrt{\frac{2.673(25.74-0)}{1.1}} + 0.8$ 4.1.6.5

$= 2.65$

$C_aO_2 = F/C_b = 2.65/0.8 = 3.32$

$C_aO = MIN(C_aO_1, C_aO_2) = 3.32$

Greatest C_aO of both cases is 5.77 ← from CASE 1

$C_aO = 5.77$

$X_d = 5\frac{C_b S_s}{\gamma}(C_aO - 1) = 5 \times \frac{0.8 \times 1.1}{2.673}(5.77-1) = 7.85m$

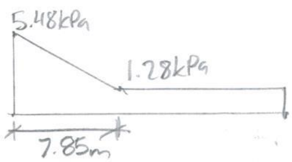
$C_w = 1$ constantly as this is classified as non-exposed Fig 4.1.6.5A

$S_{ULS} = I_s[S_s(C_b C_w C_s C_a) + S_r]$

$= 1[1.1(0.8 \times 1 \times 1 \times 5.77) + 0.4]$

$= 5.48 kPa$

→ $S_{ULS} = 1.28 kPa$ after 7.85m away from high roof.



*NOTE: CASE 3 doesn't apply

*NOTE: multiply loads by 0.9 to get S_{SLS} values

·Roof 3 (Market)

↳ $l = 37.5m$ $w = 20m$

$l_c = 2 \times w - w^2/l = 2 \times 20 - 20^2/37.5$

$= 29.33$

$70/C_w^2 = 70$

$l_c < 70/C_w^2$ so $C_b = 0.8$

↳ DRIFT

CASE 1

$h = 10m$

$C_aO_1 = \beta\gamma h / C_b S_s = 30.4$

$C_aO_2 = F/C_b = 5.77$ (same as previous roof parameters)

$C_aO = MIN(C_aO_1, C_aO_2)$

$= 5.77$

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· Roof 3 (cont'd)

CASE 2 — 4.1.6.5

$w_s = 20 \quad l_s = 37.5\,m$

$l_{cs} = 2 \times 20 - 20^2/37.5 = 29.33$

$\beta = 0.67$

$C_{a0_1} = \beta \gamma h / (C_b S_s) = \frac{0.67 \times 2.673 \times 10}{0.8 \times 1.1} = 20.4$

$F = 0.35 \times 0.67 \times \sqrt{\frac{2.673(29.33)}{1.1}} + 0.8 = 2.78$

$C_{a0_2} = F/C_b = 2.78/0.8 = 3.5$

$C_{a0} = MIN(C_{a0_1}, C_{a0_2}) = 3.5$

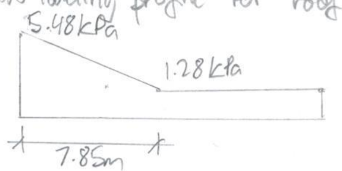
Now looking at the governing C_{a0} between Case 1 and Case 2, Case 1 is greater and is the same as the C_{a0} for Roof 2

$C_{a0} = 5.77$

↳ because C_{a0} is the same Snow load and x_d (thus distribution) are the same as well.

Snow loading profile for roof 3 is thus:

S_{ss}:



* NOTE: CASE 3 doesn't apply
* NOTE: multiply loads by 0.9 to get S_{scs} values

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WIND LOAD

$p = I_w q C_e C_t C_g C_p$ — 4.1.7.3

$I_{w\,ULS} = 1.0 \quad I_{w\,SLS} = 0.75$ Normal Importance Category — TABLE 4.1.7.3

$q = 0.46\ kPa$ Hamilton — App C Table C-2

$h = 18\ m$ (Conservative)

$C_e = (h/10)^{0.2}$ — 4.1.7.3(5)

$= (18/10)^{0.2}$

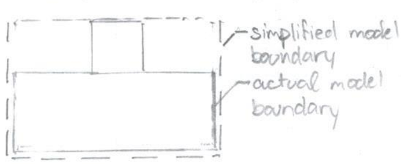
$= 1.13$

$C_t = 1$ (not on a hill) — 4.1.7.4

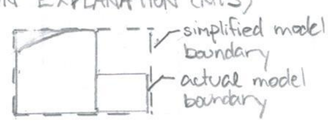
Now to calculate the CgCp and thus the rest of the wind load, a model similar to Fig 4.1.7.6.-A is required to facilitate the low-rise building calculation.

↳ As a result, the multilevel model was simplified as a large prism over the entire building.

PLAN EXPLANATION (NTS)



ELVN EXPLANATION (NTS)



CgCp for entire structure - 0° to 5° slope to be conservative — Fig 4.1.7.6-A

↳ Load Case A governs here → Case B primarily for gabled roofs.

Surface	CgCp	pULS (kPa)	pSLS (kPa)
1	1	0.52	0.39
1E	1.5	0.78	0.58
4	-0.8	-0.42	-0.31
4E	-1.2	-0.62	-0.47
2	-1.3	-0.67	-0.51
2E	-2	-1.04	-0.78
3	-0.9	-0.47	-0.35
3E	-1.3	-0.67	-0.51

$pSLS = pULS \times 0.75$

Ex calc for pULS → surface 1

$p = I_w q C_e C_t C_g C_p$

$p = 1 \times 0.46 \times 1.13 \times 1 \times 1$

$= 0.52\ kPa$

$pSLS = 0.75 \times 0.52$

$= 0.39\ kPa$

$z = \min(0.1w, 0.4h) = \min(0.1 \cdot 55, 0.4 \times 18) = \min(5.5, 7.2) = 5.5_m$ — Fig 4.1.7.6-A ↳ Notes (7)

$= 5.5_m > (0.04 \times 55, 1) = (2.2, 1)$

$y = \max(6, 2z) = \max(6, 11) = 11_m$ — ↳ Notes (6)

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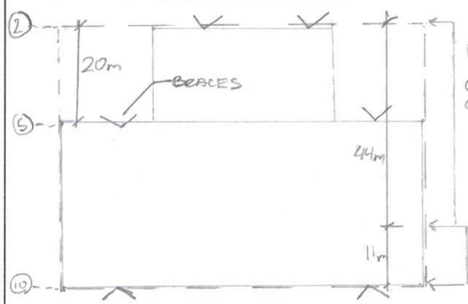


- Loads (kPa) distributed on cladding is seen in the figure to the left
- Load from 1I + 4I
  $= 44m \times (0.52+0.42) kPa \times 4m$
  $= 165 kN$
- Load from 1E + 4E
  $= 11m \times (0.78+0.62) kPa \times 4m$
  $= 61 kN$

Total load for 4m tributary width = 61 + 165 = 226 kN
Break down of total load for this wind scenario per floor

Diaphragm/Flr	Load (kN)	F_{LOAD} (kN) = 1.4×LOAD
Rf	169	237
4	282	395
3	226	316
2	226	316

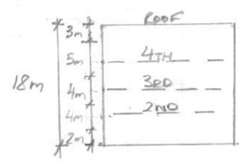
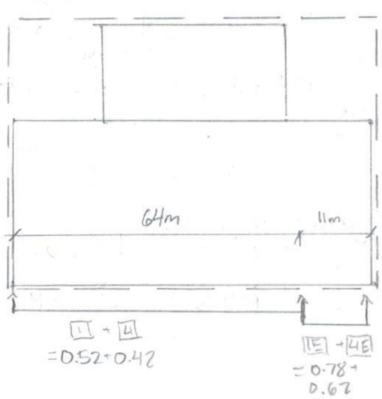


TABLE 4.1.3.2A

↳ Calculation for 3rd and 2nd floor is seen above
↳ Calculation for 4th floor $LOAD_4 = \frac{5}{4} \times 226 = 282$ kN
Calculation for Roof is $LOAD_{RF} = \frac{3}{4} \times 226 = 169$ kN

- When comparing above table with lateral loads obtained by seismic forces it is evident that wind loads govern lateral design.

LOOKING @ N-S LATERAL LOADS



- Load from 1I + 4I
  $= 64 \times (0.52+0.42) \times 4$
  $= 241$ kN
- Load from 1E + 4E
  $= 61$ kN
- Total load for 4m tributary height is 241 + 61 = 302 kN
  ↳ 302 > 226 kN > earthquake shear load on 3rd/2nd flr
  ↳ wind load governs lateral design
- Exact load on braces covered next chapter.

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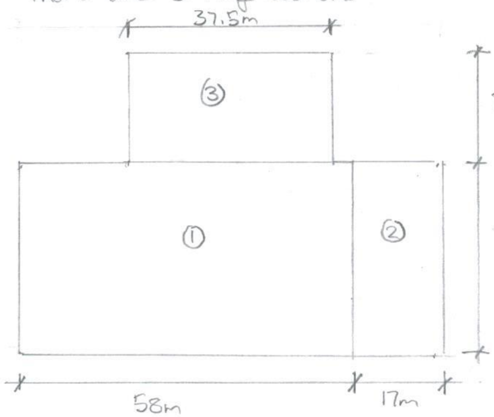
LOADS ON ROOF AND SUPPORTING STRUCTURAL MEMBERS

- There are 3 roof sections



- General Formula
  $p = I_w q C_e C_t [C_g C_p - C_{gi} C_{pi}]$ — 4.1.7.3(3)

  $C_{gi} = 2$ — 4.1.7.3(10)
  $C_{pi} = -0.15$ or 0 — Table 4.1.7.7
  ↳ small openings

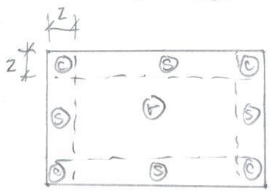
  $p = (1)(0.46)(1.13)(1)[C_g C_p - C_{gi} C_{pi}]$

  $P_{DOWN} = 0.52(C_g C_p + 0.15)$
  $P_{UP} = 0.52(C_g C_p + 0)$

  Use these eqns to find wind forces on these 3 roofs

- Finding external peak values for roof ①
  ↳ smallest tributary area is a perimeter joist on grid A between grid lines ⑦ & ⑧
  $TA = 6.2 \times \frac{7}{4} \times \frac{1}{2} = 5.4 m^2 \approx 5 m^2$

Using Fig 4.1.7.6-C — Fig 4.1.7.6-C ↳ Note (3)
$z = \min(0.1 \times 35, 0.4 \times 18) = 3.5m > 0.1w$ or $1m$



	UPLIFT	DOWN
$C_p C_g$ s =	-2.5	$+0.45$
$C_p C_g$ r =	-1.7	$+0.45$
$C_p C_g$ c =	-3.2	$+0.45$

Now using $P_{DOWN} = 0.52(C_g C_p + 0.15)$
$P_{UP} = 0.52(C_g C_p)$

UP	DOWN
$P_s = -1.3 kPa$	$P_s = 0.32 kPa$
$P_r = -0.9 kPa$	$P_r = 0.32 kPa$
$P_c = -1.7 kPa$	$P_c = 0.32 kPa$

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- Finding External peak values of C_gC_p for Roof 2
  - ↳ smallest TA is perimeter joist on grid K between grid 7 & 8

$$TA = 6.2 \times \frac{8.5}{4} \times \frac{1}{2} = 6.6 m^2$$

↳ These values are very close to what was observed for roof 1 so keep those values — Fig 4.1.7.6.C

↳ NOTE: $z = \min(0.1 \times 17, 0.4 \times 18) = 1.7m$ — Note (3)

UP	DOWN
p_s = -1.3 kPa	p_s = 0.72 kPa
p_r = -0.9 kPa	p_r = 0.72 kPa
p_c = -1.7 kPa	p_c = 0.72 kPa

- Finding external peak values of C_gC_p for Roof 3
  - ↳ smallest TA is perimeter joist on grid C1 between grid 4 & 5

$$TA = 6 \times \frac{7}{4} \times \frac{1}{2} = 5.25 m^2$$

↳ This TA is close to $5m^2$ as well so same values to be used

↳ NOTE: $z = \min(0.1 \times 20, 0.4 \times 18) = 2 m$

UP	DOWN
p_s = -1.3 kPa	p_s = 0.72 kPa
p_r = -0.9 kPa	p_r = 0.72 kPa
p_c = -1.7 kPa	p_c = 0.72 kPa

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Earthquake Load

- Classification of Building — NBCC 4.1.7.2.2

a) Assume natural frequency is less than 1.0 Hz and greater than 0.25 Hz.

b) Height of building = 18m < 60m

c) Effective width:

$$w' = \frac{\sum h_i w_i}{\sum h_i} = \frac{4m \times 60m + 6m \times 60m}{18m} = 33.3\,(m).$$

$h < 4w'$

⇒ Building is not dynamic sensitive.

- Use Equivalent Lateral Force Approach.

Level	DL (KN)	SL (KN)	Water tank (KN)
Roof	3292	3360	0
4th	11714	0	265
3rd	11362	1818	265
2nd	15447	0	265

$W_i = DL_i + 0.25SL_i + 0.6\text{ Storage} + \text{Tank}$

$W_R = 3292 + 0.25(3360) = 4132\ (KN)$

$W_4 = 11978\ (KN)$

$W_3 = 12081\ (KN)$

$W_2 = 15713\ (KN)$

$\Rightarrow \sum W_i = W_R + W_4 + W_3 + W_2 = \underline{\underline{43904}}\ (KN)$

(Total seismic weight)

$$V = \frac{S(T_a) M_v I_E M_v}{R_d R_o} W$$ NBCC - 4.1.8.11

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$I_e = \underline{\underline{1.0}}$ — NBCC 4.1.8.5

Find $S(T_a)$:

- According to NBCC Table C3

Location: Hamilton

$S_a(0.2)$	$S_a(0.5)$	$S_a(1.0)$	$S_a(2.0)$	$S_a(5.0)$	$S_a(10)$	PGA	PGV
0.26	0.128	0.061	0.028	0.0068	0.0027	0.168	0.101

$$\frac{S_a(0.2)}{PGA} = \frac{0.26}{0.168} = 1.548 < 2.0$$ — NBCC 4.1.8.4.4(a)

$\Rightarrow PGA_{ref} = 0.8(PGA) = 0.8(0.168) = 0.1344$

$\begin{cases} 0.1 < PGA_{ref} < 0.2 \rightarrow \text{take average} \\ \text{Site Class C} \end{cases}$

$F(0.2) = 1.0$ $\quad F(0.5) = 1.0$ $\quad F(1.0) = 1.0$ — NBCC Table 4.1.8.4

$F(2.0) = 1.0$ $\quad F(5.0) = 1.0$ $\quad F(10) = 1.0$ — $B \rightarrow G$

$\rightarrow S(0.2) = \max[F(0.2)S_a(0.2), F(0.5)S_a(0.5)]$

$S(0.2) = \max[1.0 \times 0.26, 1.0 \times 0.128]$

$S(0.2) = 0.26$

$\rightarrow S(0.5) = F(0.5)S_a(0.5) = 0.128$

$\rightarrow S(1.0) = F(1.0)S_a(1.0) = 0.061$

$\rightarrow S(2.0) = F(2.0)S_a(2.0) = 0.028$

$\rightarrow S(5.0) = F(5.0)S_a(5.0) = 0.0068$

$\rightarrow S(10) = F(10)S_a(10) = 0.0027$

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- See attached spreadsheet for response spectrum.

$T_a = 0.025(18m) = 0.45(s) \rightarrow$ Steel braced frame — NBCC 4.8.11.3 (a)(i)

$\Rightarrow S(T_a) = S(0.45) = \underline{\underline{0.15}}$

- Get M_v — NBCC-4.1.8.11.6, Table 4.1.8.11

$T_a = 0.35 < 0.5$

$\frac{S(0.2)}{S(5.0)} = \frac{0.26}{0.0068} = 38.24$

Steel braced frame

$\Rightarrow M_v = \underline{\underline{1.0}}$

- Get R_d, R_o — NBCC 4.1.8.9

$\rightarrow$ Moderately ductility concentrically brace frame

$\Rightarrow R_d = \underline{\underline{3.0}}$ $\quad R_o = \underline{\underline{1.3}}$

$$V = \frac{S(T_a) M_v I_E}{R_d R_o} W$$

$$\Rightarrow V_{br} = \frac{0.15(1.0)(1.0)}{(3.0)(1.3)}(43904)$$

$\Rightarrow V_{br} = 1689 \,(kN)$

- Distribute V_{br} on floors

$$F_x = (V - F_t)\frac{W_x h_x}{\sum_{i=1}^{n} W_i h_i}$$ NBCC 4.1.8.11.7

$T_a = 0.15 < 0.7 \Rightarrow F_t = 0$

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Level	W_x (kN)	h_x (m)	$W_x h_x$ (kNm)	$W_x h_x / \Sigma W_i h_i$	F_x (kN)
2^{nd}	15713	4.0	62850.5	0.166	281.05
3^{rd}	12081	8.0	96647.3	0.256	432.19
4^{th}	11978	12.0	143740.4	0.381	642.78
Roof	4132	18.0	74376	0.197	332.59
		Σ	377614.26	1.0 ✓	1688.61 ✓