Chapter #9 FRAMING DESIGN

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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9.1 DESIGN APPROACH

Greentech Engineering recognizes the importance of enhancing the triple bottom line of our stakeholders being people, planet, profit and these concepts are reflected in our design approach. People are at the core of Greentech Engineering as we focus on safety, ease of construction and familiar structural design to decrease the chances of errors. All member loads are calculated conservatively to ensure the safety and proper serviceability of the occupants and workers. Sustainability is at the core of Greentech Engineering services and it is also at the core of the project, Fruitland Vertical Farm and Marketplace and this too was an important factor in the overall design. Although it is important to be conservative and have built-in redundancy into our structural system, our engineers also recognize that the more material used, the more embodied carbon will be part of the project. It is our aim to design the structure efficiently – for instance our decision to share the same columns between the market and farm reflect our aim to decrease the material used in the structural system. With an aim to decrease material naturally follows savings which increase profit and increasing the overall value of the project for developers. All steel sections used for structural framing members are ASTM Grade A992.

9.2 JOIST DESIGN

9.2.1 Joist Design Overview

Joist is known as secondary structural member, which supports composite slab or steel deck. It carries vertical loads, such as dead load, live load, snow load, etc. and transfers them to girder. As a result, joist is subjected to shear and bending moment. It is necessary to perform adequate checks to ensure the member can resist critical load combination listed in NBCC 2015. A sustainable design requires the balance between engineering knowledge, total project cost, and construction efficiency. Thus, the building is divided into five sub-areas and design joist according to critical loading case at each area.

9.2.2 Joist Design for Floor (Market Place)

According to Chapter 6, market place subjects a uniform 4.06 kPa dead load and 4.8 kPa live load. The critical factored load combination is identified to be case 2 in NBCC Table 4.1.3.2-A

$$1.25DL + 1.5LL \quad (1)$$

Since the floor slab is sitting on joist members, it prevents lateral movement of the top flange along the joist. In addition, the loads are acting in gravity direction. Therefore, it is assumed that joists are laterally supported along compression flange, which also means there is no lateral torsional buckling. Critical loading case is chosen with largest tributary area, which lies between grid D1 and E1 and grid 3 and 4. Then, maximum factored bending moment and shear are calculated based on factored loading combination. Next, maximum bending moment is substitute into equation 2 to obtain initial z_x value for first design iteration section.

$$\phi z_x F_y \geq M_f \tag{2}$$

Last, adequate checks are performed for moment, shear and deflection according to CSA S16-14.

For detailed calculation, refer to Appendix B

9.2.3 Joist Design for Floor (Main Building)

Similar to market place, main building subjects a uniform 4.06 kPa dead load and 4.8 kPa live load. The critical factored load combination is also identified to be case 2 in NBCC Table 4.1.3.2-A. Joist member in main building are assumed laterally supported along compression flange. Critical loading case is chosen with largest tributary area, which lies between grid I and J and grid 8 and 9. Maximum factored bending moment and shear are calculated based on factored loading combination. Initial design section is chosen based on according bending moment value. Adequate checks are required for moment, shear and deflection according to CSA S16-14.

For detailed calculation, also refer to Appendix B

9.2.4 Joist Design for Roof 1 (Curved Roof of Main Building)

Critical load case for roof in gravity direction is identified to be case 3 in NBCC 2015

$$1.25DL + 1.5SL + 1.0LL \tag{3}$$

However, when designing joist for roof level, it is necessary to consider uplift caused by wind suction. The critical uplift load combination is identified to be case 4 in NBCC 2015

$$0.9DL + 1.4WL \tag{4}$$

Largest tributary area for a joist on roof 1 lies between grid C and D and grid 8 and 9. Maximum bending moment and shear are calculated in two cases, namely gravity direction load case and uplift load case. It is noticed that joist members are only laterally braced on top flange, thus, in uplift case, it is possible for lateral torsional buckling occurs. Hence, LTB check is required along with moment, shear and deflection checks as per CSA S16-14. The roof is assumed to be flatted due to small increase in angle (about 3.3^o).

For detailed calculation, refer to Appendix B

9.2.5 Joist Design for Roof 2 (Flat Roof of Main Building)

The procedure of joist design is similar to roof 1, except the value of snow load. Since roof 2 is next to and lower than roof 1. Snow drift between two roofs occurs. As a result, snow load on a portion of roof 2 is significant larger. In order to account for snow drift in a conservative approach, joist members are design for maximum snow load of 5.48 kPa.

For detailed calculation, refer to Appendix B

9.2.6 Joist Design for Roof 3 (Roof of Market Place)

Similar to roof 2, snow drift happens as a result of slope roof portion located in the middle of roof 3. To be more conservative in calculation, snow load is also taken as 5.48 kPa for roof 3. All other design procedures are same as roof 1.

For detailed calculation, refer to Appendix B

Table 1 and 2 show the factored and resistant moment, shear, deflection of chosen section for joist design.

Level	Critical Case			Moment			Shear		
	Load type	Span (mm)	Tributary Area (m2)	M_f (kNm)	M_r (kNm)	M_f/M_r	V_f (kN)	V_r (kN)	V_f/V_r
Market Floor	Gravity	7000	13.1	141	249	0.57	81	542	0.18
Main Building Floor	Gravity	7250	15.4	172	219	0.79	95	423	0.22
Roof 1	Gravity	7250	14.6	58	104	0.56	32	275	0.12
	Uplift	7250	13.5	17	38	0.45	10	275	0.04
Roof 2	Gravity	7250	15.4	149	205	0.73	82	470	0.17
	Uplift	7250	15.4	10	45	0.22	6	470	0.01
Roof 3	Gravity	7000	13.1	123	205	0.6	70	470	0.15
	Uplift	7000	13.1	16	49	0.33	9	470	0.02

Table 1: Moment and Shear Summary for Joist Design

Level	Critical Case			Deflection		Section
	Load type	Span (mm)	Tributary Area (m2)	Δ_{max} (mm)	Δ_{limit} (mm)	
Market Floor	Gravity	7000	13.1	18.3	19.4	W200x71
Main Building Floor	Gravity	7250	15.4	18.5	20.1	W310x45
Roof 1	Gravity	7250	14.6	13.3	20.1	W200x31
	Uplift	7250	13.5	-	-	
Roof 2	Gravity	7250	15.4	18.4	20.1	W360x39
	Uplift	7250	15.4	-	-	
Roof 3	Gravity	7000	13.1	14	19.4	W360x39
	Uplift	7000	13.1	-	-	

Table 2: Deflection and Section Summary for Joist Design

9.3 GIRDER DESIGN

9.3.1 Girder Design Process

Girder is an important primary structural member, which carries loads from joists and directly transfers them to columns. Therefore, under-designing girder would cause significant risk to the structure. It is necessary to perform all adequate checks stated in CSA S16-14 with respect to critical loading cases listed in NBCC 2015. Sustainable design is also applied in designing girder.

9.3.2 Girder Design for Floor (Market Place)

Since the girder carries loads from joist connected from both sides, it is assumed girder is laterally supported at joist locations. In this case, lateral torsional buckling may occur within unbraced length. Thus, critical loading case is chosen by comparing tributary area and load combinations. Then, factored moment and shear are calculated in order to compare with resistant capacity of the girder section. Adequate checks are required for moment, shear and deflection according to CSA S16-14.

For detailed calculation, refer to Appendix B

9.3.3 Girder Design for Floor (Main Building)

Similar to market place, in order to design girder main building, it needs to follow the logic of choosing critical loading case and complete necessary checks to ensure safety of the structure.

For detailed calculation, refer to Appendix B

9.3.4 Girder Design for Roof 1 (Curved Roof of Main Building)

For roof girder design, maximum wind load (happens within zone 2E) is taken to calculate the critical loading case. Next, initial member section is selected based on factored moment and shear. Last, perform all checks to ensure section is safe. Again, the roof is assumed to be flatted due to small increase in angle (about 3.3^o). For safety concern, the largest uplift pressure is used in calculation.

For detailed calculation, refer to Appendix B.

9.3.5 Girder Design for Roof 2 (Flat Roof of Main Building)

The procedure is similar to roof 1, however, the snow load in this case accounts for snow drift caused by the difference in elevation between two roofs. For safety concern, the largest uplift pressure and snow load are used in calculation.

For detailed calculation, refer to Appendix B.

9.3.6 Girder Design for Roof 3 (Roof of Market Place)

The procedure is similar to roof 1, however, the snow load in this case accounts for snow drift caused by the slope roof in the middle of the roof. For safety concern, the largest uplift pressure and snow load are used in calculation.

For detailed calculation, refer to Appendix B.

Table 3 and 4 show the factored and resistant moment, shear, deflection of chosen section for girder design.

Level	Critical Case			Moment			Shear		
	Load type	Span (mm)	Unbraced length (mm)	M_f (kNm)	M_r (kNm)	M_f/M_r	V_f (kN)	V_r (kN)	V_f/V_r
Market Floor	Gravity	7500	1875	608	624	0.97	243	996	0.24
Main Building Floor	Gravity	8500	2125	663	732	0.91	234	1114	0.21
Roof 1	Gravity	8050	2012.5	258	277	0.93	96	523	0.18
	Uplift	8050	8050	27	90	0.3	10	523	0.02
Roof 2	Gravity	8500	2125	697	732	0.95	246	1114	0.22
	Uplift	8500	8500	30	262	0.11	11	1114	0.01
Roof 3	Gravity	7500	1875	525	534	0.98	210	931	0.23
	Uplift	7500	7500	23	251	0.09	9	931	0.01

Table 3: Moment and Shear Summary for Girder Design

Level	Critical Case			Deflection		Section
	Load type	Span (mm)	Tributary Area (m2)	Δ_{max} (mm)	Δ_{limit} (mm)	
Market Floor	Gravity	7500	1875	8.57	20.8	W460x89
Main Building Floor	Gravity	8500	2125	20.4	23.6	W530x92
Roof 1	Gravity	8050	2012.5	15.4	22.4	W360x51
	Uplift	8050	8050	-	-	
Roof 2	Gravity	8500	2125	20.9	23.6	W530x92
	Uplift	8500	8500	-	-	
Roof 3	Gravity	7500	1875	20.5	20.8	W410x81
	Uplift	7500	7500	-	-	

Table 4: Deflection and Section Summary for Girder Design

9.4 COLUMN DESIGN

9.4.1 Column Design Process

Most columns carry a different load and subsequently, a bespoke design can be done per each column; however, having a different structural section for each column would not be conducive of achieving a good design that reflects value for our clients. The greatest difference between loads on columns are from the exterior columns and interior columns as the tributary width for the exterior columns are roughly half of that of an interior column. Exterior columns also carry the weight of the triple glazing curtain wall as well which increases its load significantly as well. With this in mind, Greentech Engineering has decided to use a different section for exterior and interior columns for each level and in each building. Having 4 storeys for the main farm building and 2 storeys for the marketplace, a total of 12 columns need to be designed. With less variance amongst the columns, more uniformity is introduced to the construction and thus less error as well. To design these 12 columns, it is important to first create a short list of columns that could govern the design and then design for the governing column.

During the design process and looking at the design of the building holistically, it was realized that certain columns needed to be upsized in order to accommodate for the connections. The columns didn't need to be overdesigned too much as they were situated as to have the smaller joists connect to the webs as opposed to the much larger girders. Small C_f/C_r values are due to the upsizing of columns to accommodate joist flanges fitting into the column's web.

9.4.2 Identified Possible Governing Columns

With various load considerations and column lengths, there are various columns on each level that could govern the design. Table 5 below provides a summary of columns that may govern the design in their respective category (level and exterior/interior) and the reason why they may govern.

LEVEL	INT/EXT	POSSIBLE GOVERNING COLUMN	REASONS IT MAY GOVERN
4	INT	J8	- Largest tributary area on level - KL=4m
		I8	- Possibly most heavily loaded due to accumulated snow load - KL=4m
		D6	- Large tributary area - KL=6m - overall lower loading than J8 or I8 though
	EXT	I4	KL=4m - No line load from curtain wall as it is supported below - supports drift snow load as well
		D4	- largest TA for column with KL=6m
3	INT	J8	- Big load from roof now translated to this column - Again biggest TA
		I8	- Smaller TA than J8 but large load from roof can still make this governing
	EXT	I4	- Large tributary area which is enhanced by the snow drift load

		J4	- Largest tributary area on exterior although not receiving more snow drift load than I4
2	INT	J8	- It was found that this column governed on the 3rd level and thus will govern again on this level
		G4	- Checking the load here due to it being a column shared between both buildings and there is a slight change in tributary area reduction factor as well due to the roof live load - Snow drift load has also accumulated and now contributes to the load here
		F2-3	- Interior market column which has the largest tributary area
	EXT	J4	- Largest tributary area on exterior for main building. It has governed on the 3rd level and thus will govern again here
		F2-1	- Largest tributary area on exterior for market
1	INT	J8	- It was found that this column governed on the 2nd level and thus will most likely govern again on this level
		G4	- The change in live load reduction factor due to it being an assembly occupancy may make this column govern
		F2-3	- Interior market column which has the largest tributary area
	EXT	J4	- Largest tributary area on exterior for main building. It has governed on the 2nd level and thus will govern again here
		F2-1	- Largest tributary area on exterior for market

Table 5: Columns That May Govern with Reasoning

9.4.3 Final Column Design

The loads on the columns that may govern were found and the resultant factored load was calculated as documented in Appendix B. The detailed calculations can be followed in Appendix B, but a summary of the results and the chosen section can be seen in Table 6 below where C_s, C_d, C_l is the snow load, dead load and live load placed on the column.

Level	Col Tag	Int/Ext	C_s (kN)	C_d (kN)	C_l (kN)	C_f (kN)	C_r (kN)	Chosen Section	C_f/C_r
4	J8	Int	123	69.2	61.7	333	-	-	-
	I8	Int	161.2	65	58	381.1	735	W200X42	0.52
	D6	**Int**	**64.9**	**56.8**	**50.8**	**219.2**	**395**	**W200X42**	**0.56**
	I4	**Ext**	**80**	**32**	**28.6**	**123.5**	**395**	**W200X42**	**0.32**
	D4	Ext	34.7	30.4	27.1	117.2	-	-	-
3	**J8**	**Int**	**123**	**319.7**	**269**	**926.1**	**1210**	**W200X52**	**0.77**
	I8	Int	161.2	300.1	258.1	923.5	-	-	-
	D4	Ext	34.7	195	145.5	496.7	-	-	-
	I4	Ext	80	205.5	150.7	563	-	-	-
	J4	**Ext**	**61**	**218.8**	**157.4**	**570.6**	**735**	**W200X42**	**0.78**
2	**J8**	**Int**	**123**	**570.3**	**411.2**	**1452.7**	**1680**	**W200X71**	**0.87**
	G4	**Int**	**106.5**	**358.8**	**231.6**	**902.4**		-	-
	F2-3	**Int**	**117.6**	**58.8**	**52.5**	**302.4**	**735**	**W200X42**	**0.42**
	J4	**Ext**	**61**	**383.1**	**237.7**	**896.5**	**1210**	**W200X52**	**0.75**
	F2-1	**Ext**	**33.6**	**29.4**	**26.25**	**113.5**	**735**	**W200X42**	**0.16**
1	**J8**	**Int**	**123**	**820.9**	**532.6**	**1948**	**2060**	**W200X86**	**0.95**
	G4	**Int**	**106.5**	**551**	**397**	**1391**		-	-
	F2-3	**Int**	**117.6**	**272**	**304.5**	**914.4**	**1210**	**W200X52**	**0.76**
	J4	**Ext**	**61**	**383.1**	**306.2**	**1205**	**1210**	**W200X52**	**0.99**
	F2-1	**Ext**	**33.6**	**136**	**153**	**433.1**	**735**	**W200X42**	**0.60**

Table 6: Column Design Summary

These results can be seen visually within the attached structural plans and also the column schedule.

9.5 CHEVRON BRACE DESIGN

9.5.1 Brace Design Process

There were various factors that influenced the design process of the braces. The first decision was what type of bracing we would use, and chevron and X bracing was considered. Chevron bracing was ultimately chosen due to its high elastic stiffness and strength (Razak, Kong, Zainol, Adnan, & Azimi, 2017) and that it would save more material overall. Placement of bracing was pushed more towards the corners of the building for increased moment resistance. It is important to note as well the number of braces that were used on the north and south face versus the amount used on the east and west face. A symmetric placement of the braces was also decided upon for ease of calculation but also so that the building would look better architecturally. It was also decided that the most heavily loaded brace per level would govern the design for all other braces on that level. Planning ahead for the connection design of each section, an area 1.4 times the required area through a gross area section calculation was found to accommodate for possible net section fracture that may occur during high tension.

9.5.2 Final Brace Design

In order to see detailed calculations concerning load analysis for the governing member, additional comments on the design and the final design for the member please refer to Appendix B. A summary of the design of the braces can be seen below in Table 7 and Table 8 which correspond to braces running east to west, and north to south respectively.

Level	Braces running E-W: T_f (kN)	T_r (kN)	T_f/T_r	Chosen Section	A_g
Roof	133.5	481.3	0.28	C150x12	1550
4	296.3	481.3	0.62	C150x12	1550
3	462.4	720.4	0.64	C180x18	2320
2	639.8	1102.3	0.58	C200x28	3550

Table 7: Design Summary for Braces Running East to West

Level	Braces running N-S: T_f (kN)	T_r (kN)	T_f/T_r	Chosen Section	A_g
Roof	176.7	481.3	0.37	C150x12	1550
4	373.1	574.4	0.65	C180x15	1850
3	532.9	810.4	0.66	C200x21	2610
2	700.8	1102.3	0.64	C200x28	3550

Table 8: Design Summary for Braces Running North to South

9.6 FLOOR SLAB AND STEEL DECK DESIGN

Floor slab and steel deck design in this project is based on CANAM steel deck design catalogue. Table 9 shows the summary of selected sections for floor slab and steel deck (CANAM, n.d.).

For detailed calculation, refer to Appendix B.

Table 9 shows factored and resistant load, deflection of chosen section for floor slab and steel deck design.

Level	Critical Case				Load		Deflection		Section
	Load type	Slab thick (mm)	Deck thick (mm)	Span (mm)	Factored load (kPa)	Resistance load (kPa)	Δ_{max} (mm)	Δ_{limit} (mm)	
Market Floor	Gravity	90	1.12	1812.5	13	15.99	0.62	5	P-3615
Main Building Floor	Gravity	90	1.12	1812.5	13	15.99	0.62	5	P-3615
Roof 1	Gravity	-	0.76	1812.5	4.32	8.94	15.4	22.4	P-2436 Type 22
	Uplift	-	0.76	1812.5	1.3	8.94	-	-	
Roof 2	Gravity	-	1.21	2125	10.62	13.19	5	5.9	P-2436 Type 18
	Uplift	-	1.21	2125	1.37	13.19	-	-	
Roof 3	Gravity	-	1.21	1875	10.62	13.19	5	5.9	P-2436 Type 18
	Uplift	-	1.21	1875	1.37	13.18	-	-	

Table 9: Floor Slab and Steel Deck Design Summary

REFERENCES

CANAM. (n.d.). *Steel Deck A division of Canam Group*.

Razak, S. M., Kong, T. C., Zainol, N. Z., Adnan, A., & Azimi, M. (2017). *A Review of Influence of Various Types of Structural Bracing to the Structural Performance of Buildings*. https://doi.org/10.1051/e3sconf/20183401010

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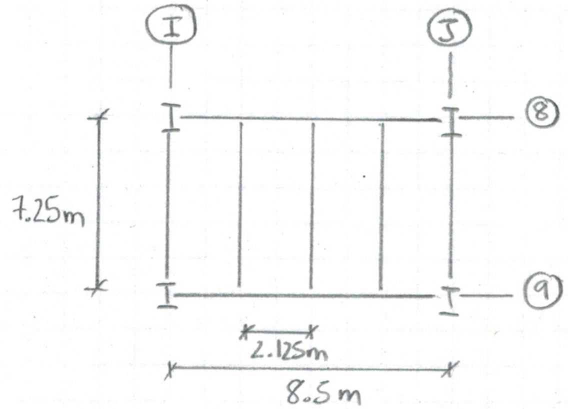
Joist Design for floor (main building)

- Choose critical case : (with largest TA).



DL = 4.06 KPa.

LL = 4.8 KPa.

$TA = 7.25 \times 2.125 = 15.4\ (m^2)$

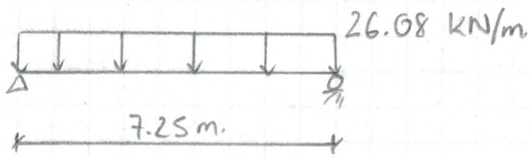
⇒ no Live Load reduction factor. — NBCC 4.1.5.8(1)

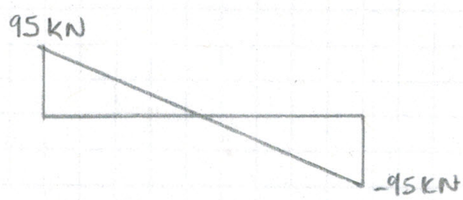
Load combination — NBCC Table 4.1.3.2 A

$w_f = 1.25DL + 1.5LL$

$= [1.25(4.06) + 1.5(4.8)] \times 2.125$

$w_f = 26.08\ (kN/m)$





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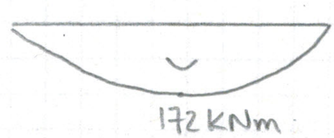

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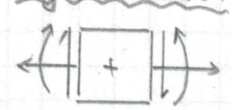

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$$M_{f\,max} = \frac{w_f L^2}{8} = 172\ (kNm)$$

$$V_{f\,max} = \frac{w_f L}{2} = 95\ (kN)$$

- Assume joist is laterally supported along compression flange. (top flange)
- Use steel ASTM A992

$F_y = 345$ MPa $F_u = 450$ MPa   S16-14 Table 6-7

- Set $M_r \geq M_{f\,max}$

$\Rightarrow \phi Z_x F_y \geq 172$ kNm

$\phi = 0.9$ S16-14 §13.1(a)

$\Rightarrow 0.9 (Z_x)(345) \geq 172 \times 10^6$

$\Rightarrow Z_x \geq 554 \times 10^3\ (mm^3)$

$$\Delta_{max} = \frac{5 w_L L^4}{384 E I_x} \leq \frac{L}{360}$$

$w_L = 10.2$ kN/m $= 10.2$ N/mm   S16-14 Table D-1. (Assume finishes susceptible to cracking).

$$\Rightarrow \frac{5(10.2)(7250)^4}{384(2\times10^5) I_x} \leq \frac{7250}{360}$$

$\Rightarrow I_x \geq 91.1 \times 10^6\ (mm^4)$

from $Z_x \geq 554 \times 10^3\ (mm^3)$ $I_x \geq 91.1 \times 10^6\ (mm^4)$

Try W310x45

$Z_x = 708 \times 10^3\ mm^3$ $I_x = 99.2 \times 10^6\ mm^4$

$S_x = 634 \times 10^3\ mm^3$

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$d = 313$ mm $b = 166$ mm $t = 11.2$ mm $w = 6.6$ mm

Check section class: S16-14 Table 5.1

Flange: $\frac{b_{el}}{t} = 7.41 < \frac{145}{\sqrt{F_y}} = 7.81 \rightarrow$ Class 1.

Web: $\frac{h}{w} = 44 < \frac{1100}{\sqrt{F_y}} = 59.2 \rightarrow$ Class 1.

$\Rightarrow$ Section class 1

Check moment capacity

$M_r = \phi Z_x F_y = 0.9\,(708\times10^3)(345)$ S16-14 §13.5(a)

$M_r = 219\,(kNm) > M_{f\,max} = 172\,(kNm) \Rightarrow$ Okay

Check shear capacity: S16-14 §13.4.1.1

$\frac{h}{w} = 44 < \frac{1041}{\sqrt{F_y}} = 56.$ S16-14 §13.4.1.1(a)

$\Rightarrow F_s = 0.66\,F_y$

$V_r = \phi A_w F_s = 0.9\,(313)(6.6)(0.66\times345)$

$\Rightarrow V_r = 423\,(kN) > V_{f\,max} = 95\,(kN) \Rightarrow$ okay

Check deflection

$$\Delta_{max} = \frac{5 w_L L^4}{384 E I_x} = \frac{5(10.2)(7250)^4}{384(2\times10^5)(99.2\times10^6)} = 18.5\,(mm).$$

$$\Delta_{Limit} = \frac{L}{360} = \frac{7250}{360} = 20.1\,(mm).$$

$\Delta_{max} < \Delta_{Limit} \Rightarrow$ okay.

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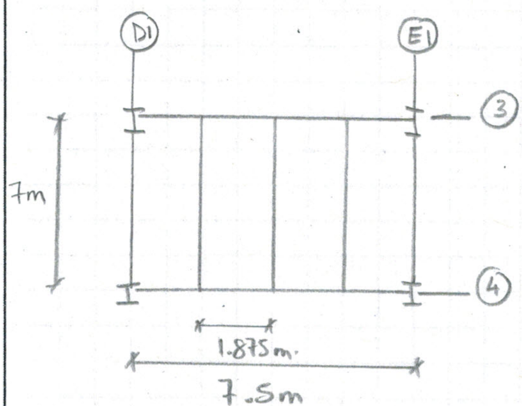
$$\frac{M_f}{M_r} = \frac{172}{219} = 0.79$$

$$\frac{V_f}{V_r} = \frac{95}{423} = 0.22$$

Joist design for floor (market place).

Choose critical case (with largest TA)



$DL = 4.06\ kPa$.

$LL = 4.8\ kPa$.

$TA = 7 \times 1.875 = 13.125\ (m^2)$

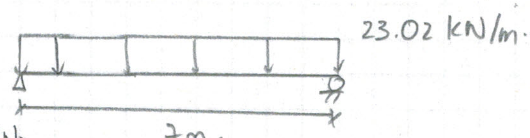
$\Rightarrow$ No LLRF — NBCC 4.1.5.8(1)

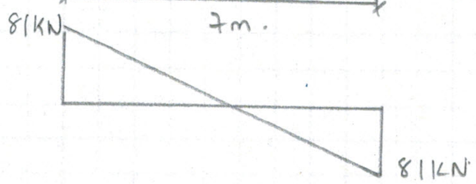
Load combination:

$W_f = 1.25\,DL + 1.5\,LL$.

$= [1.25\,(4.06) + 1.5\,(4.8)] \times 1.875$.

$W_f = 23.02\ (kN/m)$





SFD

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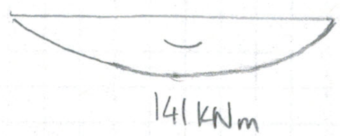
BMD.



$$M_{f\,max} = \frac{w_f L^2}{8} = 141 (kNm.)$$

$$V_{f\,max} = \frac{w_f L}{2} = 81 (kN)$$

Set $M_r \geqslant M_{f\,max}$

$\Rightarrow \phi Z_x F_y \geqslant 141$ kNm. — S16-14 § 13.5 (a)

$\Rightarrow 0.9 (Z_x)(345) \geqslant 141 \times 10^6$.

$\Rightarrow Z_x \geqslant 454 \times 10^3 \; (mm^3)$

$$\Delta_{max} = \frac{5 w_L L^4}{384 E I_x} \leq \frac{L}{360}$$

$w_L = 9$ kN/m $= 9$ N/mm — S16-14 Table D-1. Assume finishes susceptible to cracking.

$$\Rightarrow \frac{5(9)(7000)^4}{384(2\times10^5) I_x} \leq \frac{7000}{360}$$

$\Rightarrow I_x \geqslant 72 \times 10^6 \; (mm^4)$.

from $Z_x \geqslant 454 \times 10^3 \; (mm^3)$ $\quad I_x \geqslant 72 \times 10^6 (mm^4)$

Try W200 x71

$Z_x = 803 \times 10^3 (mm^3)$ $\quad I_x = 76.6 \times 10^6 \; (mm^4)$

$S_x = 709 \times 10^3 (mm^3)$

$d = 216$ mm $\quad b = 206$ mm. $\quad t = 17.4$ mm. $\quad w = 10.2$ mm

Check section class: — S16-14 Table 5-1

Flange: $\frac{b_{el}}{t} = 5.92 < \frac{145}{\sqrt{F_y}} = 7.81 \rightarrow$ Class 1

Web: $\frac{h}{w} = 17.8 < \frac{1100}{\sqrt{F_y}} = 59.2 \rightarrow$ Class 1

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⇒ Section class 1

Check moment capacity

$M_r = \phi Z_x F_y = 0.9(803 \times 10^3)(345)$ S16-14 §13.5(a)

$M_r = 249$ (KNm) $> M_{r\,max} = 141$ (KNm) ⇒ okay

Check shear capacity: S16-14 §13.4.1.1

$\frac{h}{w} = 17.8 < \frac{1041}{\sqrt{F_y}} = 56$

$\Rightarrow F_s = 0.66 F_y$

$V_r = \phi A_w F_s = 0.9(216)(10.2)(0.66 \times 345)$

$\Rightarrow V_r = 452$ (KN) $> V_{f\,max} = 81$ (KN). ⇒ okay.

Check deflection

$\Delta_{max} = \frac{5 w_L L^4}{384 E I_x} = \frac{5(9)(7000)^4}{384(2\times10^5)(76.6\times10^6)} = 18.3$ (mm).

$\Delta_{Limit} = \frac{L}{360} = \frac{7000}{360} = 19.4$ (mm)

$\Delta_{max} < \Delta_{Limit}$ ⇒ okay

$\frac{M_f}{M_r} = \frac{141}{249} = 0.57$

$\frac{V_f}{V_r} = \frac{81}{452} = 0.18$

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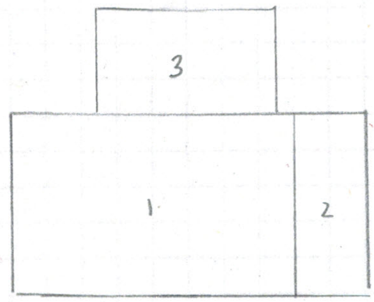


$DL = 1.12\,kPa$

$LL = 1.0\,kPa.$

$\Rightarrow$ No LLRF — NBCC Table 4.1.3.2-A

Consider load combination for critical case. — NBCC Table 4.1.3.2-A

$1.25DL + 1.5LL + 1.0SL.$

$1.25DL + 1.5SL + 1.0LL.$

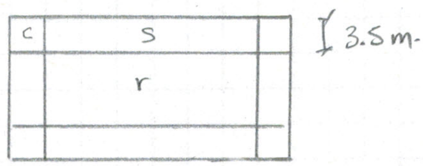
$0.9DL + 1.4WL$ (consider for uplift).

Roof 1:

$DL = 1.12\,kPa$ $LL = 1.0\,kPa.$

$SL_u = 1.28\,kPa$ $SL_s = 1.152\,kPa.$ (from Chapter 6)

Wind load:



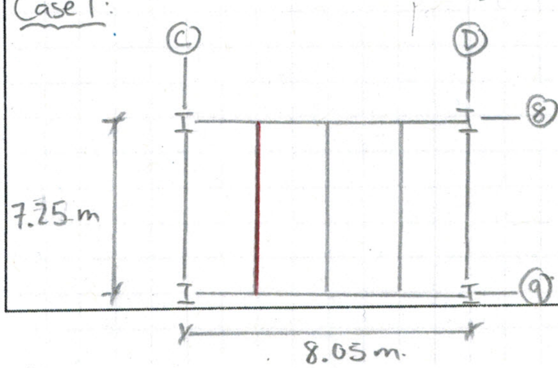
From Chapter 6:

$P_c = -1.7\,kPa$

$P_s = -1.3\,kPa$

$P_r = -0.9\,kPa.$

Case 1:



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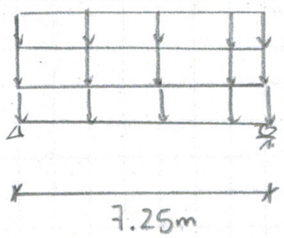


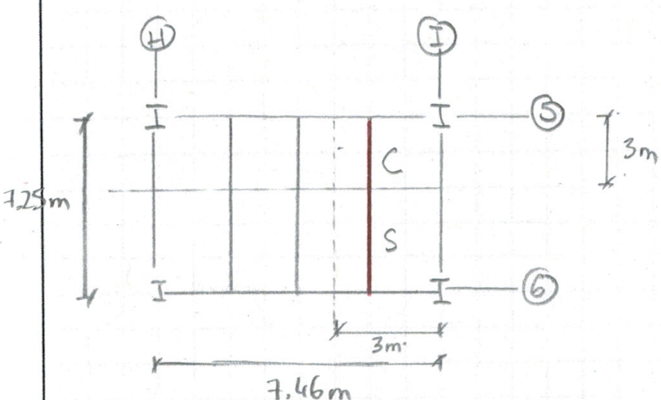
$$1.25DL + 1.5LL + 1.0SL = 2.0125(1.25\times1.12 + 1.5\times1 + 1\times1.28)$$

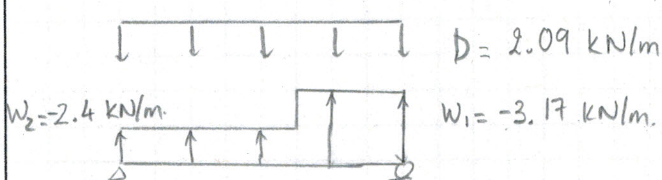
$$= 8.41\ (kN/m)$$

$$1.25DL + 1.5SL + 1.0LL = 2.0125(1.25\times1.12 + 1.5\times1.28 + 1\times1)$$

$$= 8.70\ (kN/m) \quad \text{(critical)}$$

Case 2:





for conservative take $WL = -3.17$ kN/m

$$0.9DL + 1.4WL = 0.9\times2.09 + 1.4(-3.17)$$

$$= -2.56\ (kN/m)$$

$\Rightarrow$ uplift happens

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$W_{f1} = 8.7$ KN/m

$L_1 = 7.25$ m.

$M_{fmax1} = \frac{w_{f1} L_1^2}{8} = 58$ KNm.

$V_{fmax1} = \frac{w_{f1} L_1}{2} = 32$ KN.

$W_{f2} = -2.56$ KN/m

$L_2 = 7.25$ m

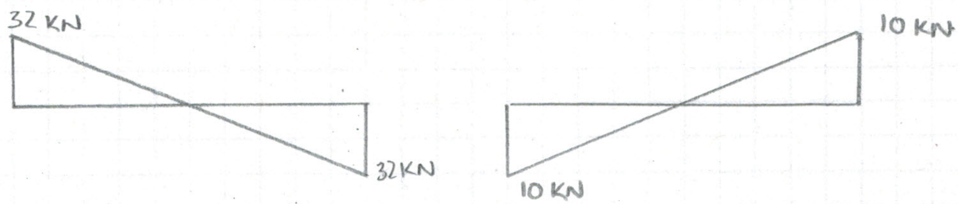
$M_{fmax2} = \frac{w_{f2} L_2^2}{8} = -17$ KNm

$V_{fmax2} = \frac{w_{f2} L_2}{2} = -10$ KN.

Laterally supported for compression flange

No laterally supported for compression flange

SFD



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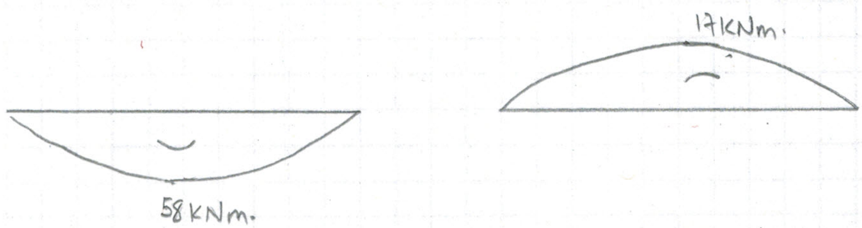

Design joist based on case 1 then check for case 2.

Set $M_{r1} \geq M_{f1max}$

$\Rightarrow \phi Z_x F_y \geq 58$ KNm

$\Rightarrow 0.9 (Z_x)(345) \geq 58 \times 10^6$

$\Rightarrow Z_x \geq 187 \times 10^3$ (mm^3)

$\Delta_{max} = \frac{5 w L^4}{384 E I_x} \leq \frac{L}{360}$

$\Rightarrow I_x \geq 20.7 \times 10^6$ (mm^4)

$W = max(W_L, W_s)$

$W = max(2.0125, 2.448)$.

$W = 2.3184$ (KN/m)

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from $Z_x \geq 187 \times 10^3\ mm^3$ and $I_x \geq 20.7 \times 10^6\ mm^4$

Try W200 x 31

$Z_x = 335 \times 10^3\ mm^3$ $I_y = 4.1 \times 10^6 mm^4$ $d = 210\ mm$ $w = 6.4 mm$

$S_x = 299 \times 10^3\ mm^3$ $J = 119 \times 10^3\ mm^4$ $b = 134\ mm$

$I_x = 31.4 \times 10^6\ mm^4$ $C_w = 40.9 \times 10^9\ mm^6$ $t = 10.2\ mm$

Check section class — S16-14 Table 5.1

Flange: $\frac{b_{el}}{t} = 6.57 < \frac{145}{\sqrt{F_y}} = 7.81 \rightarrow$ Class 1.

Web: $\frac{h}{w} = 29.6 < \frac{1100}{\sqrt{F_y}} = 59.2 \rightarrow$ class 1.

$\Rightarrow$ Section class 1.

Check moment capacity: — S16-14 §13.5(a)

$M_{r_1} = \phi Z_x F_y = 0.9 (335 \times 10^3)(345)$

$M_{r_1} = 104\ (kNm) > M_{f max 1} = 58 (kNm) \Rightarrow$ okay.

Check shear capacity: — S16-14 §13.4.1.1

$\frac{h}{w} = 29.6 < \frac{1041}{\sqrt{F_y}} = 56$

$\Rightarrow F_s = 0.66 F_y$.

$V_{r_1} = \phi A_w F_s = 0.9(210)(6.4)(0.66 \times 345)$

$\Rightarrow V_{r_1} = 275\ (kN) > V_{f max 1} = 32\ (kN) \Rightarrow$ okay.

Check deflection

$\Delta_{max} = \frac{5 w_L L^4}{384 E I_x} = 13.3\ mm < \frac{L}{360} = 20.1\ mm \Rightarrow$ okay.

$\frac{M_{f1}}{M_{r1}} = \frac{58}{104} = 0.56$

$\frac{V_{f1}}{V_{r1}} = \frac{32}{275} = 0.12$

Check for case 2 (laterally unbraced).

$L_u = 7250\ mm.$

Check moment capacity: S16-14 §13.6(a)(ii)

$M_{max} = 17\ kNm$

$M_a = 15.8\ kNm \qquad M_b = 17\ kNm \qquad M_c = 15.8\ kNm$

$$\omega_2 = \frac{4M_{max}}{\sqrt{M_{max}^2 + 4M_a^2 + 7M_b^2 + 4M_c^2}} = 1.036 < 2.5$$

$$M_u = \frac{\omega_2 \pi}{L_u}\sqrt{EI_y GJ + \left(\frac{\pi E}{L_u}\right)^2 I_y C_w}$$

$$= \frac{1.036\pi}{7250}\sqrt{(2\times10^5)(4.1\times10^6)(77000)(119\times10^3) + \left(\frac{\pi\, 2\times10^5}{7250}\right)^2 (4.1\times10^6)(40.9\times10^9)}$$

$M_u = 42.04\ (kNm)$

$M_p = Z_x F_y = 116\ (kNm)$

$M_u = 42.04\ (kNm) < 0.67M_p = 77\ (kNm)$

$\Rightarrow M_{r2} = \phi M_u = 0.9(42.04).$

$M_{r2} = 38\ (kNm) > M_{fmax2} = 17(kNm) \Rightarrow$ okay.

Check shear capacity

$V_{r2} = V_{r1} = 275\ (kN) > V_{fmax2} = 10\ kN \Rightarrow$ okay

$\frac{M_{f2}}{M_{r2}} = \frac{17}{38} = 0.45$

$\frac{V_{f2}}{V_{r2}} = \frac{10}{275} = 0.04$

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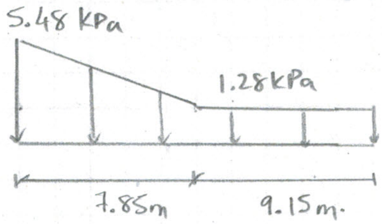
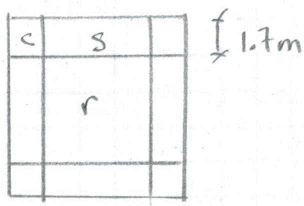
Roof 2:

DL = 1.12 kPa LL = 1.0 kPa.

Snow load from Chapter 6



Wind Load from Chapter 6.

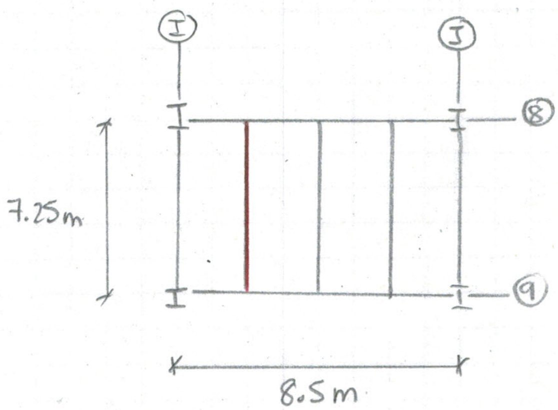


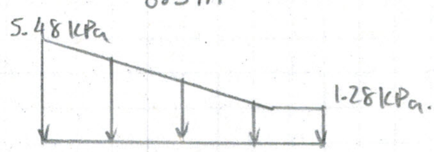
$P_c = -1.7$ kPa

$P_s = -1.3$ kPa

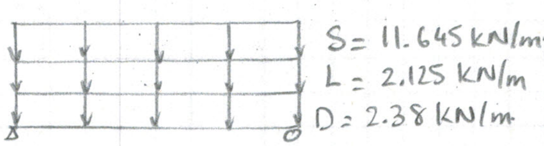
$P_r = -0.9$ kPa

Case 1:





for conservative, take 5.48 kPa for designed snow load.



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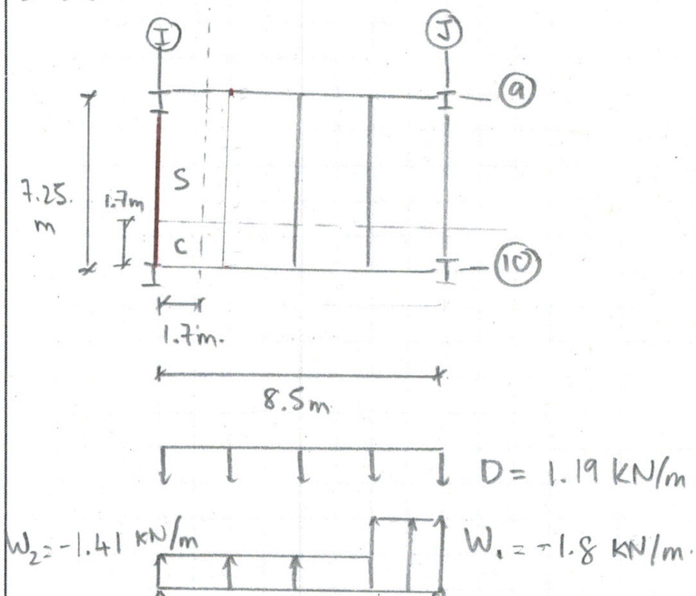
$1.25DL + 1.5LL + 1.0SL = 1.25(2.38) + 1.5(2.125) + 1(11.645)$

$= 17.8 \ (kN/m)$

$1.25DL + 1.5SL + 1.0LL = 1.25(2.38) + 1.5(11.645) + 1.0(2.125)$

$= 22.6 \ (kN/m)$ (critical)

Case 2:



for conservative take $WL = -1.8$ kN/m

$0.9DL + 1.4WL = 0.9(1.19) + 1.4(-1.8)$

$= -1.45 \ (kN/m)$

$\Rightarrow$ uplift happens

$w_{f1} = 22.6$ kN/m. $\qquad$ $w_{f2} = -1.45$ kN/m.

$L_1 = 7.25$ m. $\qquad$ $L_2 = 7.25$ m

$M_{f\,max1} = \frac{w_{f1} L_1^2}{8} = 149$ kNm $\qquad$ $M_{f\,max2} = \frac{w_{f2} L_2^2}{8} = -10$ kNm.

$V_{f\,max1} = \frac{w_{f1} L_1}{2} = 82$ kN $\qquad$ $V_{f\,max2} = \frac{w_{f2} L_2}{2} = -6$ kN

Laterally supported $\qquad$ No laterally supported

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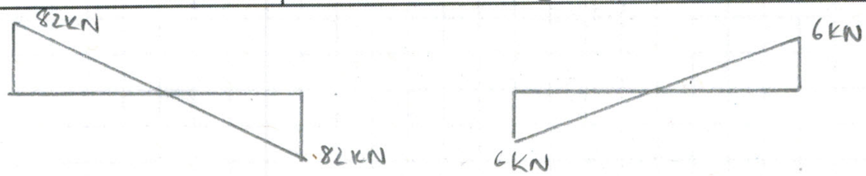

BMD

Design based on case 1 then check for case 2.

Set $M_{r1} \geq M_{f\,max}$

$\Rightarrow \phi Z_x F_y \geq 149\ kNm$

$\Rightarrow Z_x \geq 480 \times 10^3\ (mm^3)$

$\Delta_{max} = \frac{5wL^4}{384EI_x} \leq \frac{L}{360}$ $\qquad w = max(w_L, w_S)$

$= max(2.125, 10.48)$.

$\Rightarrow I_x \geq 94 \times 10^6\ (mm^4)$ $\qquad w = 10.48\ kN/m$

from $Z_x > 480 \times 10^3\ mm^3$ and $I_x > 94 \times 10^6\ mm^4$

Try W360x39

$Z_x = 662 \times 10^3\ mm^3$ $\quad I_y = 3.75 \times 10^6\ mm^4$ $\quad d = 353\ mm$ $\quad w = 6.5\ mm$

$S_x = 580 \times 10^3\ mm^3$ $\quad J = 150 \times 10^3\ mm^4$ $\quad b = 128\ mm$

$I_x = 102 \times 10^6\ mm^4$ $\quad C_w = 110 \times 10^9\ mm^6$ $\quad t = 10.7\ mm$

Check section class $\qquad$ S16-14 Table 5-1

Flange: $\frac{bel}{t} = 5.98 < \frac{145}{\sqrt{F_y}} = 7.81 \rightarrow$ Class 1

Web = $\frac{h}{w} = 51 < \frac{1100}{\sqrt{F_y}} = 59.2 \rightarrow$ class 1

$\Rightarrow$ Section class 1.

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Check moment capacity — S16-14 §13.5(a)

$M_{r1} = \phi Z_x F_y = 0.9(662 \times 10^3)(345)$

$M_{r1} = 205\ (kNm) > M_{fmax1} = 149\ (kNm) \Rightarrow$ okay

Check shear capacity — S16-14 §13.4.1.1

$\frac{h}{w} = 51 < \frac{1041}{\sqrt{F_y}} = 56$

$\Rightarrow F_s = 0.66 F_y$.

$V_{r1} = \phi A_w F_s = 0.9(353)(6.5)(345 \times 0.66)$

$V_{r1} = 470\ (kN) > V_{f1max} = 82\ (kN) \Rightarrow$ okay.

Check deflection:

$\Delta_{max} = \frac{5wL^4}{384EI_x} = 18.4 (mm) < \frac{L}{360} = 20.1\ mm \Rightarrow$ okay

$\frac{M_{f1}}{M_{r1}} = \frac{149}{205} = 0.73$

$\frac{V_{f1}}{V_{r1}} = \frac{82}{470} = 0.17$.

Check for case 2 (laterally unbraced)

$L_u = 7250\ mm$.

Check moment capacity — S16-14 §13.6(a)(ii)

$M_{max} = 10\ kNm$

$M_a = 8.17\ kNm \qquad M_b = 10\ kNm \qquad M_c = 8.17\ kNm$.

$\omega_2 = 1.095 < 2.5$

$M_u = \frac{\omega_2 \pi}{L_u} \sqrt{EI_y GJ + \left(\frac{\pi E}{L_u}\right)^2 I_y C_w} = 51\ kNm$.

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$M_p = Z_x F_y = 228$ kNm

$M_u < 0.67 M_p$

$\Rightarrow M_{r2} = \phi M_u = 0.9(51)$

$M_{r2} = 45$ (kNm) $> M_{f2max} = 10$ kNm $\Rightarrow$ okay

Check shear capacity

$V_{r2} = V_{r1} = 470$ (kN)

$V_{r2} > V_{f2max} = 6$ kN $\Rightarrow$ okay

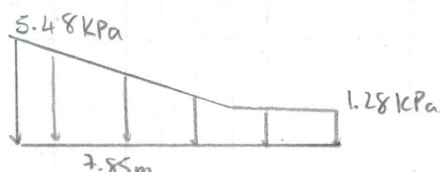
$$\frac{M_{f2}}{M_{r2}} = \frac{10}{45} = 0.22$$

$$\frac{V_{f2}}{V_{r2}} = \frac{6}{470} = 0.01$$

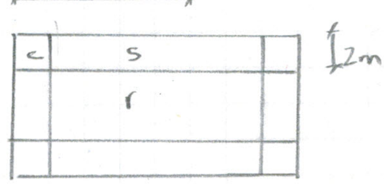
Roof 3:

DL = 1.12 kPa   LL = 1.0 kPa

Snow Load from chapter 6



Wind Load from chapter 6

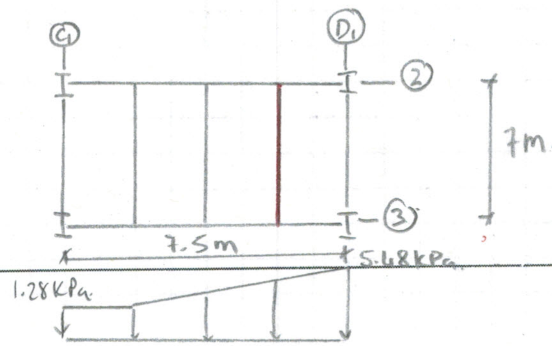


$P_c = -1.7$ kPa

$P_s = -1.3$ kPa

$P_r = -0.9$ kPa

Case 1



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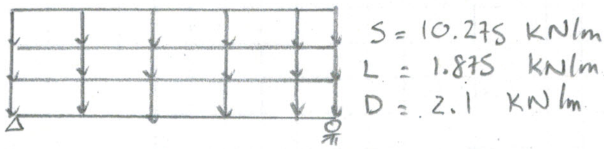
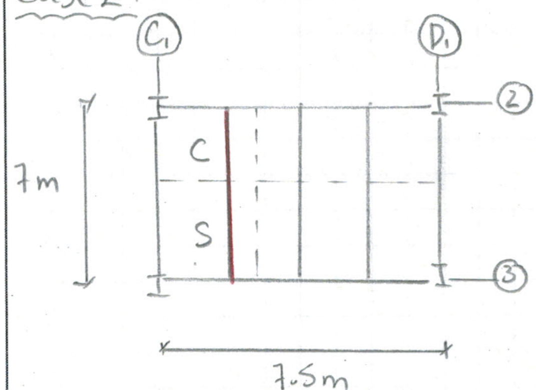
for conservative, take 5.48 kPa for designed snow load

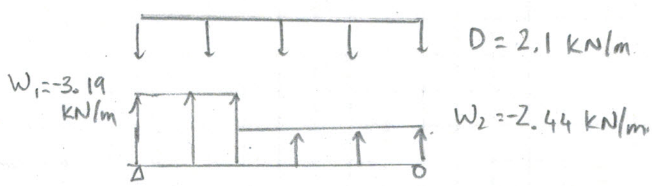


$1.25DL + 1.5LL + 1.0SL = 15.71$ (kN/m)

$1.25DL + 1.5SL + 1.0LL = 20.0$ (kN/m) (critical)

Case 2:





for conservative take $WL = -3.19$ kN/m.

$0.9DL + 1.4WL = -2.576$ (kN/m)

$\Rightarrow$ Uplift happens.

$W_{f1} = 20$ kN/m. $\qquad W_{f2} = -2.576$ kN/m.

$L_1 = 7000$ mm $\qquad L_2 = 7000$ mm.

$M_{f1max} = \frac{W_{f1} L_1^2}{8} = 123$ kNm. $\qquad M_{f2max} = \frac{W_{f2} L_2^2}{8} = -16$ kNm

$V_{f1max} = \frac{W_{f1} L_1}{2} = 70$ kN $\qquad V_{f2max} = \frac{W_{f2} L_2}{2} = -9$ kN

Laterally supported $\qquad$ Laterally unsupported

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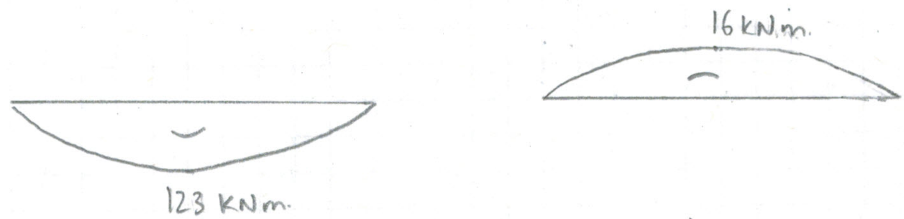
SFD

BMD



Design for case 1 then check for case 2.

Set $M_{r_1} \geq M_{f, max}$

$\Rightarrow \phi Z_x F_y \geq 123$ kNm

$\Rightarrow Z_x \geq 336 \times 10^3$ (mm^3).

$$\Delta_{max} = \frac{5wL^4}{384EI_x} \leq \frac{L}{360}$$

$w = \max(w_L, w_S)$

$= \max(1.875, 9.25)$

$w = 9.25$ kN/m.

$\Rightarrow I_x \geq 74 \times 10^6$ (mm^4)

from $Z_x \geq 336 \times 10^3\ mm^3$ and $I_x \geq 74 \times 10^6\ mm^4$

Try W360x39

$Z_x = 662 \times 10^3\ mm^3$ $I_y = 3.75 \times 10^6\ mm^4$ $b = 353$ mm $w = 6.5$ mm

$S_x = 580 \times 10^3\ mm^3$ $J = 150 \times 10^3\ mm^4$ $d = 128$ mm

$I_x = 102 \times 10^6\ mm^4$ $C_w = 110 \times 10^9\ mm^6$ $t = 10.7$ mm.

Check section class — S16-14 Table 5-1.

Flange: $\frac{b_{el}}{t} = 5.98 < \frac{170}{\sqrt{F_y}} = 9.15 \rightarrow$ Class 2.

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Web: $\frac{h}{w} = 51 < \frac{1100}{\sqrt{F_y}} = 59.2 \rightarrow$ class 1

$\Rightarrow$ Section class 2

Check moment capacity — S16-14 § 13.5(a)

$M_{r_1} = \phi Z_x F_y = 0.9(662 \times 10^3)(345)$

$M_{r_1} = 205\ (kNm) > M_{f,max} = 123\ (kNm) \Rightarrow$ okay

Check shear capacity — S16-14 § 13.4.1.1

$\frac{h}{w} = 51 < \frac{1041}{\sqrt{F_y}} = 56$

$\Rightarrow F_s = 0.66 F_y$

$V_{r_1} = \phi A_w F_s = 0.9(353)(6.5)(0.66 \times 345)$

$\Rightarrow V_{r_1} = 470\ (kN) > V_{f,max} = 70\ (kN) \Rightarrow$ okay

Check deflection

$\Delta_{max} = \frac{5wL^4}{384EI_x} = 14\ (mm) < \frac{L}{360} = 19.4\ (mm) \Rightarrow$ okay

$\frac{M_{f_1}}{M_{r_1}} = \frac{123}{205} = 0.60$

$\frac{V_{f_1}}{V_{r_1}} = \frac{70}{470} = 0.15$

Check for case 2 (laterally unsupported)

$L_u = 7000\ mm$

Check moment capacity — S16-14 13.6(a)(ii)

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$M_{max} = 15.8$ kNm

$M_a = 11.8$ kNm $\quad M_b = 15.8$ kNm $\quad M_c = 11.8$ kNm.

$\omega_2 = 1.133$

$$M_u = \frac{\omega_2 \pi}{L_u}\sqrt{E I_y G J + \left(\frac{\pi E}{L_u}\right)^2 I_y C_w} = 55 \text{ (kNm)}$$

$M_p = Z_x F_y = 228$ kNm

$M_u < 0.67 M_p$

$\Rightarrow M_{r_2} = \phi M_u = 0.9(55)$

$\Rightarrow M_{r_2} = 49$ kNm $> M_{f_2 max} = 16$ kNm $\Rightarrow$ okay

Check shear capacity

$V_{r_2} = V_{r_1} = 470$ (kN) $> V_{f_2 max} = 9$ (kN) $\Rightarrow$ okay

$$\frac{M_{f_2}}{M_{r_2}} = \frac{16}{49} = 0.33$$

$$\frac{V_{f_2}}{V_{r_2}} = \frac{9}{470} = 0.02$$

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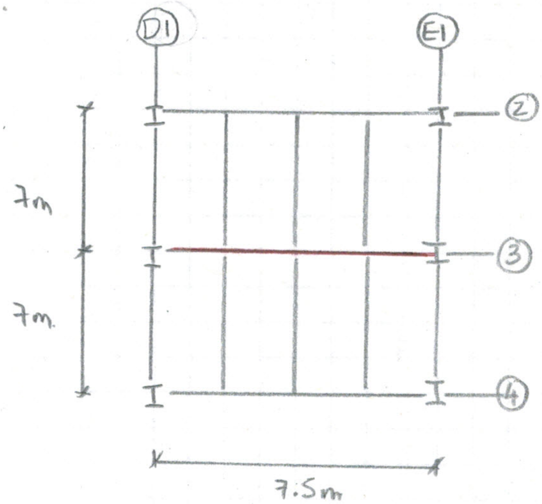
Beam design for floor (market place)

DL = 4.06 kPa

LL = 4.8 kPa

- Choose critical case (with largest TA)



TA = 7m x 7.5m

TA = 52.5 m²

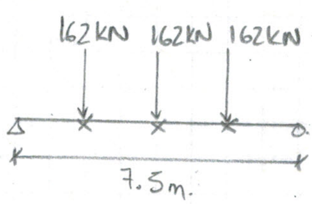
$\begin{cases} TA < 80\,m^2 \\ LL = 4.8\ kPa \end{cases}$ — Assembly occupancy area

⇒ No LLRF — NBCC 4.1.5.8(1).

$P_f = 81 \times 2 = 162$ (KN)

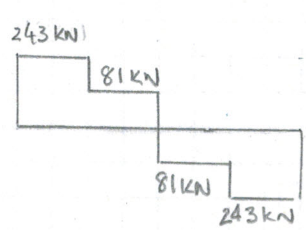
$P_L = 2\frac{w_L L}{2} = 4.8 \times 7 = 33.6$ (KN)

- Beam is laterally braced compression flange at joists.



$L_u = 1.875m$

SFD



$V_{f\,max} = 1.5\,(162) = 243$ (kN)

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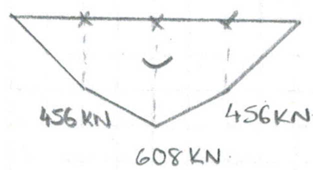
BMD:



$M_{f_{max}} = 608$ (KNm).

Try W460 x 89

$Z_x = 2010 \times 10^3\ mm^3$ $I_y = 20.9 \times 10^6\ mm^4$ $d = 463\ mm$ $w = 10.5\ mm$

$S_x = 1770 \times 10^3\ mm^3$ $J = 905 \times 10^3\ mm^4$ $b = 192\ mm$.

$I_x = 409 \times 10^6\ mm^4$ $C_w = 1040 \times 10^9\ mm^6$ $t = 17.7\ mm$

Check section class — S16-14 Table 5-1

Flange: $\frac{b_{el}}{t} = 5.42 < \frac{145}{\sqrt{F_y}} = 7.81 \rightarrow$ class 1.

Web: $\frac{h}{w} = 40.7 < \frac{1100}{\sqrt{F_y}} = 59.2 \rightarrow$ class 1

$\Rightarrow$ Section class 1

Check moment capacity — S16-14 S13.6.(a)(i)

$L_u = 1875\ mm$.

$M_{max} = 608\ kNm$.

$M_a = 494\ kNm$ $M_b = 532\ kNm$ $M_c = 570\ kNm$.

$\omega_2 = 1.13 < 2.5$.

$$M_u = \frac{\omega_2 \pi}{L_u} \sqrt{E I_y G J + \left(\frac{\pi E}{L_u}\right)^2 I_y C_w} = 3131\ (kNm)$$

$M_p = Z_x F_y = 693\ (kNm)$

$M_u > 0.67 M_p$

$$\Rightarrow M_r = 1.15 \phi M_p \left[1 - \frac{0.28 M_p}{M_u}\right] = 673\ (kNm) > \phi M_p = 624\ kNm$$

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$\Rightarrow M_r = 624 (kNm) > M_{f\,max} = 608 (kNm) \Rightarrow$ okay

Check shear capacity: S16-14 §13.4.1.1

$\frac{h}{w} = 40.7 < \frac{1041}{\sqrt{F_y}} = 56$

$\Rightarrow F_s = 0.66 F_y$

$V_r = \phi A_w F_s = 0.9 (463)(10.5)(0.66 \times 345)$

$\Rightarrow V_r = 996 (kN) > V_{f\,max} = 243 (kN) \Rightarrow$ okay

Check deflection:

$\Delta_{max} = \frac{P_L a}{24 E I_x}(3L^2 - 4a^2) + \frac{P_L L^3}{48 E I_x}$ $\quad P_L = 33.6$ kN

$= 8.57 (mm)$. $\quad a = 1875$ mm.

$\Delta_{max} = 8.57 (mm) < \frac{L}{360} = 20.8 (mm) \Rightarrow$ okay

$\frac{M_f}{M_r} = \frac{608}{624} = 0.97$

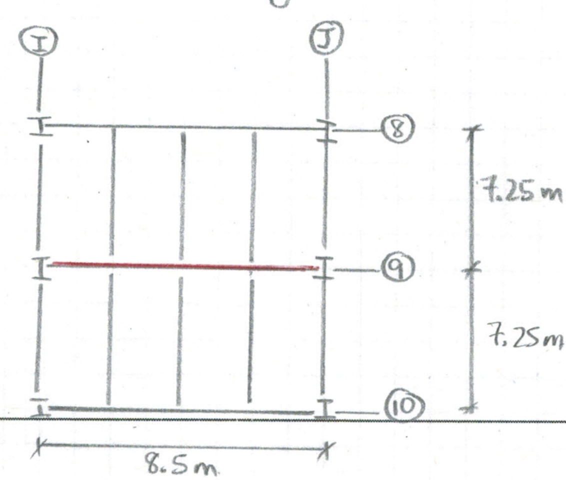
$\frac{V_f}{V_r} = \frac{243}{996} = 0.24$

Beam design for floor (main building)

DL = 4.06 kPa

LL = 4.8 kPa

Choose critical case (with largest TA)



TA = 61.62 m^2 > 20 m^2

$\Rightarrow LLRF = 0.3 + \sqrt{9.8/61.62}$ NBCC 4.1.5.8(3)

LLRF = 0.7

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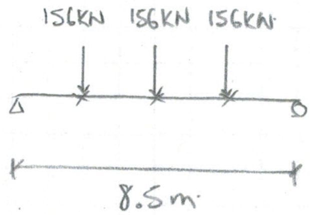
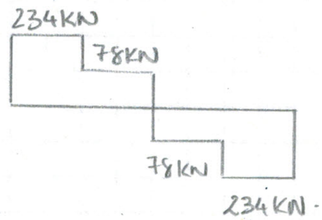
$P_f = 156\ (KN)$

$P_L = 74\ (KN)$



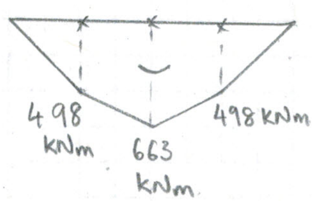
$L_u = 2.125\ m$

SFD



$V_{f\,max} = 234 (KN)$

BMD



$M_{f\,max} = 663 (kNm)$

Try W530x92

$Z_x = 2360 \times 10^3\ mm^3$ $I_y = 23.8 \times 10^6\ mm^4$ $d = 533mm$ $w = 10.2\ mm$

$S_x = 2070 \times 10^3\ mm^3$ $J = 762 \times 10^3\ mm^4$ $b = 209mm$

$I_x = 552 \times 10^6\ mm^4$ $C_w = 1590 \times 10^9\ mm^6$ $t = 15.6mm$

Check section class: S16-14 Table 5-1

Flange : $\frac{b_{el}}{t} = 6.7 < \frac{145}{\sqrt{F_y}} = 7.81 \rightarrow$ class 1

Web : $\frac{h}{w} = 49.2 < \frac{1100}{\sqrt{F_y}} = 59.2 \rightarrow$ class 1

$\Rightarrow$ Section class 1

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S16-14 §13.6(a)(i)

Check moment capacity

$L = 2125\,mm$

$M_{max} = 663\ KNm$

$M_a = 539.25\ KNm \quad M_b = 580.5\ KNm \quad M_c = 621.75\ KNm$

$\omega_2 = 1.13 < 2.5$

$$M_u = \frac{\pi \omega_2}{L_u}\sqrt{EI_y GJ + \left(\frac{\pi E}{L_u}\right)^2 I_y C_w} = 3164\ (KNm)$$

$M_u = 3164\ KNm > 0.67 M_p = 546\ KNm$

$$\Rightarrow M_r = 1.15\phi M_p \left[1 - \frac{0.28 M_p}{M_u}\right] = 782\ KNm > \phi M_p = 732\ KNm$$

$\Rightarrow M_r = 732 (KNm) > M_{f max} = 663 (KNm) \Rightarrow$ okay

Check shear capacity.

$$\frac{h}{w} = 49.2 < \frac{1041}{\sqrt{F_y}} = 56$$

$\Rightarrow F_s = 0.66 F_y$

$V_r = \phi A_w F_s = 0.9(533)(10.2)(0.66 \times 345)$

$\Rightarrow V_r = 1114\ (KN) > V_{f max} = 234\ (KN) \Rightarrow$ okay

Check deflection:

$$\Delta_{max} = \frac{P_L a}{24EI_x}(3L^2 - 4a^2) + \frac{P_L L^3}{48EI_x} = 20.4\ (mm)$$

$P_L = 74\,KN$

$a = 2125\,mm$

$$\Delta_{max} = 20.4\ (mm) < \frac{L}{360} = 23.6\ (mm) \Rightarrow \text{okay.}$$

$$\frac{M_f}{M_r} = \frac{663}{732} = 0.91$$

$$\frac{V_f}{V_r} = \frac{234}{1114} = 0.21$$

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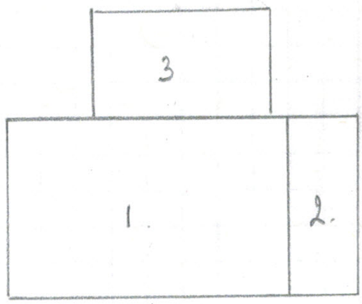


DL = 1.12 kPa.

LL = 1 kPa

$\Rightarrow$ No LLRF. NBCC Table 4.1.3.2-A.

Consider load combination for critical case: NBCC Table 4.1.3.2-A

1.25 DL + 1.5 LL + 1.0 SL

1.25 DL + 1.5 SL + 1.0 LL

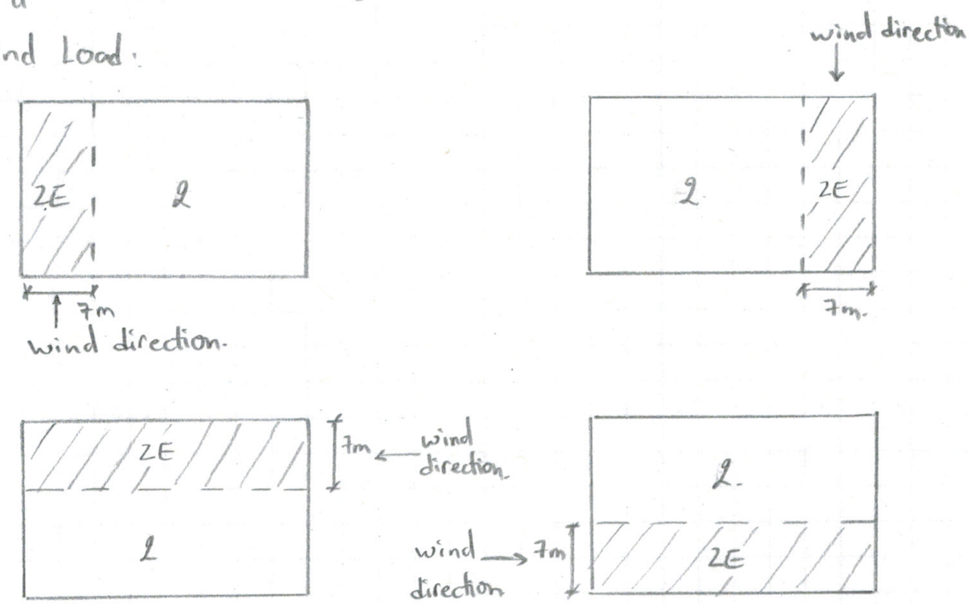
0.9 DL + 1.4 WL (consider for uplift)

Roof 1:

DL = 1.12 kPa LL = 1.0 kPa

SL_u = 1.28 kPa SL_s = 1.152 kPa.

Wind Load:



P_2 = −0.676 kPa P_{2E} = −1.04 kPa.

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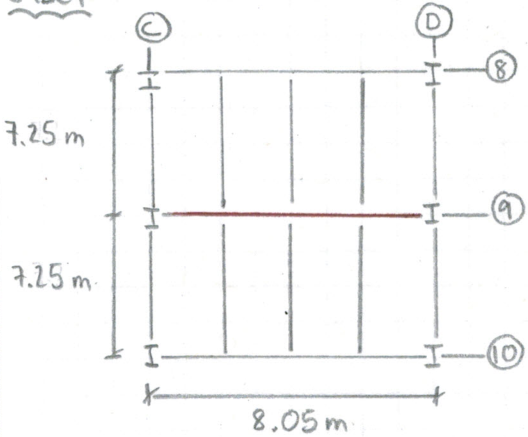
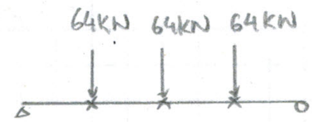
Case 1:

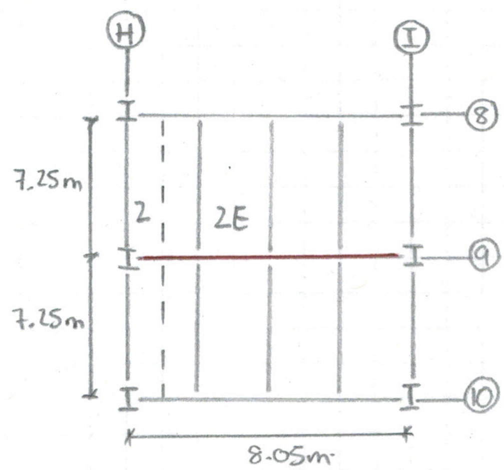


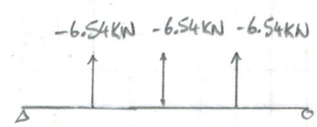


Critical case: 1.25DL + 1.5SL + 1.0LL.

$\Rightarrow P_{f_1} = 64$ (KN)

Case 2:





Critical case: 0.9DL + 1.4WL

For conservative, assume all members lie in zone 2E

$\Rightarrow P_{f_2} = -6.54$ (KN)

$P_{f_1} = 64$ (KN) $\quad$ $P_{f_2} = -6.54$ (KN)

$L_{u_1} = 2.0125$ m $\quad$ $L_{u_2} = 8.05$ m

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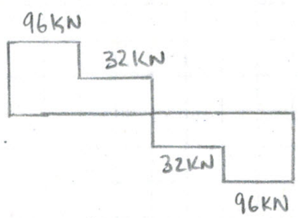
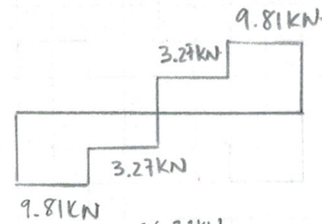
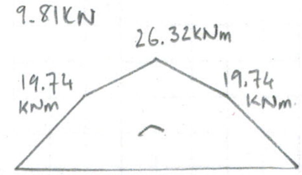
$M_{f max1} = 258$ kNm $M_{f max2} = 27$ kNm

$V_{f max1} = 96$ kN $V_{f max2} = 10$ kN

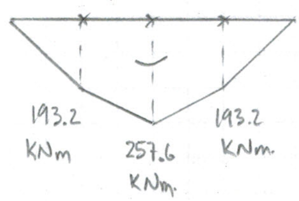
SFD







BMD



Try W360x51

$Z_x = 893\times10^3\ mm^3$ $I_y = 9.68\times10^6\ mm^4$ $d = 355$ mm $w = 7.2$ mm

$S_x = 796\times10^3\ mm^3$ $J = 237\times10^3\ mm^4$ $b = 171$ mm

$I_x = 141\times10^6\ mm^4$ $C_w = 285\times10^9\ mm^6$ $t = 11.6$ mm

Check section class — S16-14 Table 5-1

Flange : $\frac{b_{el}}{t} = 7.37 < \frac{145}{\sqrt{F_y}} = 7.81 \rightarrow$ class 1

Web : $\frac{h}{w} = 46.1 < \frac{1100}{\sqrt{F_y}} = 59.2 \rightarrow$ class 1

$\Rightarrow$ Section class 1

Check moment capacity — S16-14 §13.6(a)(i)

Case 1:

$M_{max} = 257.6$ (kNm)

$M_a = 209.3$ kNm $M_b = 225.4$ kNm $M_c = 241.5$ kNm

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$\omega_2 = 1.13 < 2.5$

$$M_u = \frac{\omega_2 \pi}{L_u}\sqrt{E I_y G J + \left(\frac{\pi E}{L_u}\right)^2 I_y C_w} = 974 \text{ (kNm)}$$

$M_p = Z_x F_y = 308 \text{ (kNm)}$

$M_u > 0.67 M_p$

$$\Rightarrow M_{r_1} = 1.15 \phi M_p \left[1 - \frac{0.28 M_p}{M_u}\right] = 290 \text{ (kNm)} > \phi M_p = 277 \text{ kNm}$$

$\Rightarrow M_{r_1} = 277 \text{(kNm)} > M_{f max1} = 258 \text{ (kNm)} \Rightarrow$ okay

Case 2: — S16-14 §13.6(a)(ii)

$M_{max} = 26.32$ kNm.

$M_a = M_c = 19.74$ kNm $\quad M_b = 26.32$ kNm.

$\omega_2 = 1.13 < 2.5$

$$M_u = \frac{\omega_2 \pi}{L_u}\sqrt{E I_y G J + \left(\frac{\pi E}{L_u}\right)^2 I_y C_w} = 100 \text{ (kNm)}$$

$M_p = Z_x F_y = 308 \text{ (kNm)}$

$M_u < 0.67 M_p$.

$\Rightarrow M_{r_2} = \phi M_u = 0.9(100)$

$\Rightarrow M_{r_2} = 90 \text{ (kNm)} > M_{f max2} = 27.2 \text{ (kNm)} \Rightarrow$ okay

Check shear capacity — S16-14 §13.4.1.1

$$\frac{h}{w} = 46.1 < \frac{1041}{\sqrt{F_y}} = 56.$$

$\Rightarrow F_s = 0.66 F_y$

$V_r = \phi A_w F_s = 0.9 (355)(7.2)(0.66 \times 345)$

$\Rightarrow V_r = 523 \text{ (kN)} > V_{f max1} = 96 \text{(kN)} \Rightarrow$ okay

$V_r = 523 \text{ (kN)} > V_{f max2} = 10 \text{ (kN)} \Rightarrow$ okay

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Check deflection

$$\Delta_{max} = \frac{Pa}{24EI_x}(3L^2 - 4a^2) + \frac{PL^3}{48EI_x}$$

$a = 2012.5\,mm$

$P = \max(P_L, P_S)$

$P = 16.8\ (kN)$

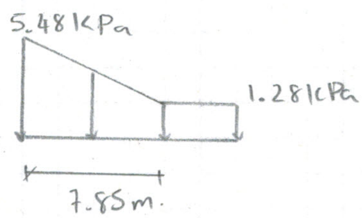
$\Delta_{max} = 15.38\ (mm)$

$\Delta_{max} = 15.38\ (mm) < \frac{L}{360} = 22.4\ (mm) \Rightarrow$ okay

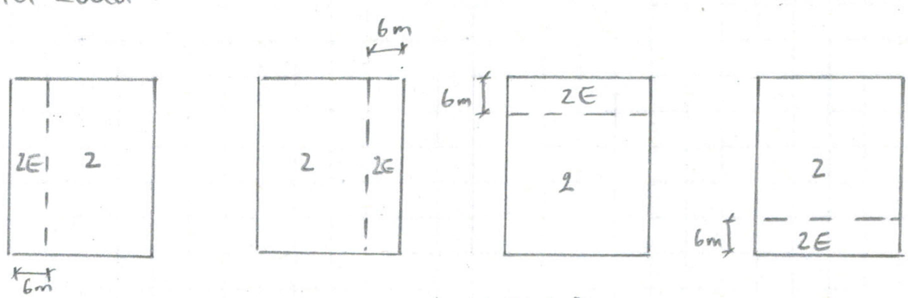
Roof 2:

DL = 1.12 KPa   LL = 1.0 kPa.

Snow Load:

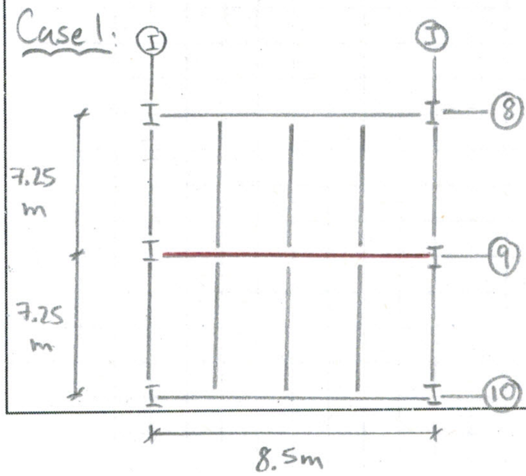


Wind Load:



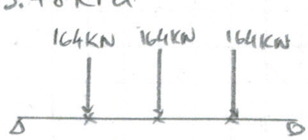
$P_2 = -0.676\ kPa$ $P_{2E} = -1.04\ kPa$.

Case 1:



For conservative, assume

SL = 5.48 kPa



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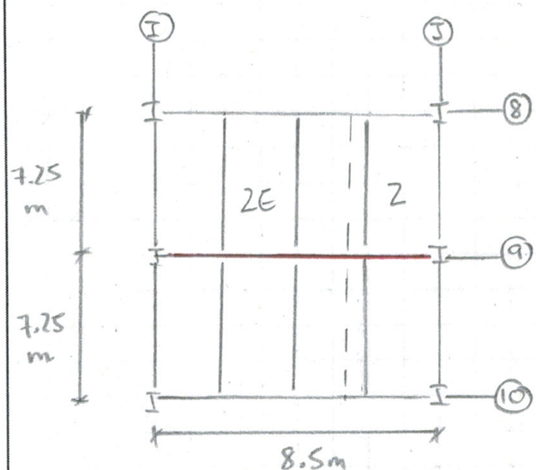
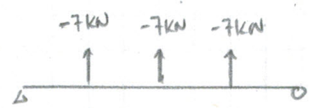
Critical case : 1.25DL + 1.5SL + 1.0 LL.

$P_{f_1} = 164$ (KN).

Case 2:





Critical case : 0.9 DL + 1.4 WL.

For conservative, assume all members lie in zone 2E

$\Rightarrow P_{f_2} = -7$ (KN)

$P_{f_1} = 164$ (KN)

$L_{u1} = 2.125$ m

$M_{fmax1} = 697$ (KNm).

$V_{fmax1} = 246$ (KN).

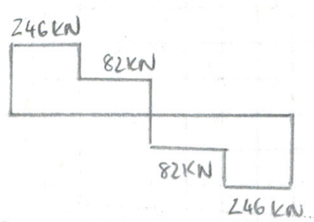
$P_{f_2} = -7$ (KN)

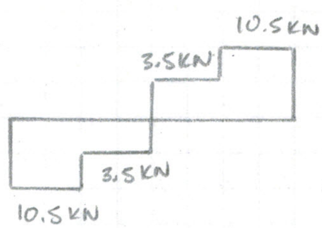
$L_{u2} = 8.5$ m.

$M_{fmax2} = 29.75$ (KNm)

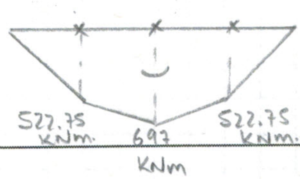
$V_{fmax2} = 11$ (KN)

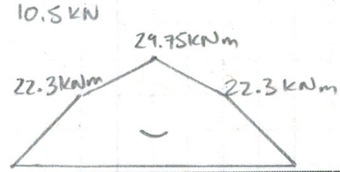
SFD





BMD





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<u>Try W530x92</u>

$Z_x = 2360 \times 10^3\ mm^3$ $I_y = 23.8 \times 10^6\ mm^4$ $d = 533\ mm$ $w = 10.2\ mm$

$S_x = 2070 \times 10^3\ mm^3$ $J = 762 \times 10^3\ mm^4$ $b = 209\ mm$

$I_x = 552 \times 10^6\ mm^4$ $C_w = 1590 \times 10^9\ mm^6$ $t = 15.6\ mm.$

Check section class: S16-14 Table 5-1

Flange: $\frac{b_{el}}{t} = 6.7 < \frac{145}{\sqrt{F_y}} = 7.81 \longrightarrow$ class 1

Web: $\frac{h}{w} = 49.2 < \frac{1100}{\sqrt{F_y}} = 59.2 \longrightarrow$ class 1

$\Rightarrow$ Section class 1

Check moment capacity

Case 1 S16-14 §13.6 (a)(i)

$M_{max} = 697\ kNm.$

$M_a = 570\ kNm.$ $M_b = 612.4\ kNm.$ $M_c = 655\ kNm.$

$\omega_2 = 1.13 < 2.5$

$$M_u = \frac{\omega_2 \pi}{L_u} \sqrt{E I_y G J + \left(\frac{\pi E}{L_u}\right)^2 I_y C_w} = 3166\ (kNm)$$

$M_p = Z_x F_y = 814.2\ (kNm)$

$M_u > 0.67 M_p$

$$\Rightarrow M_r = 1.15 \phi M_p \left[1 - \frac{0.28 M_p}{M_u}\right] = 782 (kNm) > \phi M_p = 732\ (kNm)$$

$\Rightarrow M_{r_1} = 732\ (kNm) > M_{f_{max1}} = 697\ (kNm) \Rightarrow$ okay

Case 2: S16-14 §13.6 (a)(ii)

$M_{max} = 30\ kNm.$

$M_a = M_c = 22.3\ kNm$ $M_b = 30\ kNm.$

$\omega_2 = 1.13 < 2.5$

$M_u = \frac{\omega_2 \pi}{L_u} \sqrt{E I_y G J + \left(\frac{\pi E}{L_u}\right)^2 I_y C_w} = 291.5\ (kNm)$

$M_p = Z_x F_y = 814.2\ (kNm)$

$M_u < 0.67 M_p$

$\Rightarrow M_{r_2} = \phi M_u = 0.9(291.5)$

$\Rightarrow M_{r2} = 262\ (kNm) > M_{f max2} = 30\ (kNm) \Rightarrow$ okay

Check shear capacity: S16-14 § 13.4.1.1

$\frac{h}{w} = 49.2 < \frac{1041}{\sqrt{F_y}} = 56$

$\Rightarrow F_s = 0.66 F_y$

$V_r = \phi A_w F_s = 0.9(533)(10.2)(0.66 \times 345)$

$\Rightarrow V_r = 1114\ (kN) > V_{f max1} = 246\ (kN) \Rightarrow$ okay

$V_r = 1114\ (kN) > V_{f max2} = 11\ (kN) \Rightarrow$ okay

Check deflection:

$\Delta_{max} = \frac{Pa}{24EI_x}(3L^2 - 4a^2) + \frac{PL^3}{48EI_x}$

$a = 2125\ mm$

$P = \max(P_L, P_S)$

$\Delta_{max} = 20.9\ (mm)$

$P = 76\ (kN)$

$\Delta_{max} = 20.9\ (mm) < \frac{L}{360} = 23.6\ (mm)$

Case 1

$\frac{M_{f1}}{M_{r1}} = \frac{697}{732} = 0.95$

$\frac{V_{f1}}{V_{r1}} = \frac{246}{1114} = 0.22$

Case 2

$\frac{M_{f2}}{M_{r2}} = \frac{30}{262} = 0.11$

$\frac{V_{f2}}{V_{r2}} = \frac{11}{1114} = 0.01$

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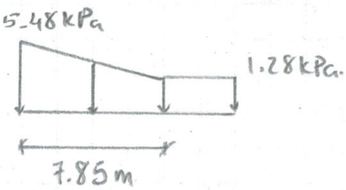
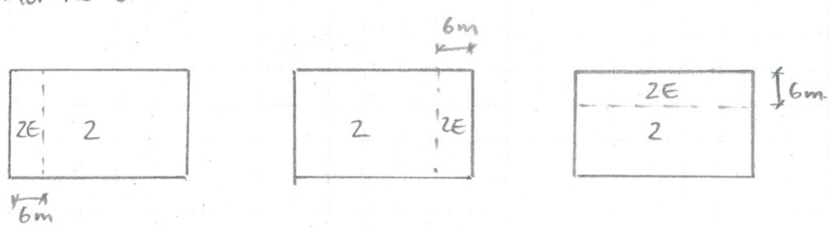
Roof 3:

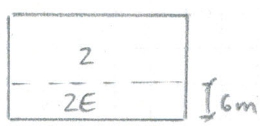
$DL = 1.12\ kPa$ $LL = 1.0\ kPa.$

Snow Load:



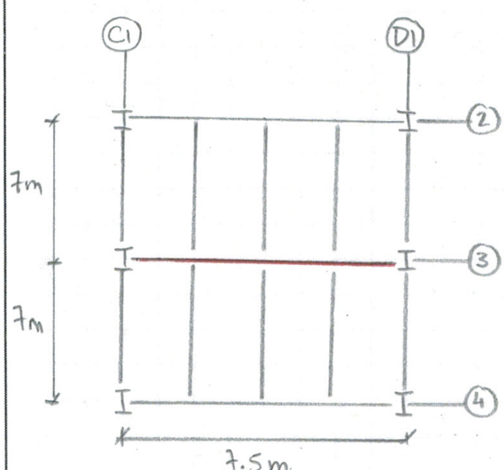
Wind Load:





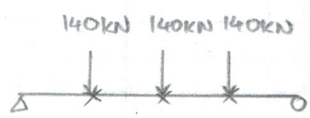
$P_2 = -0.676\ kPa$ $P_{2E} = -1.04\ kPa.$

Case 1:



For conservative, assume

$SL = 5.48\ kPa.$



Critical case: $1.25DL + 1.5SL + 1.0LL$

$P_{f1} = 140\ (kN)$

Case 2:

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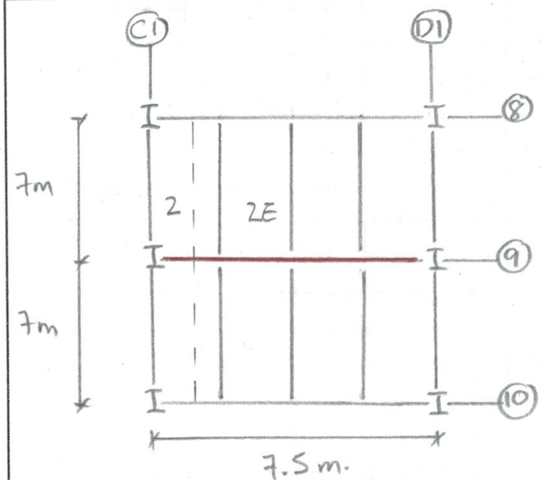
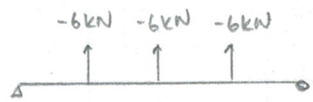




Critical case : 0.9DL + 1.4WL

For conservative, assume all members lie in zone 2E.

$\Rightarrow P_{f2} = -6$ (KN).

$P_{f1} = 140$ (kN)

$L_{u1} = 1.875$ m

$M_{fmax1} = 525.$ (kNm)

$V_{fmax1} = 210$ (kN)

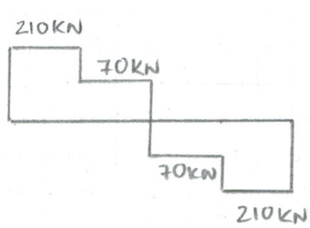
$P_{f2} = -6$ (kN)

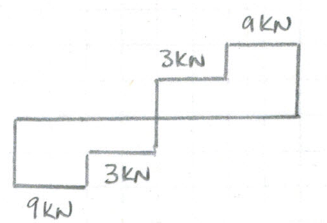
$L_{u2} = 7.5$ m

$M_{fmax2} = 23.$ (kNm)

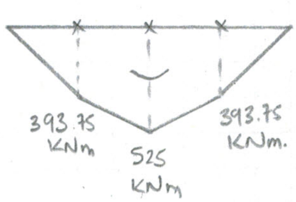
$V_{fmax2} = 9$ (kN)

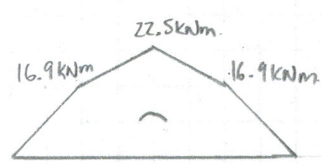
SFD





BMD





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Try W410x85

$Z_x = 1720 \times 10^3\ mm^3$ $I_y = 18 \times 10^6\ mm^4$ $d = 417\ mm$ $w = 10.9mm$

$S_x = 1510 \times 10^3\ mm^3$ $J = 924 \times 10^3\ mm^4$ $b = 181mm$

$I_x = 315 \times 10^6\ mm^4$ $C_w = 717 \times 10^9\ mm^6$ $t = 18.2mm$

Check section class — S16-14 Table 5-1

Flange: $\frac{b_{el}}{t} = 4.97 < \frac{145}{\sqrt{F_y}} = 7.81 \rightarrow$ class

Web: $\frac{h}{w} = 34.9 < \frac{1100}{\sqrt{F_y}} = 59.2 \longrightarrow$ class

$\Rightarrow$ section class

Check moment capacity:

Case 1 — S16-14 §13.6(a)(i)

$M_{max} = 525\ kNm$

$M_a = 426.6\ kNm$ $M_b = 459.3\ kNm$ $M_c = 492.2\ kNm$

$\omega_2 = 1.13 < 2.5$

$$M_u = \frac{\omega_2 \pi}{L_u}\sqrt{EI_y GJ + \left(\frac{\pi E}{L_u}\right)^2 I_y C_w} = 2474\ (kNm)$$

$M_p = Z_x F_y = 593\ (kNm)$

$M_u > 0.67\ M_p$

$$\Rightarrow M_{r_1} = 1.15\phi M_p\left[1 - \frac{0.28M_p}{M_u}\right] = 573\ (kNm) > \phi M_p = 534\ (kNm)$$

$\Rightarrow M_{r_1} = 534\ (kNm) > M_{f_{max1}} = 525\ (kNm) \rightarrow$ okay

Case 2: — S16-14 §13.6(a)(ii)

$M_{max} = 22.5\ kNm$

$M_a = M_c = 16.9\ kNm$ $M_b = 22.5\ kNm$

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$\omega_2 = 1.13 < 2.5$

$$M_u = \frac{\omega_2 \pi}{L_u}\sqrt{EI_y GJ + \left(\frac{\pi E}{L_u}\right)^2 I_y C_w} = 279 \text{ (kNm)}$$

$M_p = Z_x F_y = 593$ (kNm)

$M_u < 0.67 M_p$

$\Rightarrow M_{r2} = \phi M_u = 0.9(279)$

$\Rightarrow M_{r2} = 251$ (kNm) $> M_{f max2} = 23$ (kNm) $\Rightarrow$ okay

Check shear capacity — S16-14 §13.4.1.1

$\frac{h}{w} = 34.9 < \frac{1041}{\sqrt{F_y}} = 56.$

$\Rightarrow F_s = 0.66 F_y$

$V_r = \phi A_w F_s = 0.9(417)(10.9)(0.66 \times 345)$

$\Rightarrow V_r = 931$ (kN) $> V_{f max1} = 210$ (kN) $\Rightarrow$ okay

$V_r = 931$ (kN) $> V_{f max2} = 9$ (kN) $\Rightarrow$ okay

Check deflection

$$\Delta_{max} = \frac{Pa}{24EI_x}(3L^2 - 4a^2) + \frac{PL^3}{48EI_x}$$

$a = 1875$ mm.

$P = \max(P_L, P_S)$

$P = 62$ kN

$\Delta_{max} = 20.5$ (mm)

$\Delta_{max} = 20.5$ (mm) $< \frac{L}{360} = 20.8$ (mm) $\Rightarrow$ okay

Case 1	Case 2
$\frac{M_{f1}}{M_{r1}} = \frac{525}{534} = 0.98$	$\frac{M_{f2}}{M_{r2}} = \frac{23}{251} = 0.09$
$\frac{V_{f1}}{V_{r1}} = \frac{210}{930} = 0.23$	$\frac{V_{f2}}{V_{r2}} = \frac{9}{930} = 0.01$

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NBCC2015

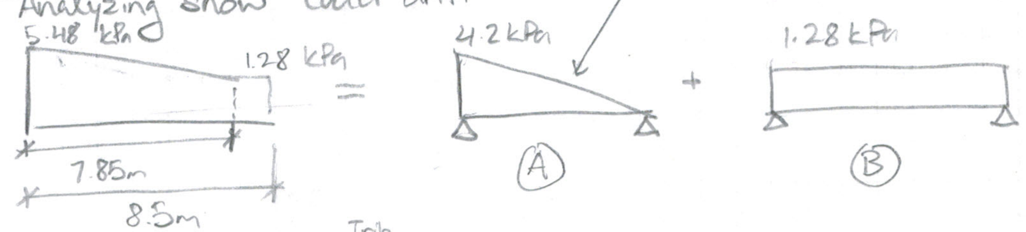
COLUMN LOADS

NOTE: This section is best read while referring to struct plans
Reason for finding loads on these columns is detailed in report.

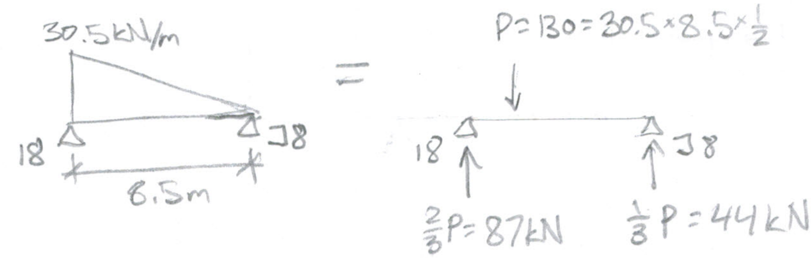
4TH STOREY COLUMNS (INTERIOR)

$\boxed{J8}$ TA = 7.25 × 8.5 = 61.7 m²

Analyzing snow load drift



(A) 4.2 kPa × 7.25 m = 30.5 kN/m (Trib width)



Load from typ snow load = 1.28 kPa × 61.7 m² = 79.0 kN

Loads by type on J8

C_S = 44 + 79 = 123 kN
C_D = 1.12 kPa × 61.7 m² = 69.2 kN
C_L = 1 kPa × 61.7 m² = 61.7 kN

C_f = 1.25D + 1.5S + 1.0 × L
= 1.25 × 69.2 + 1.5 × 123 + 61.7
= 333 kN

TABLE 4.1.3.2-A

→ Need to check factored load on 18 as Snow Drift is high

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$\boxed{18}$ Part of the TA is slanted so calculate roof slope, α

$\tan\alpha = \frac{2}{35}$ $\alpha = 3.3°$ $\frac{1}{\cos 3.3} = 1.002$ kPa → negligible increase

$TA = 7.25 \times 7.98 = 57.9 m^2$

Loads by type on 18

$C_S = \overset{DRIFT}{87} + 1.28 kPa \times 57.9 m^2 = 161.2$ kN
$C_D = 1.12 kPa \times 57.9 m^2 = 65.0$ kN
$C_L = 1 kPa \times 57.9 m^2 = 58.0$ kN

rounding more than compensates for 0.2% increase

$C_f = 1.25D + 1.5S + 1.0L$
$= 1.25 \times 65 + 1.5 \times 161.2 + 58 = 381.1$ kN

TABLE 4.1.3.2A

↳ this factored load is greater than J8's so it governs

Another critical column is D6 as it is almost 6m & has large TA

$\boxed{D6}$ $TA = 7.07 \times 7.15 = 50.6 m^2$

$C_D = 50.6 \times 1.12 \times \underset{\alpha\uparrow}{1.002} = 56.8$ kN
$C_S = 50.6 \times 1.28 \times 1.002 = 64.9$ kN
$C_L = 50.6 \times 1 \times 1.002 = 50.8$ kN

$C_F = 1.25 \times D + 1.5S + 1.0L$
$= 1.25 \times 56.8 + 1.5 \times 64.9 + 50.8$
$= 219.2$ kN

TABLE 4.1.3.2A

NOTE: Design will occur after. Load analysis first.

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4TH STOREY COLUMNS (EXTERIOR)

$\boxed{I4}$ KL = 4m but extra load due to drift
The snow drift analysis on p.1 showed that load was 87 kN for columns on grid I, with tributary width = 7.25m

↳ Tributary Width is $\frac{7.15m}{2} = 3.58m$

Snow load due to drift $= 87 \times \frac{3.58}{7.25} = 42.9\,kN$

$TA = 3.58m \times \frac{15.96m}{2} = 28.57\,m^2$

Loads by type on I4

$C_S = 28.57m^2 \times 1.28\,kPa + 42.9\,kN = 80.0\,kN$
$C_D = 28.57m^2 \times 1.12\,kPa = 32.0\,kN$
$C_L = 28.57m^2 \times 1.0\,kPa = 28.6\,kN$

$C_f = 1.25D + 1.5S + 1.0L$ — TABLE 4.1.3.2A
$= 1.25 \times 32 + 1.5 \times 36.6 + 28.6$
$= 123.5\,kN$

$\boxed{D4}$ D4 is 6m tall and has a large tributary area

$TA = \frac{15.15}{2} \times \frac{7.15}{2} = 27.1\,m^2$

Loads by type on D4

$C_S = 27.1 \times 1.28 = 34.7\,kN$
$C_D = 27.1 \times 1.12 = 30.4\,kN$
$C_L = 27.1 \times 1.0 = 27.1\,kN$

$C_f = 1.25D + 1.5S + 1.0L$ — TABLE 4.1.3.2A
$= 1.25 \times 30.4 + 1.5 \times 34.7 + 27.1$
$= 117.2\,kN$

$\boxed{J4}$ $TA = 8.5 \times \frac{7.15}{2} = 30.4\,m^2$
Loads by type
$C_S = 44/2 + 1.28 \times 30.4 = 61.0\,kN$
$C_D = 1.12 \times 30.4 = 34.1\,kN$
$C_L = 1 \times 30.4 = 30.4\,kN$
} to be used on lower levels

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3RD STOREY COLUMNS (INTERIOR)

For floors: D = 4.06 kPa   L = 4.8 kPa (non-assembly)

LLRF = $0.3 + \sqrt{9.8/B}$ 4.1.5.8 (3)

$\boxed{J8}$ From level above C_S = 123 kN C_D = 69.2 C_L = 61.7 kN
TA = 61.7 m²

This level TA = 61.7 m²

Reduceable TA = 61.7 m²

LLRF = $0.3 + \sqrt{9.8/61.7}$ = 0.70   4.1.5.8(3)

Loads by type on J8

C_S = 123 kN
C_D = 69.2 + 61.7 × 4.06 = 319.7 kN
C_L = 61.7 + (61.7 × 4.8) × 0.70 = 269 kN

C_f = 1.25 × D + 1.5 L + 1.0 S   TABLE 4.1.3.2A
= 1.25 × 319.7 + 1.5 × 269 + 123
= 926.1 kN

$\boxed{I8}$ From level above C_S = 161.2 kN C_D = 65 kN C_L = 58 kN
TA = 57.9 m²

This level TA = 57.9 m²
Reduceable TA = 57.9 m²

LLRF = $0.3 + \sqrt{9.8/57.9}$ = 0.72   4.1.5.8(3)

Loads by type on I8

C_S = 161.2 kN
C_D = 65 kN + 57.9 × 4.06 = 300.1 kN
C_L = 58 kN + (57.9 × 4.8) × 0.72 = 258.1 kN

C_f = 1.25 D + 1.5 L + 1.0 S   TABLE 4.1.3.2A
= 1.25 × 300.1 + 1.5 × 258.1 + 161.2
= 923.5 kN

J8 loading governs representing the live load from the extra tributary area for J8 overcame the extra snow load that I8 carried

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3RD STOREY COLUMNS (EXTERIOR)

$\boxed{D4}$ Finding loads on this to design 2nd storey columns

From floor above: $C_S = 34.7kN$ $C_D = 30.4kN$ $C_L = 27.1kN$

Reduceable TA = $27.1m^2$

$LLRF = 0.3 + \sqrt{9.8/27.1} = 0.91$ §4.1.5.8(3)

Dead Load from curtain wall = 7.2 kN/m ← for 6m curtain wall

$C_D = 30.4 + 27.1 \times 4.06 + \frac{15.15m}{2} \times 7.2kN/m = 195.0kN$

$C_S = 34.7kN$

$C_L = (27.1 + (27.1m^2 \times 4.8kPa) \times 0.91 = 145.5kN$

$C_f = 1.25 \times 195 + 1.5 \times 145.5 + 34.7 = 496.7kN$ TABLE 4.1.3.2A

$\boxed{14}$ Largest snow load due to drift

From floor above: $C_S = 80.0kN$ $C_D = 32kN$ $C_L = 28.6kN$

Reduceable TA = $28.57m^2$

$LLRF = 0.3 + \sqrt{9.8/28.57} = 0.89$ §4.1.5.8(3)

Loads by type on 14

$C_S = 80.0kN$

$C_D = 32kN + 4.06kPa \times 28.57m^2 + \frac{15.96m}{2} \times 7.2kN/m = 205.5kN$

$C_L = 28.6kN + (4.8kPa \times 28.57)0.89 = 150.7kN$

$C_f = 1.25D + 1.5L + 1.0S$ TABLE 4.1.3.2A

$= 1.25 \times 205.5 + 1.5 \times 150.7 + 80$

$= 563 kN$

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$\boxed{J4}$ Largest TA on exterior

From floor above: $C_S = 61\,kN$, $C_D = 34.1\,kN$ $C_L = 30.4\,kN$ — NBCC 2015

Reduceable $TA = 30.4\,m^2$

$LLRF = 0.3 + \sqrt{9.8/30.4} = 0.87$ — 4.1.5.8(3)

Loads by type on J4

$C_S = 61\,kN$

$C_D = 34.1 + 30.4 \times 4.06 + 8.5m \times 7.2kN/m = 218.8\,kN$

$C_L = (30.4 + (30.4 \times 4.8) \times 0.87 = 157.4\,kN$

$C_f = 1.25D + 1.5L + 1.0S$ — TABLE 4.1.3.2A

$= 1.25 \times 218.8 + 1.5 \times 157.4 + 61$

$= 570.6\,kN$

↳ J4 governs ext. columns on 3rd storey

NBCC 2015

2ND STOREY COLUMNS (INTERIOR)

$\boxed{J8}$ Column here governed last time so it will govern again
From level above: $C_S = 123\ kN$ $C_D = 319.7\ kN$

Reduceable $TA = 2 \times 61.7 = 123.4\ m^2$

$LLRF = 0.3 + \sqrt{9.8/123.4} = 0.59$ — 4.1.5.8(3)

Loads by type on J8

$C_S = 123\ kN$
$C_D = 319.7 + 61.7 \times 4.06 = 570.3\ kN$
$C_L = 61.7 + 2(61.7 \times 4.8 \times 2)0.59 = 411.2\ kN$

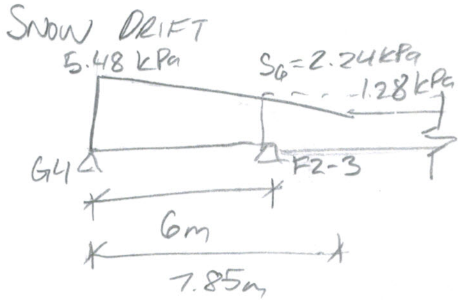
$Cf = 1.25D + 1.5L + 1.0S$ — TABLE 4.1.3.2A
$= 1.25 \times 570.3 + 1.5 \times 411.2 + 123$
$= 1452.7\ kN$

$\boxed{G4}$ Column inbetween farm and market
→ Using D4 loads to find G4 loads → Using trib width (TW)
$TW_{E4}/TW_{D4} = \frac{14.4m}{15.15m} = 0.94$

↳ Loads would be similar: use D4 loads from floor above
$C_S = 34.7\ kN$ $C_D = 195.0\ kN$

Loads from market roof
$D = 1.12\ kPa$ $L = 1\ kPa$ $S = 1.28\ kPa$ typ

SNOW DRIFT



$S_C = (5.48 - 1.28) \times \frac{1.85}{7.85} + 1.28$
$= 2.24\ kPa$

$S_{DRIFT}^{G4} = (5.48 - 2.24) \times \frac{6m}{2} \times \frac{14.4m}{2} \times \frac{1}{2} \times \frac{2}{3} = 23.4\ kN$

ON F2-3 load will be overestimated as 2.24 kPa to the right to facilitate a conservative estimate

$S_{DRIFT}^{F2\text{-}3} = (5.48 - 2.24) \times \frac{6m}{2} \times 7.5m \times \frac{1}{2} \times \frac{1}{3} = 12.2\ kN$

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$\boxed{G4}$ CONT'D

Loads by type on G4

$C_S = 34.7 + 23.4 + 2.24\,kPa \times \frac{14.4m}{2} \times 6m \times \frac{1}{2} = 106.5\,kN$

$C_D = 195\,kN + 1.12\,kPa \times \frac{14.4m}{2} \times \frac{6m}{2} + 4.08\,kPa \times \frac{14.4m}{2} \times \frac{7.15m}{2}$

$+ 4.8\,kN/m \times \frac{14.4m}{2}$

$= 358.8\,kN$

Reduceable $TA = 2 \times \frac{14.4}{2} \times \frac{7.15}{2} = 51.5m^2$

$LLRF = 0.3 + \sqrt{9.8/51.5} = 0.74$ 4.1.5.8(3)

$C_L = \left(27.1 + 1kPa \times \frac{14.4}{2} \times \frac{6}{2}\right) + \left(4.8\,kPa \times \frac{14.4}{2} \times \frac{7.15}{2} \times 2\right) \times 0.74$

$= 231.6\,kN$

$C_f = 1.25D + 1.5L + 1.0S$ TABLE 4.1.3.2A

$= 1.25 \times 358.8 + 1.5 \times 231.6 + 106.5$

$= 902.4\,kN$

$\boxed{E2\text{-}3}$ Interior market column

→ Using $S = 2.24\,kPa$ as seen from prev analysis

$TA = 7m \times 7.5m = 52.5m^2$

$C_S = 2.24 \times 52.5 = 117.6\,kN$

$C_D = 1.12 \times 52.5 = 58.8\,kN$

$C_L = 1 \times 52.5 = 52.5\,kN$

$C_f = 1.25D + 1.5S + 1.0L$ TABLE 4.1.3.2A

$= 1.25 \times 58.8 + 1.5 \times 117.6 + 52.5$

$= 302.4\,kN$

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2ND STOREY COLUMNS (EXTERIOR)

$\boxed{J4}$ Largest TA on exterior

From Floor above: $C_S = 61\,kN$ $C_D = 218.8\,kN$

Reduceable TA $= 30.4 \times 2 = 60.8$

$LLRF = 0.3 + \sqrt{9.8/60.8} = 0.71$ 4.1.5.8(3)

Loads by type on J4

$C_S = 61\,kN$
$C_D = 218.8 + 4.06 \times 30.4 + 4.8\,kN/m \times 8.5m = 383.1\,kN$
$C_L = 30.4 + (30.4 \times 4.8 \times 2) \times 0.71 = 237.7\,kN$

$C_f = 1.25D + 1.5L + 1.0S$ TABLE 4.1.3.2A
$= 1.25 \times 383.1 + 1.5 \times 237.7 + 61$
$= 896.5\,kN$

$\boxed{F2\text{-}1}$ Largest TA on exterior for Market

Total TA $= 7.5 \times 7 \times \frac{1}{2} = 26.25\,m^2$

Loads by type on F2-1

$C_S = 1.28 \times 26.25 = 33.6\,kN$
$C_D = 1.12 \times 26.25 = 29.4\,kN$
$C_L = 1 \times 26.25 = 26.25\,kN$

$C_f = 1.25D + 1.5S + 1.0L$ TABLE 4.1.3.2A
$= 1.25 \times 29.4 + 1.5 \times 33.6 + 26.3$
$= 113.5\,kN$

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1ST STOREY COLUMNS (INTERIOR)

$\boxed{J8}$ From level above: $C_S = 123\,kN$ $C_D = 570.3\,kN$

Reduceable $TA = 61.7 \times 3 = 185.1\,m^2$

$LLRF = 0.3 + \sqrt{9.8/185.1} = 0.53$ 4.1.5.8(3)

Loads by type on J8

$C_S = 123\,kN$
$C_D = 570.3 + 61.7 \times 4.06 = 820.9\,kN$
$C_L = 61.7 + (61.7 \times 4.8 \times 3) \times 0.53 = 532.6\,kN$

$C_f = 1.25D + 1.5L + 1.0S$ TABLE 4.1.3.2A
$= 1.25 \times 820.9 + 1.5 \times 532.6 + 123$
$= 1948\,kN$

$\boxed{G4}$ From level above: $C_S = 106.5\,kN$ $C_D = 358.8\,kN$
* NOTE Live Load from restaurant is assembly occupancy
$TA = \frac{14.4}{2} \times \frac{6}{2} = 21.6\,m^2 < 80$ no LLRF 4.1.5.8(2)

Reduceable $TA = \frac{14.4}{2} \times \frac{7.15}{2} \times 3 = 77.22\,m^2$
$LLRF = 0.3 + \sqrt{9.8/77.22} = 0.66$
$C_S = 106.5\,kN$
$C_D = 358.8\,kN + \frac{14.4}{2} \times \frac{13.15}{2} \times 4.06\,kPa = 551\,kN$
$C_L = (27.1 + 1\,kPa \times \frac{14.4}{2} \times \frac{6}{2}) + (4.8\,kPa \times \frac{14.4}{2} \times \frac{7.15}{2} \times 3) \times 0.66 + 4.8 \times \frac{14.4}{2} \times \frac{6}{2}$

$= 397\,kN$

$C_f = 1.25D + 1.5L + 1.0S$ TABLE 4.1.3.2A
$= 1.25 \times 551 + 1.5 \times 397 + 106.5$
$= 1391\,kN$

$\boxed{F2\text{-}3}$ Interior market column. From previous level: $C_S = 117.6\,kN$ $C_D = 58.8\,kN$ $C_L = 52.5\,kN$
$\rightarrow TA = 52.5 < 80 \rightarrow$ no LLRF
$C_S = 117.6\,kN$
$C_D = 58.8 + 4.06 \times 7 \times 7.5 = 272.0\,kN$
$C_L = 52.5 + 4.8 \times 52.5 = 304.5\,kN$

$C_f = 1.25D + 1.5L + 1.0S$ TABLE 4.1.3.2A
$= 1.25 \times 272 + 1.5 \times 304.5 + 117.6$
$= 914.4\,kN$

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1ST STOREY COLUMNS (EXTERIOR)

$\boxed{J4}$ From level above: $C_S = 61$ kN $C_D = 383.1$ kN

Reduceable $TA = 30.4 \times 3 = 91.2\ m^2$

$LLRF = 0.3 + \sqrt{9.8/91.2} = 0.63$ 4.1.5.8(3)

Loads by type on J4

$C_S = 61$ kN
$C_D = 383.1 + 4.06 \times 30.4 + 4.8 \times 8.5 = 547.4$ kN
$C_L = 30.4 + (30.4 \times 4.8 \times 3) \times 0.63 = 306.2$ kN

$C_f = 1.25D + 1.5L + 1.0S$ TABLE 4.1.3.2A
$= 1.25 \times 547.4 + 1.5 \times 306.2 + 61$
$= 1205$ kN

$\boxed{F2\text{-}1}$ From level above: $C_S = 33.6$ kN $C_D = 29.4$ kN $C_L = 26.25$ kN
No LLRF as $TA < 80$
$TA = 26.25$

Loads by type on F2-1

$C_S = 33.6$ kN
$C_D = 29.4 + 4.06 \times 26.25 = 136$ kN
$C_L = 26.25 + 26.25 \times 4.8 = 153$ kN

$C_f = 1.25D + 1.5L + 1.0S$ TABLE 4.1.3.2A
$= 1.25 \times 136 + 1.5 \times 153 + 33.6$
$= 433.1$ kN

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CSA S16-14

COLUMN DESIGN

*NOTE: Some sections overdesigned to accomodate connection from joists

Summary of Design is seen below

LVL	COL TAG	INT/EXT	(kN) C_S	(kN) C_D	(kN) C_L	(kN) C_f	(kN) C_r	Chosen Section	C_f/C_r
4	J8	INT	123	70	62	333	-	-	-
	I8	INT	162	65	58	382	735	W200x42	0.52
	D6	INT	65	57	51	220	395	W200x42	0.56
4	I4	EXT	80	32	29	124	395	W200x42	0.32
	D4	EXT	35	31	28	118	-	-	-
3	J8	INT	123	320	269	927	1210	W200x52	0.77
	I8	INT	162	301	259	924	-	-	-
3	D4	EXT	35	195	146	497	-	-	-
	I4	EXT	80	206	151	563	-	-	-
	J4	EXT	61	219	158	571	735	W200x42	0.78
2	J8	INT	123	571	412	1453	1680	W200x71	0.87
	G4	INT	107	357	232	903	-	-	-
	F2-3	INT	118	59	53	303	735	W200x42	0.42
2	J4	EXT	61	384	238	897	1210	W200x52	0.75
	F2-1	EXT	34	30	27	114	735	W200x42	0.16
1	J8	INT	123	821	533	1948	2060	W200x86	0.95
	G4	INT	107	551	397	1391	-	-	-
	F2-3	INT	118	272	305	915	1210	W200x52	0.76
1	J4	EXT	61	384	307	1205	1210	W200x52	0.99
	F2-1	EXT	34	136	153	434	735	W200X42	0.60

NOTE: Sections were picked keeping in mind ease of construction, cost of having too many varying sections and most importantly safety.

↳ Some sections are marked "–" meaning that the C_f showed that the column loading didn't govern and the design will thus be based off a more heavily loaded column.

→ Sections were picked using CSA S16-14 pp. 4-26 to 4-27 which is the column design aid. CSA S16-14 pp4-26 to 27

↳ NOTE: All sections pass local buckling test for web and flange as seen on these pages as well. §13.3.1 Table 1 pp. 1-161

↳ NOTE: All members pass slenderness check $\frac{KL}{r} \leq 200$ §10.4.2.1

Ex: with longest member (6m) with the smallest section

$\frac{KL}{r} = \frac{1 \times 6000}{38.3} = 157 < 200$ ✓ok

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LATERAL LOADS ON BRACING & COLUMNS

- Lateral loads consist of Notional Loads & Wind Loads

* Assuming Tension only bracing.

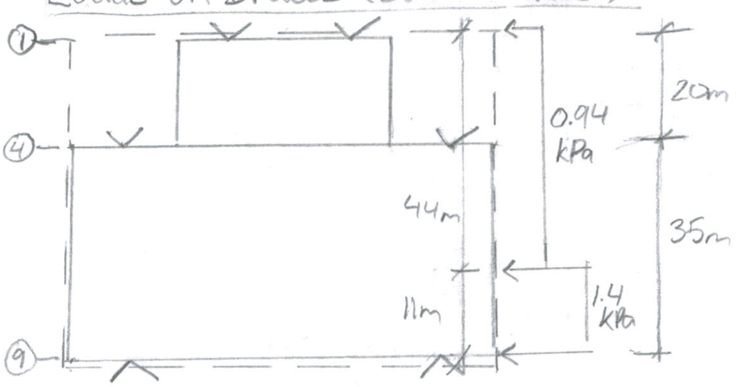
NOTIONAL LOAD SUMMARY

Level	Notional DL (kN)	Notional LL (kN)	Notional SL (kN)	Greatest Factored Load 1.25D + 1.5S + 1.0L or 1.25D + 1.5L + 1.0S (kN)
Roof	16.46	13.13	16.8	59.0
4	58.57	63	-	167.8
3	56.81	67.69	9.09	181.64
2	77.24	85.5	-	224.8

} Factored Notional Load per each level

TABLE 4.1.3-2A

Loads on Braces (East to West)



M_D about gridline 2 $= 11m \times (1.4kPa) \times 4m \times (55 - \frac{11}{2}m) + 44m \times (0.94kPa) \times 4m \times \frac{44m}{2}$

(for 4m height) $= 6648.5\,kNm$

↳ M_D is driving moment from forces

To calculate M_r there are two unknowns from the braces at grid 4 and 9. The moment that the braces resist will be proportioned by the braces distance from gridline ①.

→ Let the lateral force resisted by braces at ⑨ be called "F".

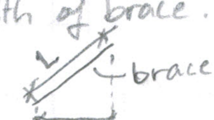
$\frac{20}{55} \times F \times 20 + 55F = M \qquad 62.27F = M$

$F = \frac{6648.5\,kNm}{62.27m} = 106.8\,kN$

→ Let T_L represent tension in left brace and T_r for right brace

→ The resisting tension is based off the width of brace.

$T_L \frac{W_L}{L_L} + T_L \frac{W_R}{W_L} \times \frac{W_R}{L_R} = F$ as $T_R = T_L \frac{W_R}{W_L}$



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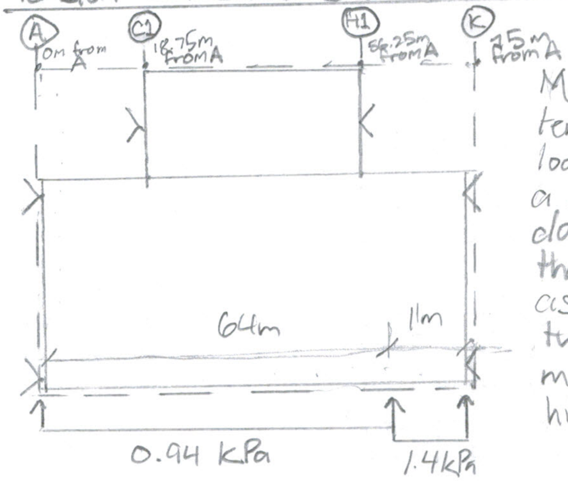
Tension on Braces (North to South)



Method of analysis is still by tension only bracing. To distribute load amongst 4 lines of braces a moment about gridline (A) will be done. It is important to note that the fourth floor diaphragm as well as th roof only has two lines of braces as the marketplace only extends 2 storeys high.

Driving M about A $= 1m \times 1.4kPa \times 4m \times (75m - \frac{1m}{2}) + 64m \times (0.94kPa) \times 4m \times (\frac{64m}{2}) = 11900\,kNm$
↳ this is for a 4m tributary height

Fdiaphragm for 2nd and 3rd flr
Solve: $F/2 \times 18.75/75 \times 18.75 + F/2 \times 56.25/75 \times 56.25 + 75F = M$
↳ Fdiaphragm for 2nd and 3rd flr = 121 kN ← at grid (K) braces

Fdiaphragm for 4th flr and rf
Solve $75F = M$
↳ Fdiaphragm for 4th flr and rf = 159 kN ← at grid (K) braces

NOTE: The bays for the braces are very similar (100mm difference) and thus loads when analyzing braces on the same grid will share the diaphragm load equally.

Ex. calc for the roof:
↳ Tributary height is 3m instead of 4m.

Diaphragm force at grid (K) is $159kN \times 3/4 = 119\,kN$

Tension in either brace (upper or lower)
↳ chevron brace width = 3625mm
height = 6000 mm ← designed for longest.
length of brace = 7010 mm

↳ $\frac{119kN}{2} \times \frac{7010}{3625} = 115.1\,kN$

NOTE: notional loads will be split among the 4 lines of bracing (not just 3 which was done on E→W wind)
↳ notional tension found in similar manner as above

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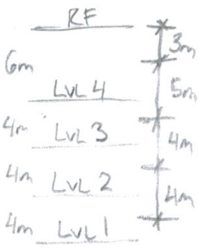
Left Brace Dims
$H_L = 4000mm$
$W_L = 3475mm$
$L_L = 5299mm$
$(L_L)_{6m} = 6934mm$

Right Brace Dims
$H_R = 4000mm$
$W_R = 4250mm$
$L_R = 5836mm$
$(L_R)_{6m} = 7353mm$ ← these values are for top storey where $H = 6m$



* NOTE: the moment previously calculated was based off a tributary height of 4m. For other tributary heights, the moment previously calculated can be adjusted with the appropriate ratio.

• Ex of Finding Tension in Left and Right Brace
Roof notional load = 59/3 = 19.7
$3/4 F = 3/4 \times 106.8 = 80.1kN$ ← diaphragm force at (9) from wind

Using $T_L \frac{W_L}{L_L} + T_L \frac{W_R}{W_L} \times \frac{W_R}{L_R} = F+N$ to find T_L

$$T_L\left(\frac{3475}{6934} + \frac{4250}{3475} \times \frac{4250}{7353}\right) = 80.1$$

$$T_L = 66.3kN$$

$$T_R = T_L \frac{W_R}{W_L} = 66.3 \times \frac{4250}{3475} = 81.1kN$$

Now to do this for all other braces using accumulated lateral forces

LVL	Wind Load at (9) (kN)	Total W Load	Total N Load at (9) (kN)	W Load in Left Brace	W Load in Right Brace	N Load in Left Brace	N Load in Right Brace
RF	80.1	80.1	19.7	66.3	81.1	16.3	19.9
4	133.5	213.6	75.6	138.1	168.9	48.9	59.8
3	106.8	320.3	136.2	207.2	253.3	88.1	107.7
2	106.8	427.1	211.1	276.2	337.8	136.5	167

· All loads in table above are in (kN) and loads in braces are tension forces
· Total Notional and Wind Loads represent an accumulation of these loads as you progress down the building.

*Checks were done to show that factored gravity load combos govern column design with braces as opposed to tension in lateral load combos

↳ Ex: Ext. 3rd storey columns designed for $D = 219kN \rightarrow 0.9D = 197kN > 133kN$ TABLE 4.1.3.2D
Here tension on rf level braces connecting to 3rd storey cols $= 1.4 \times 81.1 + 19.9$
$= 133kN$ (at most)
↳ this value is even smaller when finding T on cols.

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LVL	Wind Load at ⓚ (kN)	Total W Load	Notional Load at ⓚ (kN)	Total N Load	Wind load in either brace	Notional Load in either brace
RF	119	119	15	15	116	14.3
4	199	318	42	57	237	42.2
3	121	439	46	103	327	76.1
2	121	560	57	160	417	117.9

- All loads in table above are in kN and loads and braces are tension forces
- Total Notional and Wind Loads represent an accumulation of these loads as you ~~progress~~ down the building

* Again, calculating $1.4W+0.9D$ for these columns still results in compression and thus these columns only need to be designed for the factored compressive loads. TABLE 4.1.3.2A

Design Load Summary

LVL	Braces running E-W Factored Tension: $1.4W+N$ (kN)	Braces running N-S Factored Tension (kN)	
RF	133.5	176.7	ACTUAL DESIGN LOADS
4	296.3	373.1	
3	462.4	532.9	
2	639.8	700.8	

↳ Ex: From braces running N-S at roof level TABLE 4.1.3.2A

$1.4W+N = 1.4 \times 116 + 14.3 = 176.7$ kN

I will multiply these loads by 1.4 due to net section fracture which may occur during connection design.

Loads used to find appropriate section

LVL	Braces running E-W $1.4 \times T_f$ (kN)	Braces running N-S $1.4 \times T_f$ (kN)
RF	187	248
4	415	523
3	648	747
2	896	982

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Design Summary for Braces Running East to West

LVL	Chosen Section	(kN) Tf	(kN) Tr	Ag of Section (mm²)	Required Ag (mm²)	Tf/Tr
Rf	C150×12	134	482	1550	603	0.28
4	C150×12	297	482	1550	1340	0.62
3	C180×18	463	721	2320	2087	0.65
2	C200×28	640	111	3550	2886	0.59

Design Summary for Braces Running North to South

LVL	Chosen Section	(kN) Tf	(kN) Tr	Ag of Section (mm²)	Required Ag (mm²)	Tf/Tr
Rf	C150×12	177	482	1550	799	0.37
4	C180×15	374	574	1850	1684	0.65
3	C200×21	533	810	2616	2406	0.66
2	C200×28	701	110	3550	3163	0.64

NOTE: Req'd Ag is based off of a load of 1.4×Tf which is done to accomodate net section fracture

Ex of Req'd Ag calculation using E-W brace on LVL-Rf

↳ $T_r = \phi A_g F_y \qquad F_y = 345\,MPa \quad \phi = 0.9$ S16-14 9.13.2

$1.4\,T_f = T_r$

$A_g = \frac{T_f(1.4)}{\phi F_y}$

$= \frac{134(1.4)\times 1000}{0.9\times 345}$

$= 603\ mm^2$

NOTE: Sections were chosen by referring to S16-14, C-channel info S16-14 pp6-62

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Floor slab:

DL = 4.06 kPa LL = 4.8 kPa

Critical load combination:

$1.25DL + 1.5LL = 1.25(4.06) + 1.5(4.8)$

$= 13 (kPa)$

According to CANAM steel deck catalogue (Canada)

- Choose slab P-3615

Slab thick = 90 mm span = max span of floor

Deck thick = 1.12 mm = 1812.5 mm.

⇒ Resistance capacity = 15.99 kPa > 13 kPa ⇒ okay.

Check deflection:

$\Delta_{max} = \frac{5wL^4}{384 E_c I_{comp}}$ $E_c = 203 \times 10^3$ MPa.

$I_{comp} = 5.36 \times 10^6 \ mm^4$

$= 0.62 \ (mm)$

$w = 4.8$ kPa.

$\Delta_{max} = 0.62 \ mm < \frac{L}{360} = \frac{1812.5}{360} = 5.0 \ mm$ ⇒ okay

Steel deck

Roof 1:

DL = 1.12 kPa LL = 1.0 kPa SL = 1.28 kPa

Critical Load combination:

$1.25DL + 1.5SL + 1.0LL = 1.25(1.12) + 1.5(1.28) + 1.0(1.0)$

$= 4.32 \ (kPa)$

According to CANAM Steel deck catalogue (Canada)

- Choose deck P-2436 Type 22

Deck thick = 0.76 mm

Span = max span of roof 1 = 1812.5 mm

$\Rightarrow$ Resistance capacity = 8.94 kPa > 4.32 kPa $\Rightarrow$ okay

(for 2100 mm span)

Check deflection:

$\Delta_{max} = \frac{5wL^4}{384EI}$ $\quad$ $I = 1.006 \times 10^6 \ mm^4$

$= 0.8 (mm)$ $\quad$ $w = 1.152$ kPa (service snow load)

$\Delta_{max} = 0.8 (mm) < \frac{L}{360} = \frac{1812.5}{360} = 5 (mm) \Rightarrow$ okay

Roof 2:

DL = 1.12 kPa $\quad$ LL = 1.0 kPa $\quad$ SL = 5.48 kPa.

Critical Load combination.

1.25 DL + 1.5 SL + 1.0 LL = 10.62 (kPa).

- Choose deck P-2436 type 18

Deck thick = 1.21 mm

Span = max span of roof 2 = 2125 mm.

$\Rightarrow$ Resistance capacity = 13.19 kPa > 10.62 kPa $\Rightarrow$ okay

Check deflection

$\Delta_{max} = \frac{5wL^4}{384EI} = 5.0 (mm)$. $\quad$ $w = 3.8$ kPa (service snow load)

$\Delta_{max} = 5.0 \ mm < \frac{L}{360} = \frac{2125}{360} = 5.9 \ mm \Rightarrow$ okay

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Roof 3:

DL = 1.12 kPa. LL = 1 kPa SL = 5.48 kPa.

Critical Load Combination

1.25 DL + 1.5 SL + 1.0 LL = 10.62 (kPa).

- Choose deck P-2436 type 18

Deck thick = 1.21 mm

Span = max span of roof 3 = 1875 mm

⇒ Resistance capacity = 13.19 kPa > 10.62 kPa ⇒ okay

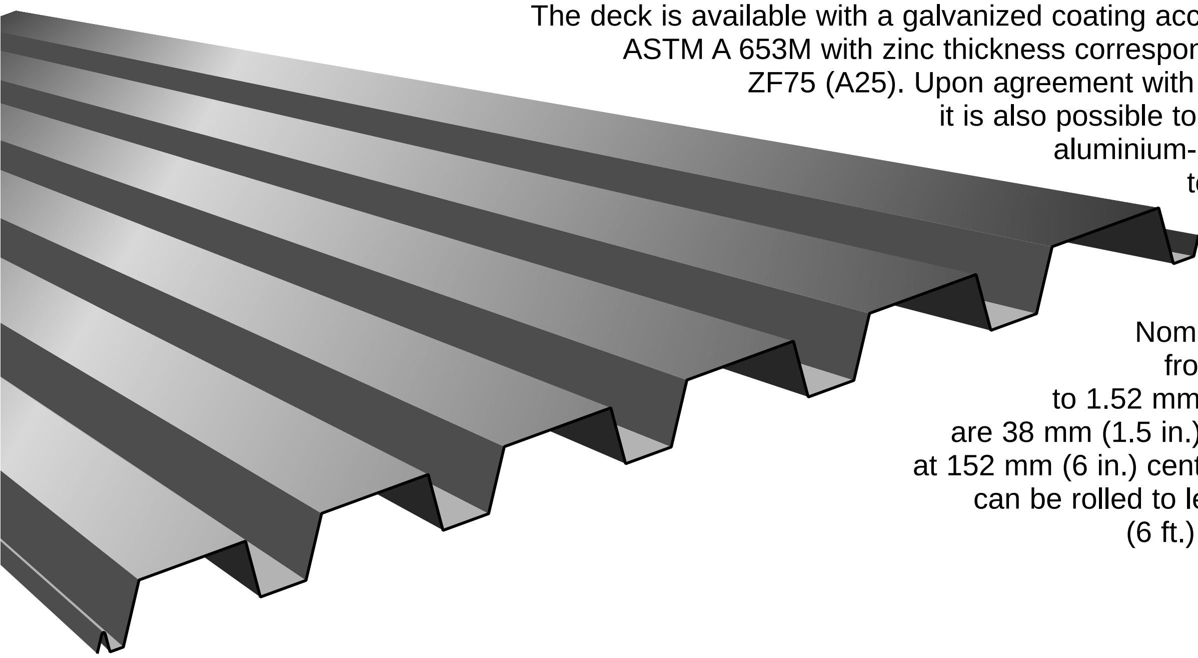
Check deflection

$\Delta_{max} = \frac{5wL^4}{384EI} = 5.0 (mm)$ $w = 3.8$ kPa.

$\Delta_{max} = 5.0$ mm $< \frac{L}{360} = \frac{1875}{360} = 5.2$ mm ⇒ okay

P-3615 & P-3606

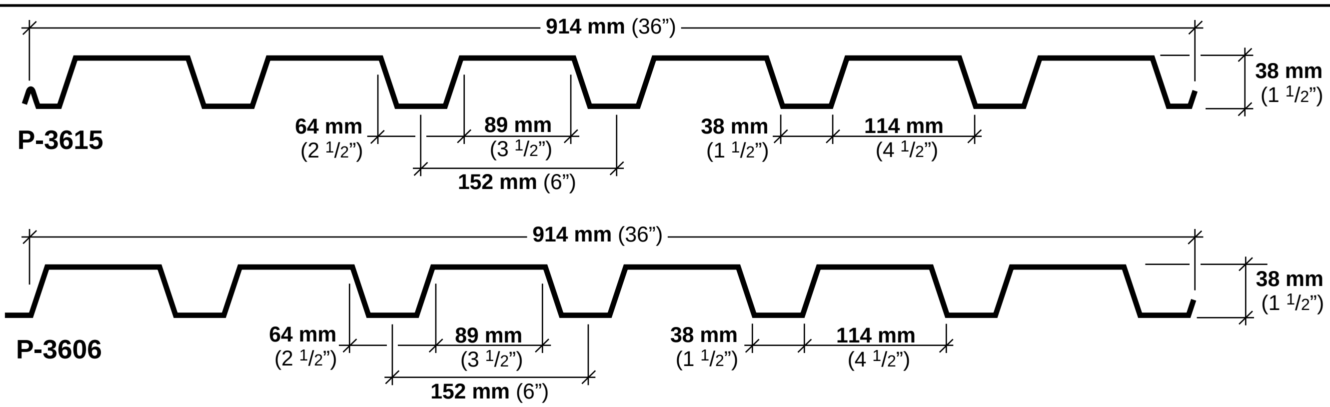
Canam's steel deck profiles P-3615 and P-3606 are roll formed to cover 914 mm (36 in.).



The deck is available with a galvanized coating according to the standard ASTM A 653M with zinc thickness corresponding to Z275 (G90) or ZF75 (A25). Upon agreement with our sales department, it is also possible to obtain steel deck with aluminium-zinc coating according to designation AZM150 (AZ50) of the standard ASTM A 792M.

Nominal thicknesses range from 0.76 mm (0.030 in.) to 1.52 mm (0.060 in.). The flutes are 38 mm (1.5 in.) deep and are spaced at 152 mm (6 in.) center to center. The deck can be rolled to lengths from 1 800 mm (6 ft.) to 12 200 mm (40 ft.).

Dimensions



Physical Properties

Type	Nominal Thickness	Design Thickness	Overall Depth	Weight	Section Modulus M⁺	Section Modulus M⁻	Moment of Inertia for Deflection
	mm (in.)	mm (in.)	mm (in.)	kg/m^2 (lb/ft^2)	mm^3 (in^3)	mm^3 (in^3)	mm^4 (in^4)
22	**0.76** (0.030)	**0.762** (0.0300)	**37.4** (1.47)	**8.50** (1.74)	**9 529** (0.1772)	**10 081** (0.1875)	**202 228** (0.1481)
20	**0.91** (0.036)	**0.909** (0.0358)	**37.5** (1.48)	**10.07** (2.06)	**11 558** (0.2150)	**12 005** (0.2233)	**254 750** (0.1865)
18	**1.21** (0.048)	**1.217** (0.0479)	**37.8** (1.49)	**13.26** (2.72)	**15 813** (0.2941)	**15 994** (0.2975)	**363 493** (0.2662)
16	**1.52** (0.060)	**1.511** (0.0595)	**38.1** (1.50)	**16.34** (3.35)	**19 786** (0.3680)	**19 786** (0.3680)	**452 472** (0.3313)

- Effective properties are based on a unit width of 1 000 mm (S.I. units) or 12 in. (imperial units).
- Material according to ASTM A 653M SS Grade 230, yield strength of 230 MPa (33 ksi).
- Tables are calculated according to CAN/CSA-S136-01 standard.

CANAM
Solutions + Service

Factored and Service Loads Table (kPa)

Metric

Type	Nominal Thickness (mm)		SPAN (mm) 1 200	1 350	1 500	1 650	1 800	1 950	2 100	2 250	2 400	2 550	2 700	2 850	3 000
			SINGLE SPAN												
22	0.76	F	10.69	8.49	6.90	5.72	4.82								
		D	7.60	5.34	3.89	2.92	2.25								
20	0.91	F	12.95	10.29	8.37	6.93	5.84	4.98							
		D	9.58	6.73	4.90	3.68	2.84	2.23							
18	1.21	F	17.70	14.06	11.44	9.48	7.98	6.82	5.89	5.13					
		D	13.66	9.60	7.00	5.26	4.05	3.18	2.55	2.07					
16	1.52	F	22.14	17.59	14.31	11.86	9.99	8.53	7.36	6.42	5.65				
		D	17.01	11.95	8.71	6.54	5.04	3.96	3.17	2.58	2.13				
			DOUBLE SPAN												
22	0.76	F	11.11	8.85	7.22	5.99	5.05	4.32	3.73						
		D	18.31	12.86	9.38	7.04	5.43	4.27	3.42						
20	0.91	F	13.23	10.54	8.59	7.14	6.02	5.14	4.44	3.88					
		D	23.07	16.20	11.81	8.87	6.84	5.38	4.30	3.50					
18	1.21	F	17.63	14.05	11.45	9.51	8.02	6.85	5.92	5.17	4.55	4.03	3.60		
		D	32.92	23.12	16.85	12.66	9.75	7.67	6.14	4.99	4.11	3.43	2.89		
16	1.52	F	21.82	17.39	14.17	11.77	9.92	8.48	7.33	6.39	5.63	4.99	4.46	4.00	3.62
		D	40.97	28.78	20.98	15.76	12.14	9.55	7.65	6.22	5.12	4.27	3.60	3.06	2.62
			TRIPLE SPAN												
22	0.76	F	(13.60)	10.88	8.90	7.40	6.25	5.35	4.63	4.04					
		D	14.35	10.08	7.35	5.52	4.25	3.34	2.68	2.18					
20	0.91	F	16.19	12.96	10.59	8.82	7.45	6.37	5.51	4.82	4.24	3.77			
		D	18.08	12.70	9.26	6.96	5.36	4.21	3.37	2.74	2.26	1.88			
18	1.21	F	21.59	17.27	14.12	11.75	9.93	8.49	7.35	6.42	5.65	5.02	4.48	4.03	
		D	25.80	18.12	13.21	9.92	7.64	6.01	4.81	3.91	3.22	2.69	2.26	1.93	
16	1.52	F	26.72	21.38	17.47	14.54	12.28	10.51	9.09	7.94	6.99	6.21	5.55	4.98	4.50
		D	32.11	22.56	16.44	12.35	9.52	7.48	5.99	4.87	4.01	3.35	2.82	2.40	2.06

Factored and Service Loads Table (psf)

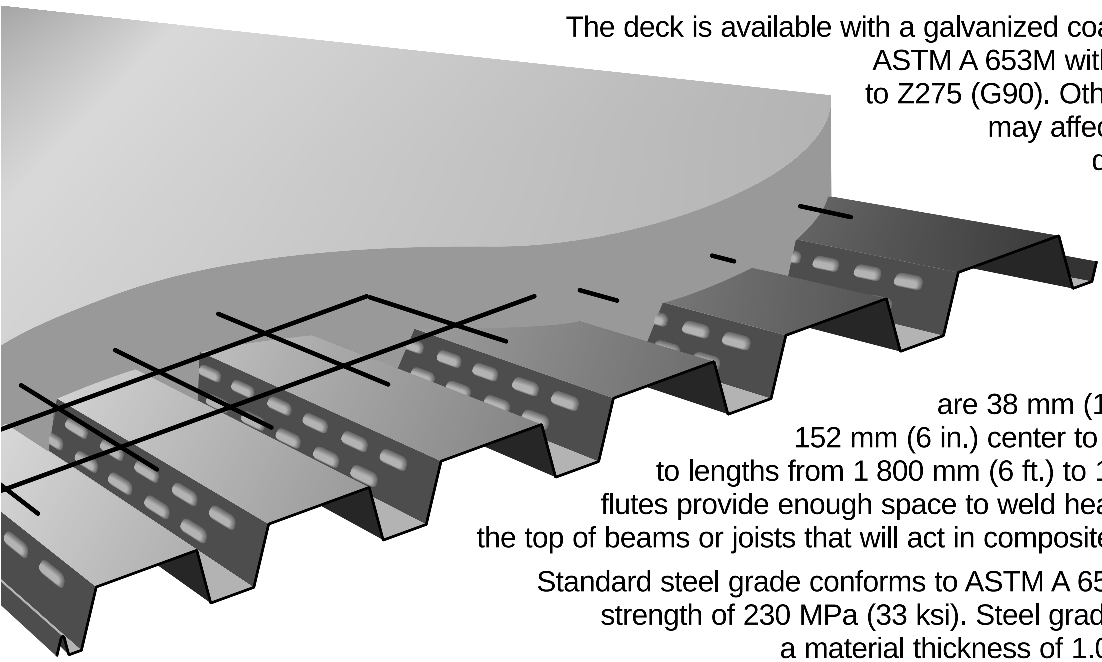
Imperial

Type	Nominal Thickness (in.)		SPAN (ft.-in.) 4'-0"	4'-6"	5'-0"	5'-6"	6'-0"	6'-6"	7'-0"	7'-6"	8'-0"	8'-6"	9'-0"	9'-6"	10'-0"
			SINGLE SPAN												
22	0.030	F	216	172	140	116	97								
		D	151	106	78	58	45								
20	0.036	F	262	208	169	140	118	101							
		D	191	134	98	73	57	44							
18	0.048	F	358	285	232	192	162	138	119	104					
		D	272	191	139	105	81	63	51	41					
16	0.060	F	448	356	290	240	202	173	149	130	114				
		D	339	238	173	130	100	79	63	51	42				
			DOUBLE SPAN												
22	0.030	F	225	179	146	121	102	87	76						
		D	365	256	187	140	108	85	68						
20	0.036	F	268	214	174	144	122	104	90	78					
		D	459	323	235	177	136	107	86	70					
18	0.048	F	357	285	232	193	162	139	120	105	92	82	73		
		D	655	460	336	252	194	153	122	99	82	68	58		
16	0.060	F	442	352	287	238	201	172	148	129	114	101	90	81	73
		D	816	573	418	314	242	190	152	124	102	85	72	61	52
			TRIPLE SPAN												
22	0.030	F	276	220	180	150	127	108	94	82					
		D	286	201	146	110	85	67	53	43					
20	0.036	F	328	263	215	179	151	129	112	98	86				
		D	360	253	184	139	107	84	67	55	45				
18	0.048	F	438	350	286	238	201	172	149	130	114	102	91		
		D	514	361	263	198	152	120	96	78	64	54	45		
16	0.060	F	542	433	354	294	249	213	184	161	142	126	112	101	91
		D	640	449	327	246	189	149	119	97	80	67	56	48	41

- Loads in rows marked "F" are the maximum factored loads controlled by the bending capacity, and those in rows marked "D" are the uniform service loads that produce a deflection of L/240.
- Loads in rows marked "F" should be compared to factored loads according to CAN/CSA-S16-01 Limit States Design of Steel Structure.
- The live loads producing deflection equal to the span/180 or span/360 can be calculated by multiplying the loads in the "D" rows by 1.33 or 0.66 respectively.
- Web crippling controls loads in brackets calculated with the end bearing length equal to 40 mm (1.6 in.) and the interior bearing length equal to 102 mm (4 in.). Refer to page 24 for web crippling tables and examples.
- The span is the shortest of the following dimensions: dimension c/c of the supports, or the clear dimension between the supports plus the depth of the deck at each end.
- Refer to page 34 for maximum spans approved by Factory Mutual (FM).

P-3615 & P-3606 COMPOSITE

Canam's composite P-3615 and P-3606 steel deck profiles are roll formed to cover 914 mm (36 in.).

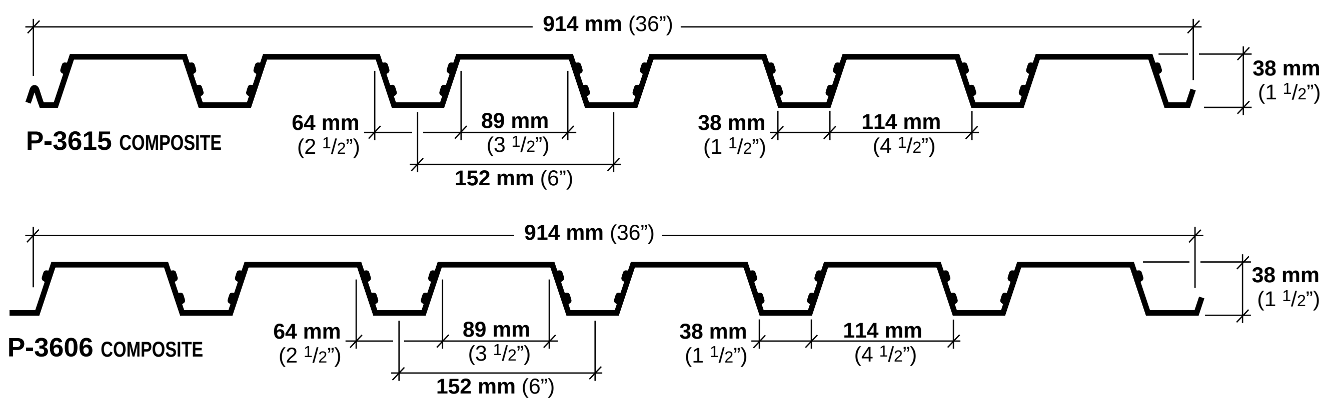


The deck is available with a galvanized coating according to the standard ASTM A 653M with zinc thickness corresponding to Z275 (G90). Other types of steel sheet finishes may affect the bond properties between deck and concrete. Contact our sales department for more information.

Nominal thicknesses are 0.76 mm (0.030 in.), 0.91 mm (0.036 in.) and 1.21 mm (0.048 in.). The flutes are 38 mm (1.5 in.) deep and are spaced at 152 mm (6 in.) center to center. The deck can be rolled to lengths from 1 800 mm (6 ft.) to 12 200 mm (40 ft.). The narrow flutes provide enough space to weld headed studs through the deck to the top of beams or joists that will act in composite action with the concrete slab.

Standard steel grade conforms to ASTM A 653M SS Grade 230 with a yield strength of 230 MPa (33 ksi). Steel grades up to 350 MPa (50 ksi) and a material thickness of 1.07 mm (0.042 in.) are available given sufficient delivery time.

Dimensions



Physical Properties

Type	Nominal Thickness	Design Thickness	Overall Depth	Weight	Section Modulus M+	Section Modulus M−	Moment of Inertia	Steel Area	Center of Gravity
	mm (in.)	mm (in.)	mm (in.)	kg/m² (lb/ft²)	mm³ (in³)	mm³ (in³)	mm⁴ (in⁴)	mm² (in²)	mm (in.)
22	**0.76** (0.030)	**0.762** (0.0300)	**37.4** (1.47)	**8.50** (1.74)	**9 529** (0.1772)	**10 081** (0.1875)	**202 228** (0.1481)	**1 016** (0.480)	**22.50** (0.89)
20	**0.91** (0.036)	**0.909** (0.0358)	**37.5** (1.48)	**10.07** (2.06)	**11 558** (0.2150)	**12 005** (0.2233)	**254 750** (0.1865)	**1 212** (0.573)	**22.58** (0.89)
18	**1.21** (0.048)	**1.217** (0.0479)	**37.8** (1.49)	**13.26** (2.72)	**15 813** (0.2941)	**15 994** (0.2975)	**363 493** (0.2662)	**1 622** (0.766)	**22.73** (0.89)

- Effective properties are based on a unit width of 1 000 mm (S.I. units) or 12 in. (imperial units).
- Material according to ASTM A 653M SS Grade 230, yield strength of 230 MPa (33 ksi).
- Tables are calculated according to CAN/CSA-S136-01 standard.

P-3615 & P-3606 COMPOSITE

FACTORED RESISTANCE TABLE OF COMPOSITE SLAB (kPa)

METRIC

Slab Thick. (mm)	Deck Thick. (mm)	Maximum Unshored Span			Self Weight (kPa)	Comp. Mom. of Inertia (10^6 mm^4)	SPAN (mm)												
		Single (mm)	Double (mm)	Triple (mm)			1 200	1 350	1 500	1 650	1 800	1 950	2 100	2 250	2 400	2 550	2 700	2 850	3 000
90																			
	0.76	1 690	1 995	1 980	1.62	3.917	20.00	20.00	20.00	20.00	18.90	15.99	13.69	11.84	10.33	9.08	8.04	7.16	6.42
	0.91	1 940	2 285	2 265	1.63	4.185	20.00	20.00	20.00	20.00	20.00	18.35	16.01	14.11	12.55	11.24	10.14	9.21	8.40
	1.21	2 405	2 735	2 790	1.66	4.690	20.00	20.00	20.00	20.00	20.00	20.00	20.00	20.00	19.07	17.59	16.31	15.20	13.85
100																			
	0.76	1 630	1 920	1 905	1.85	5.360	20.00	20.00	20.00	20.00	20.00	18.36	15.72	13.59	11.86	10.43	9.23	8.22	7.37
	0.91	1 865	2 195	2 170	1.86	5.721	20.00	20.00	20.00	20.00	20.00	20.00	18.38	16.20	14.41	12.91	11.65	10.57	9.65
	1.21	2 305	2 630	2 670	1.89	6.403	20.00	20.00	20.00	20.00	20.00	20.00	20.00	20.00	20.00	20.00	18.74	17.46	16.33
115																			
	0.76	1 550	1 820	1 805	2.20	8.134	20.00	20.00	20.00	20.00	20.00	20.00	18.76	16.22	14.15	12.45	11.02	9.82	8.79
	0.91	1 770	2 075	2 055	2.22	8.666	20.00	20.00	20.00	20.00	20.00	20.00	20.00	19.34	17.20	15.41	13.90	12.62	11.52
	1.21	2 180	2 490	2 515	2.24	9.678	20.00	20.00	20.00	20.00	20.00	20.00	20.00	20.00	20.00	20.00	20.00	20.00	19.51
125																			
	0.76	1 505	1 765	1 745	2.44	10.432	20.00	20.00	20.00	20.00	20.00	20.00	20.00	17.98	15.68	13.79	12.21	10.88	9.74
	0.91	1 715	2 010	1 985	2.45	11.101	20.00	20.00	20.00	20.00	20.00	20.00	20.00	20.00	19.06	17.08	15.41	13.98	12.76
	1.21	2 110	2 410	2 430	2.48	12.378	20.00	20.00	20.00	20.00	20.00	20.00	20.00	20.00	20.00	20.00	20.00	20.00	20.00
140																			
	0.76	1 440	1 690	1 670	2.79	14.627	20.00	20.00	20.00	20.00	20.00	20.00	20.00	20.00	17.98	15.81	14.00	12.47	11.17
	0.91	1 640	1 920	1 895	2.81	15.535	20.00	20.00	20.00	20.00	20.00	20.00	20.00	20.00	20.00	19.58	17.66	16.03	14.63
	1.21	2 010	2 300	2 315	2.83	17.278	20.00	20.00	20.00	20.00	20.00	20.00	20.00	20.00	20.00	20.00	20.00	20.00	20.00
150																			
	0.76	1 405	1 645	1 625	3.03	17.965	20.00	20.00	20.00	20.00	20.00	20.00	20.00	20.00	19.51	17.16	15.19	13.53	12.12
	0.91	1 595	1 870	1 845	3.04	19.056	20.00	20.00	20.00	20.00	20.00	20.00	20.00	20.00	20.00	20.00	19.17	17.40	15.88
	1.21	1 955	2 235	2 245	3.07	21.155	20.00	20.00	20.00	20.00	20.00	20.00	20.00	20.00	20.00	20.00	20.00	20.00	20.00

- **The table is based on concrete density of 2 400 kg/m^3 and minimum compressive resistance (f'_c) equal to 20 MPa at 28 days.**
- During construction, the steel deck must support itself, the concrete and a construction uniform load of 1 kPa or a transverse load of 2 kN/m, as specified by the Canadian Sheet Steel Building Institute.
- The maximum unshored spans shown in the table are established for bending under the slab self-weight and the construction loads, for web crippling and for the deflection under wet concrete to be less than the span over 180 (L/180). The web crippling resistance is calculated assuming the end bearing length equal to 40 mm and the interior bearing length equal to 102 mm.

  If the bearing length is shorter, the design engineer must verify the web crippling factored resistance with the reaction produced by wet concrete and construction factored loads (refer to page 24 for web crippling tables and examples).
- Contact Canam sales personnel when the total uniform load exceeds 20 kPa, as this is an indication that significant concentrated loads will be used. The composite slab and its reinforcing should be verified for the effect of concentrated loads (see notes on page 5).
- Shaded values indicate that the deck should be shored at mid-span during the pour and the curing of concrete for those spans and concrete thickness conditions. Shaded values correspond to the maximum unshored span values shown at the left of the table.
- The design engineer is responsible for specifying size and location of the wire mesh in the concrete slab in order to respect current concrete practices.

EXAMPLE

Triple span of 1 800 mm, total slab thickness of 100 mm with 62 mm of concrete cover on top of 38 mm deck profile.

Once the concrete is cured, the composite slab will have to support these loads:

Dead load = 1.50 kPa
Service live load = 4.80 kPa

According to the table of maximum unshored span above, we need to use a deck with a nominal thickness of 0.76 mm for a triple span condition.

Deck and concrete weights are 1.85 kPa (shown in the table).

Total factored load

$w_f = 1.25 \times (1.85 + 1.50) + 1.5 \times 4.80 = 11.39$ kPa

Factored resistance

$w_r = 20.00$ kPa for a span of 1 800 mm, with a 100 mm slab and a 0.76 mm thick deck.

$w_r > w_f$ OK

Service load w = 4.80 kPa

Composite moment of inertia is 5.360 x 10^6 mm^4 (from the table).

$$\text{Deflection} = \frac{5\,w\,L^4}{384\,E_s\,I_{comp}} = \frac{5 \times 4.80 \times 1\,800^4}{384 \times 203\,000 \times 5\,360\,000}$$

$$= 0.6 \text{ mm} < \frac{1\,800}{360} = 5.0 \text{ mm} \quad \textbf{OK}$$

GENERAL NOTES

1. GENERAL

1.1. BEFORE PROCEEDING WITH THE WORK, VERIFY ALL DIMENSIONS WITH THE ACTUAL CONDITIONS AND REPORT DISCREPANCIES TO THE CONSULTANT.

1.2. PROVIDE LABOUR, MATERIALS, PLANT AND EQUIPMENT TO COMPLETE ALL STRUCTURAL WORK INDICATED ON THE CONTRACT DOCUMENTS.

1.3. THESE DRAWINGS SHOW THE COMPLETED STRUCTURE. THE CONTRACTOR IS RESPONSIBLE FOR SAFETY ON THE JOB SITE, AND DESIGN, INSTALLATION AND SUPERVISION OF ALL TEMPORARY BRACING, SHORING, FORM WORK AND FALSE WORK, REQUIRED TO COMPLETE THE WORK.

1.4. ANY DAMAGE TO EXISTING BUILDING OR TO NEIGHBORING PROPERTIES IS NOT PERMITTED. THE CONTRACTOR IS RESPONSIBLE FOR MAKING GOOD ALL UNAVOIDABLE DAMAGE.

1.5. THE USE OF THESE DRAWINGS SHALL BE STRICTLY LIMITED TO THE INSTRUCTIONS IN THE REVISIONS BLOCK. BUILDING FROM THESE DRAWINGS SHALL PROCEED ONLY WHEN MARKED "FOR CONSTRUCTION".

2. REFERENCE STANDARDS/CODES AND ACTS:

2.1. CONFORM WITH THE ONTARIO BUILDING CODE LATEST EDITION AND AMENDMENTS, AND ANY APPLICABLE ACTS OF ANY AUTHORITY HAVING JURISDICTION, AND THE FOLLOWING:

2.2. ALL MATERIALS, CONSTRUCTION AND WORKMANSHIP SHALL CONFIRM TO CSA S16-14 AND CSA G40.21.

2.3. DO WELDING WORK TO CSA W59-18 UNLESS SPECIFIED OTHERWISE.

2.4. COMPLY WITH OCCUPATIONAL HEALTH AND SAFETY ACT AND REGULATIONS.

2.5. WHERE THERE ARE DIFFERENCES IN REQUIREMENTS OF THE DOCUMENTS AND THE STANDARDS, CODES AND ACTS, THE MOST STRINGENT REQUIREMENTS SHALL GOVERN.

3. MATERIALS:

3.1. STRUCTURAL STEEL FOR SECONDARY BEAMS (JOISTS) AND GIRDERS TO BE ASTM GRADE A992

3.2. CSA G40.21 350W STEEL ANGLES TO BE USED FOR BRACING

3.3. COLUMNS TO BE ASTM GRADE A992 STEEL

3.4. BASE PLATES TO BE CAN/CSA G40.20/G40.21 GRADE 300W STEEL

3.5. ALL BOLTS TO BE $\frac{3}{4}$" ASTM A325 BOLTS

4. QUALITY CONTROL:

4.1. IMPLEMENT A SYSTEM OF QUALITY CONTROL TO ENSURE THAT MINIMUM STANDARDS SPECIFIED HEREIN ARE ATTAINED.

4.2. BRING TO ATTENTION OF ENGINEER ANY DEFECTS IN THE WORK OR DEPARTURES FROM CONTRACT DOCUMENTS, WHICH MAY OCCUR DURING CONSTRUCTION. ENGINEER WILL DECIDE UPON CORRECTIVE ACTION AND GIVE RECOMMENDATIONS IN WRITING.

4.3. ENGINEER'S GENERAL REVIEW DURING CONSTRUCTION AND INSPECTION AND TESTING BY INDEPENDENT INSPECTION AND TESTING AGENCIES REPORTING TO ENGINEER ARE BOTH UNDERTAKEN TO INFORM THE OWNER/CLIENT OF CONTRACTOR'S PERFORMANCE AND SHALL IN NO WAY RELIEVE CONTRACTOR OF CONTRACTUAL RESPONSIBILITIES.

DRAWING LIST

S1 - GENERAL NOTES
S2 - LEVEL 1 FRAMING PLAN
S3 - LEVEL 2 FRAMING PLAN
S4 - LEVEL 3 FRAMING PLAN
S5 - LEVEL 4 FRAMING PLAN
S6 - ROOF FRAMING PLAN
S7 - EAST AND WEST ELEVATION
S8 - NOTH AND SOUTH ELEVATION
S9 - JOIST AND GIRDER SCHEDULE
S10 - COLUMN SCHEDULE

No.	DATE	DESCRIPTION
1	20/01/20	REVIEW

STAMP

NORTH

greentech engineering

KEVIN LUONG
DAVID MOORE
REGAN O'HENLY
OLIVIA PARSONS
DESMOND YEUNG
DAN ZHAO

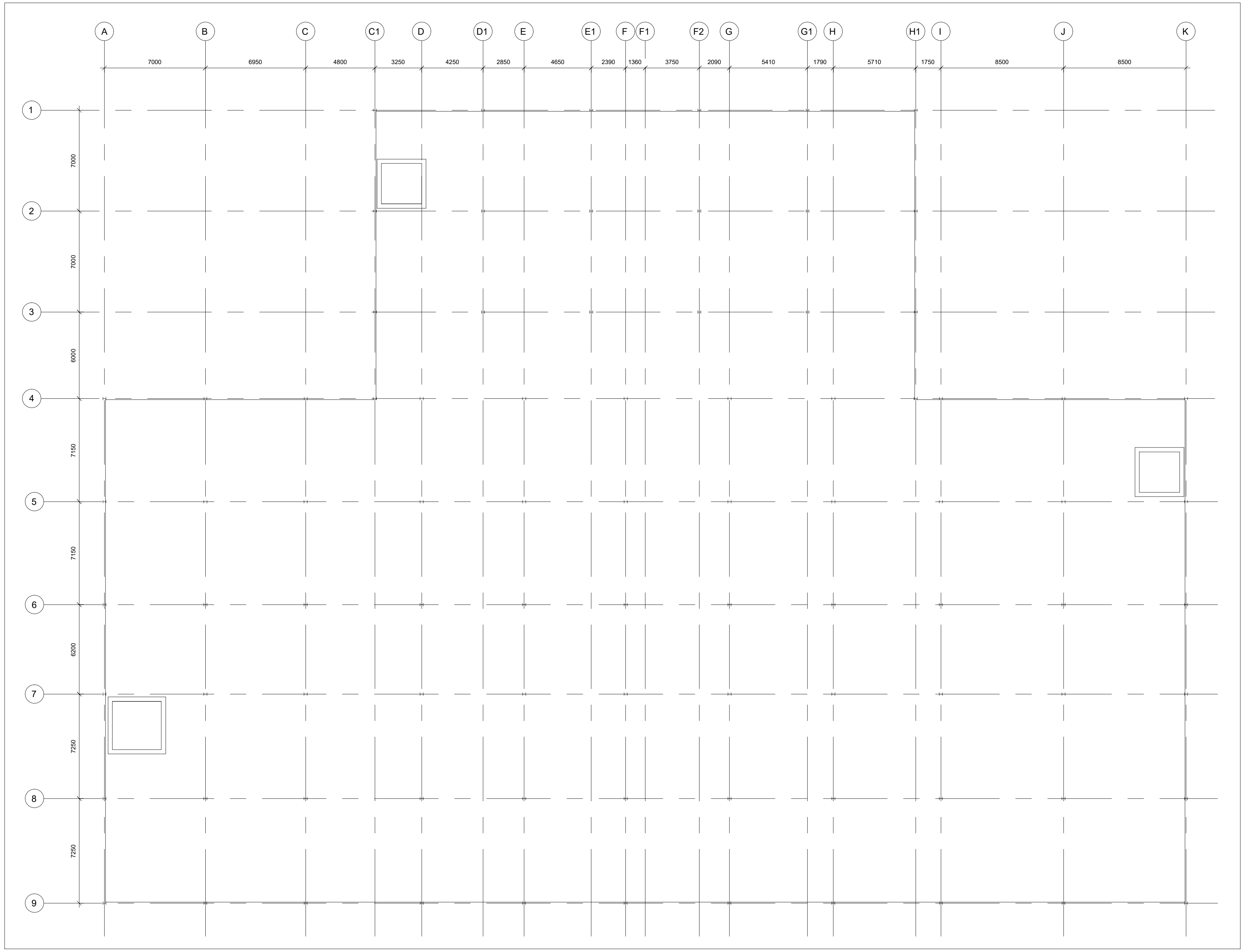
LOCATION
STONEY CREEK, ON

PROJECT
FRUITLAND VERTICAL FARM AND MARKETPLACE

DRAWING TITLE
GENERAL NOTES

PROJECT No: 001
DATE: 20 JANUARY 2019
SCALE: –
DRAWING BY: KL & DM
APPROVED BY: KL & DM
DRAWING No: S1

PRODUCED BY AN AUTODESK STUDENT VERSION
PRODUCED BY AN AUTODESK STUDENT VERSION



1 LEVEL 1 FRAMING PLAN
S2 1:350

GROUND FLOOR FRAMING PLAN

NOTES:

1. GROUND FLOOR DATUM ELEVATION (0.00).
2. DO NOT SCALE THE DRAWING.
3. UNLESS OTHERWISE SPECIFIED, ALL DIMENSIONS SHALL BE IN MILLIMETER.
4. ALL COLUMNS MUST BE RIGIDLY CONNECTED TO FOOTINGS.
5. REFER TO DRAWING S1 FOR GENERAL NOTES.
6. REFER TO DRAWING S2-S6 FOR TYPICAL DETAILS.
7. REFER TO DRAWING S7-S8 FOR ELEVATION VIEW.
9. REFER TO DRAWING S9 FOR JOIST AND GIRDER SCHEDULE.
10. REFER TO DRAWING S10 FOR COLUMN SCHEDULE.

CONTRACTOR MUST CHECK AND VERIFY ALL DIMENSIONS AND JOB SITE CONDITIONS AND REPORT ANY DISCREPANCIES TO THE ENGINEER PRIOR TO COMMENCING CONSTRUCTION.

No.	DATE	DESCRIPTION
1	20/01/20	REVIEW

STAMP

NORTH

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LOCATION

STONEY CREEK, ON

PROJECT

FRUITLAND VERTICAL FARM AND MARKETPLACE

DRAWING TITLE

LEVEL 1 FRAMING PLAN

PROJECT No: 001

DATE: 20 JANUARY 2019

SCALE: –

DRAWING BY: KL & DM

APPROVED BY: KL & DM

DRAWING No

S2

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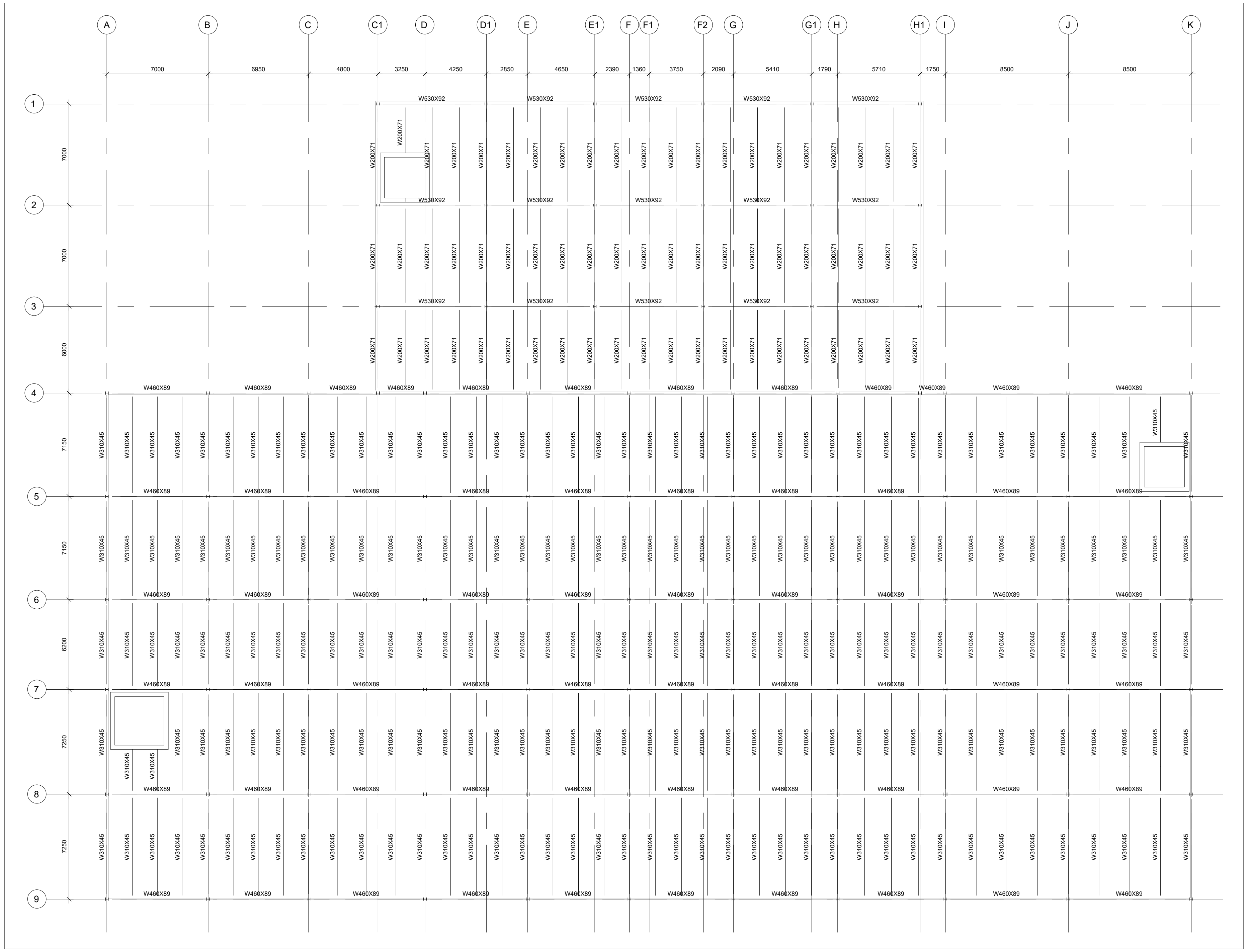

1 LEVEL 2 FRAMING PLAN
S3 1:350

SECOND FLOOR FRAMING PLAN

NOTES:

1. SECOND FLOOR DATUM ELEVATION +4000.00 mm RELATIVE TO THE GROUND FLOOR, EXCEPT OTHERWISE STATED.
2. DO NOT SCALE THE DRAWING.
3. UNLESS OTHERWISE SPECIFIED, ALL DIMENSIONS SHALL BE IN MILLIMETER.
4. SUPERIMPOSED LOADS USED IN DESIGN:
   DEAD LOAD: 1.72 kPa
   LIVE LOAD: 4.8 kPa
5. SELF WEIGHT OF STRUCTURE USED IN DESIGN:
   DECK & SLAB: 1.84 kPa
   FRAMING: 0.5 kPa
6. THICKNESS OF STEEL DECK AND CONCRETE SLAB:
   STEEL DECK: 1.12 mm
   CONCRETE SLAB: 90 mm
7. MAINTAIN THE SLAB THICKNESS, EXCEPT OTHERWISE STATED.
8. REFER TO DRAWING S1 FOR GENERAL NOTES.
9. REFER TO DRAWING S2-S6 FOR TYPICAL DETAILS.
10. REFER TO DRAWING S7-S8 FOR ELEVATION VIEW.
11. REFER TO DRAWING S9 FOR JOIST AND GIRDER SCHEDULE.
12. REFER TO DRAWING S10 FOR COLUMN SCHEDULE.

No.	DATE	DESCRIPTION
1	20/01/20	REVIEW

STAMP

NORTH

greentech engineering

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REGAN O'HENLY
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LOCATION
STONEY CREEK, ON

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DRAWING TITLE
LEVEL 2 FRAMING PLAN

PROJECT No: 001

DRAWING No

DATE: 20 JANUARY 2019

SCALE: –

DRAWING BY: KL & DM

APPROVED BY: KL & DM

S3

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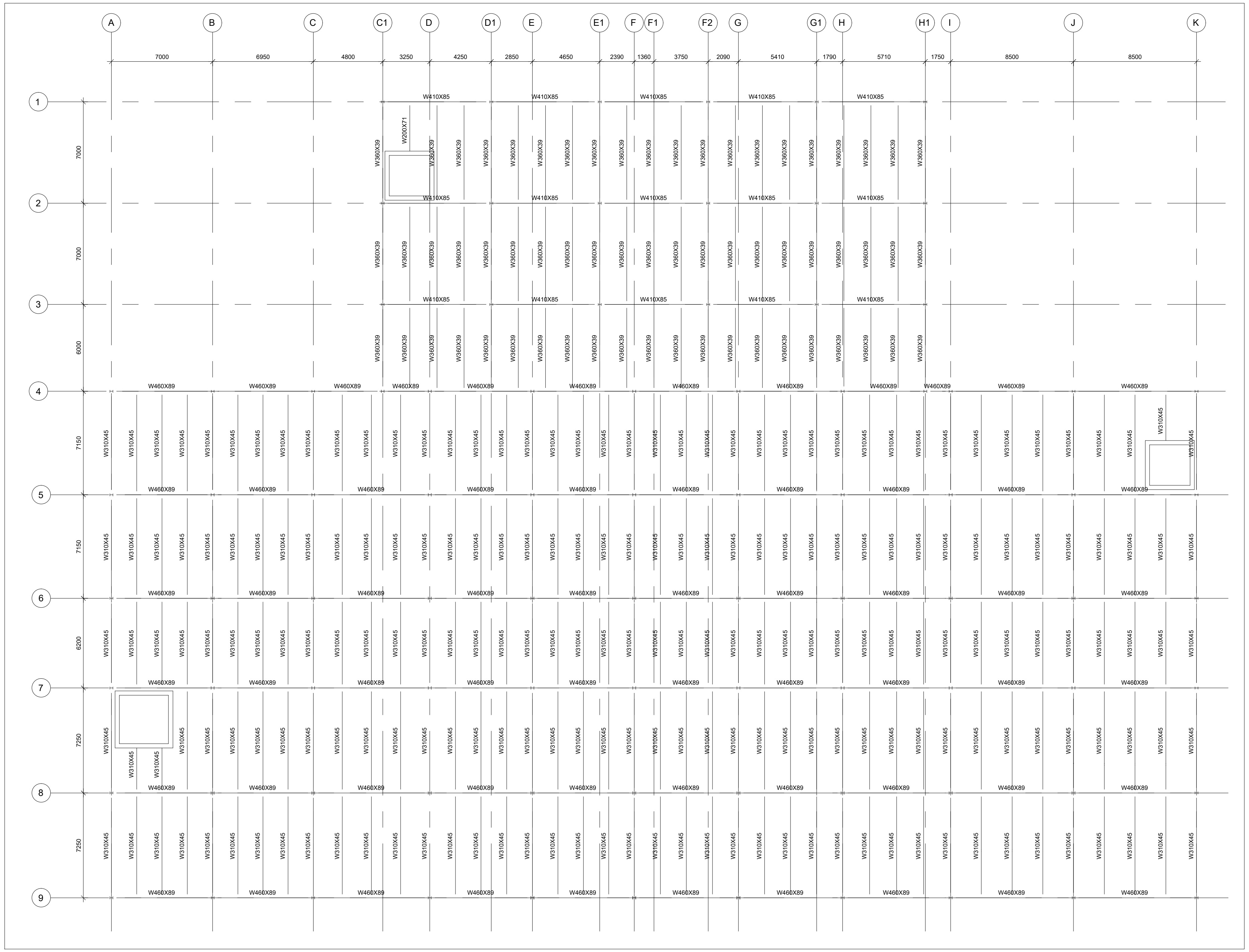

1 LEVEL 3 FRAMING PLAN
S4 1:350

THIRD FLOOR AND ROOF (MARKET PLACE) FRAMING PLAN

NOTES:

1. THIRD FLOOR AND MARKET PLACE'S ROOF DATUM ELEVATION +8000.00 mm RELATIVE TO THE GROUND FLOOR, EXCEPT OTHERWISE STATED.
2. DO NOT SCALE THE DRAWING.
3. UNLESS OTHERWISE SPECIFIED, ALL DIMENSIONS SHALL BE IN MILLIMETER.
4. SUPERIMPOSED LOADS USED IN MAIN BUILDING DESIGN:
   DEAD LOAD: 1.72 kPa
   LIVE LOAD: 4.8 kPa
5. SELF WEIGHT OF STRUCTURE USED IN MAIN BUILDING DESIGN:
   DECK & SLAB: 1.84 kPa
   FRAMING: 0.5 kPa
6. THICKNESS OF STEEL DECK AND CONCRETE SLAB (MAIN BUILDING):
   STEEL DECK: 1.12 mm
   CONCRETE SLAB: 90 mm
7. MAINTAIN THE SLAB THICKNESS, EXCEPT OTHERWISE STATED.
8. SUPERIMPOSED LOADS USED IN ROOF (MARKET PLACE) DESIGN:
   DEAD LOAD: 0.32 kPa
   LIVE LOAD: 1.0 kPa
   SNOW & RAIN LOAD: 5.48 kPa
9. SELF WEIGHT OF STRUCTURE USED IN ROOF (MARKET PLACE) DESIGN:
   DECK 0.3 kPa
   FRAMING: 0.5 kPa
10. THICKNESS OF STEEL DECK (MARKET PLACE):
    STEEL DECK: 1.12 mm
11. REFER TO ARCHITECTURAL DRAWING FOR ROOF SLOPE.
12. REFER TO DRAWING S1 FOR GENERAL NOTES.
13. REFER TO DRAWING S2-S6 FOR TYPICAL DETAILS.
14. REFER TO DRAWING S7-S8 FOR ELEVATION VIEW.
15. REFER TO DRAWING S9 FOR JOIST AND GIRDER SCHEDULE.
16. REFER TO DRAWING S10 FOR COLUMN SCHEDULE.

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DRAWING TITLE

LEVEL 3 FRAMING PLAN

PROJECT No: 001

DRAWING No

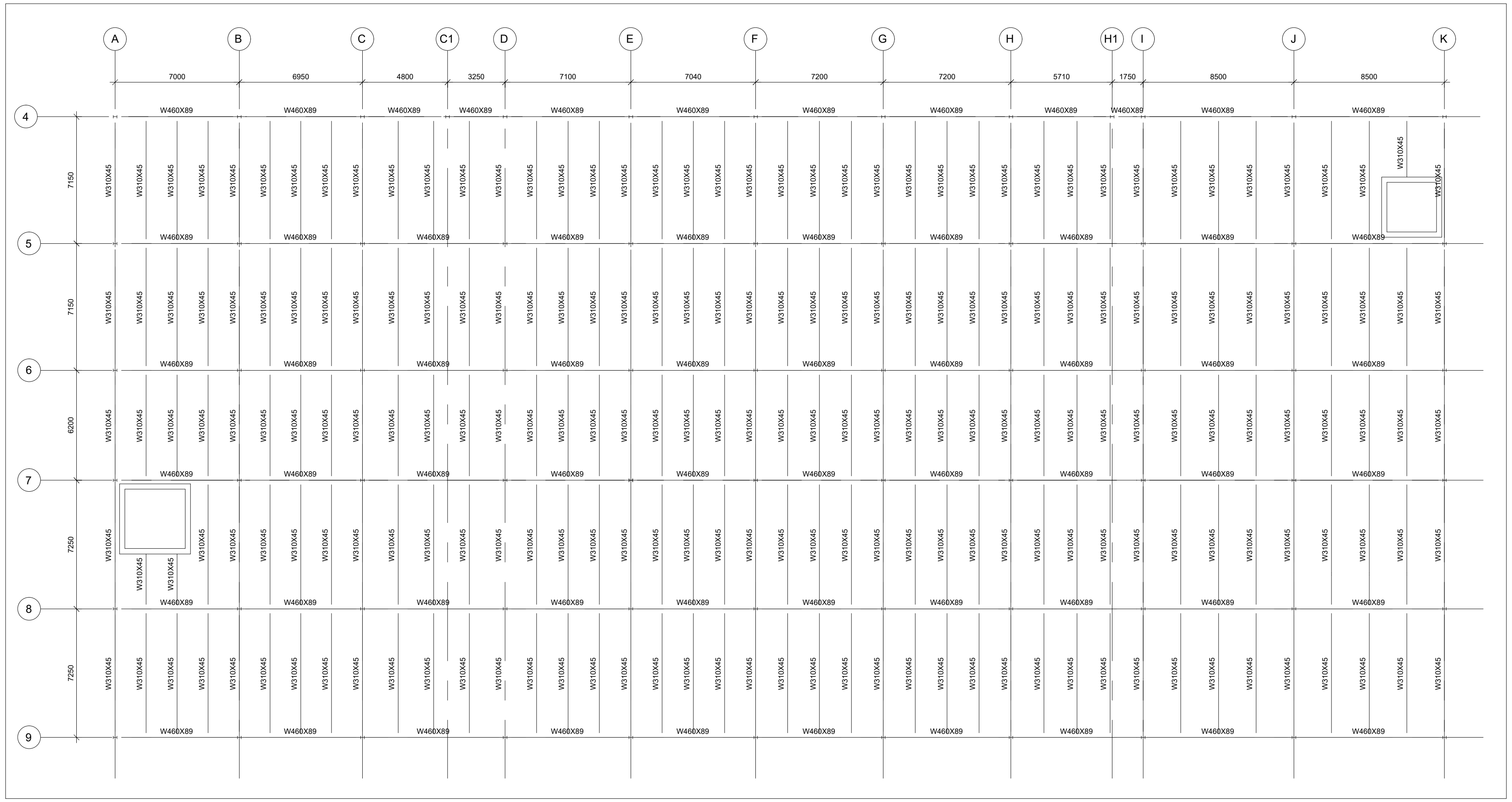
DATE: 20 JANUARY 2019

SCALE: –

DRAWING BY: KL & DM

APPROVED BY: KL & DM

S4



1 LEVEL 4 FRAMING PLAN
S5 1:350

FOUR FLOOR FRAMING PLAN

NOTES:

1. FOUR FLOOR DATUM ELEVATION +12000.00 mm RELATIVE TO THE GROUND FLOOR, EXCEPT OTHERWISE STATED.
2. DO NOT SCALE THE DRAWING.
3. UNLESS OTHERWISE SPECIFIED, ALL DIMENSIONS SHALL BE IN MILLIMETER.
4. SUPERIMPOSED LOADS USED IN DESIGN:
   DEAD LOAD: 1.72 kPa
   LIVE LOAD: 4.8 kPa
5. SELF WEIGHT OF STRUCTURE USED IN DESIGN:
   DECK & SLAB: 1.84 kPa
   FRAMING: 0.5 kPa
6. THICKNESS OF STEEL DECK AND CONCRETE SLAB:
   STEEL DECK: 1.12 mm
   CONCRETE SLAB: 90 mm
7. MAINTAIN THE SLAB THICKNESS, EXCEPT OTHERWISE STATED.
8. REFER TO DRAWING S1 FOR GENERAL NOTES.
9. REFER TO DRAWING S2-S6 FOR TYPICAL DETAILS.
10. REFER TO DRAWING S7-S8 FOR ELEVATION VIEW.
11. REFER TO DRAWING S9 FOR JOIST AND GIRDER SCHEDULE.
12. REFER TO DRAWING S10 FOR COLUMN SCHEDULE.

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STONEY CREEK, ON

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FRUITLAND VERTICAL FARM AND MARKETPLACE

DRAWING TITLE
LEVEL 4 FRAMING PLAN

PROJECT No: 001
DRAWING No S5
DATE: 20 JANUARY 2019
SCALE: –
DRAWING BY: KL & DM
APPROVED BY: KL & DM

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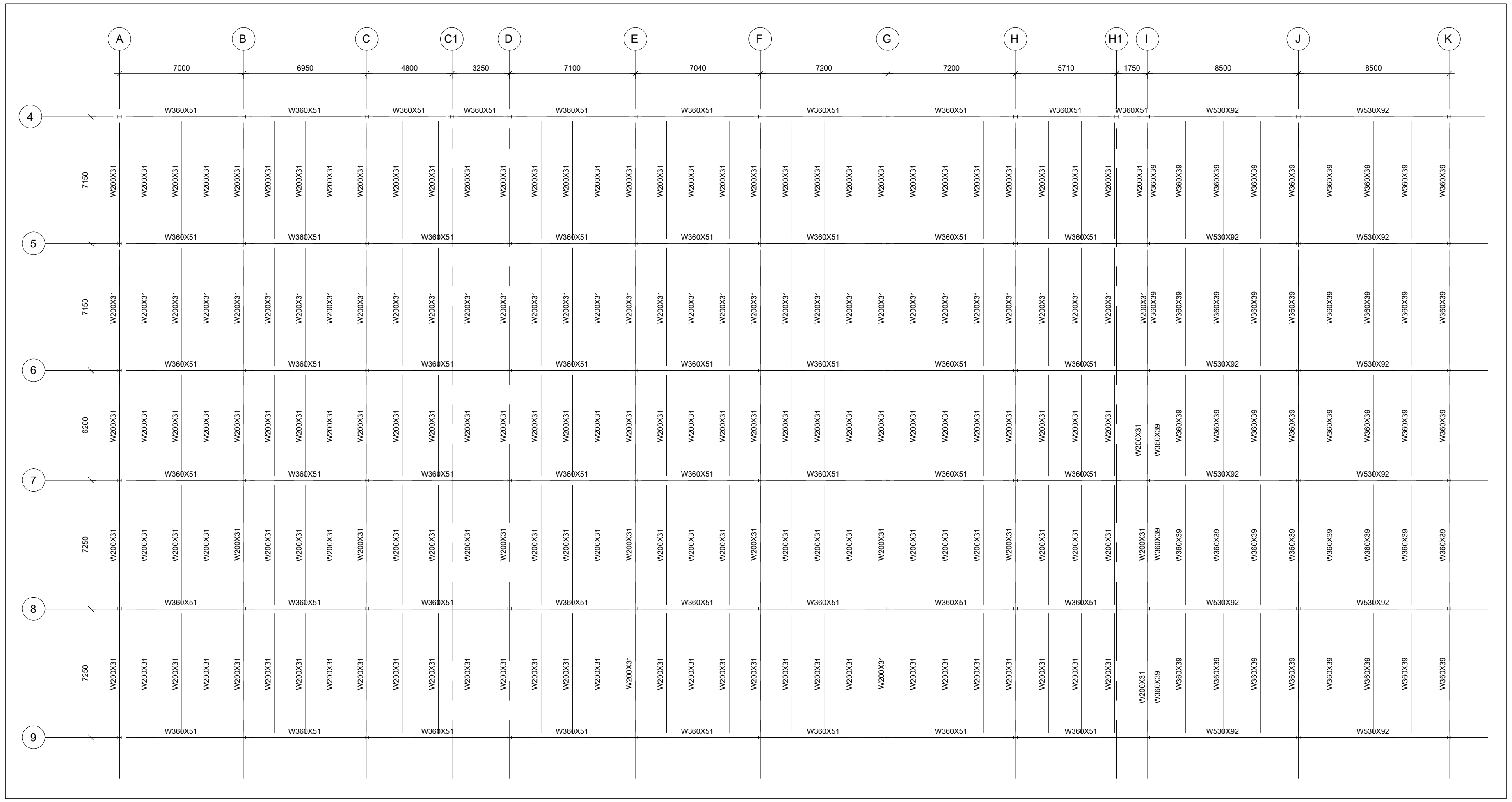

1 ROOF FRAMING PLAN
S6 1:350

ROOF (MAIN BUILDING) FRAMING PLAN

NOTES:

1. MAIN BUILDING'S ROOF DATUM ELEVATION +16000.00 mm RELATIVE TO THE GROUND FLOOR, EXCEPT OTHERWISE STATED.
2. DO NOT SCALE THE DRAWING.
3. UNLESS OTHERWISE SPECIFIED, ALL DIMENSIONS SHALL BE IN MILLIMETER.
4. SUPERIMPOSED LOADS USED IN CURVED ROOF DESIGN:
   DEAD LOAD: 0.32 kPa
   LIVE LOAD: 1.0 kPa
   SNOW & RAIN LOAD: 1.28 kPa
5. SELF WEIGHT OF STRUCTURE USED IN CURVED ROOF DESIGN:
   DECK 0.3 kPa
   FRAMING: 0.5 kPa
6. THICKNESS OF STEEL DECK OF CURVED ROOF:
   STEEL DECK: 0.76 mm
7. SUPERIMPOSED LOADS USED IN FLAT ROOF DESIGN:
   DEAD LOAD: 0.32 kPa
   LIVE LOAD: 1.0 kPa
   SNOW & RAIN LOAD: 5.48 kPa
8. SELF WEIGHT OF STRUCTURE USED IN FLAT ROOF DESIGN:
   DECK 0.3 kPa
   FRAMING: 0.5 kPa
9. THICKNESS OF STEEL DECK OF FLAT ROOF:
   STEEL DECK: 1.21 mm
10. REFER TO ARCHITECTURAL DRAWING FOR ROOF SLOPE.
11. REFER TO DRAWING S1 FOR GENERAL NOTES.
12. REFER TO DRAWING S2-S6 FOR TYPICAL DETAILS.
13. REFER TO DRAWING S7-S8 FOR ELEVATION VIEW.
14. REFER TO DRAWING S9 FOR JOIST AND GIRDER SCHEDULE.
15. REFER TO DRAWING S10 FOR COLUMN SCHEDULE.

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DRAWING TITLE
ROOF FRAMING PLAN

PROJECT No: 001
DRAWING No S6
DATE: 20 JANUARY 2019
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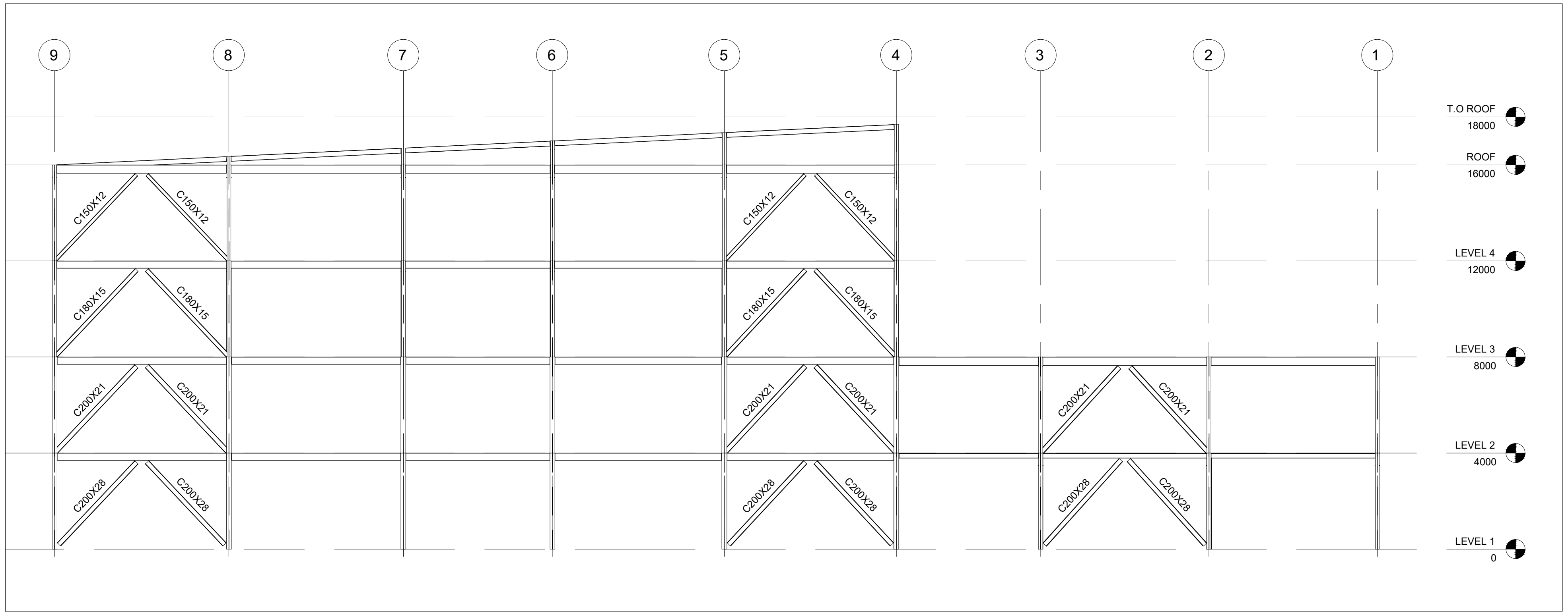
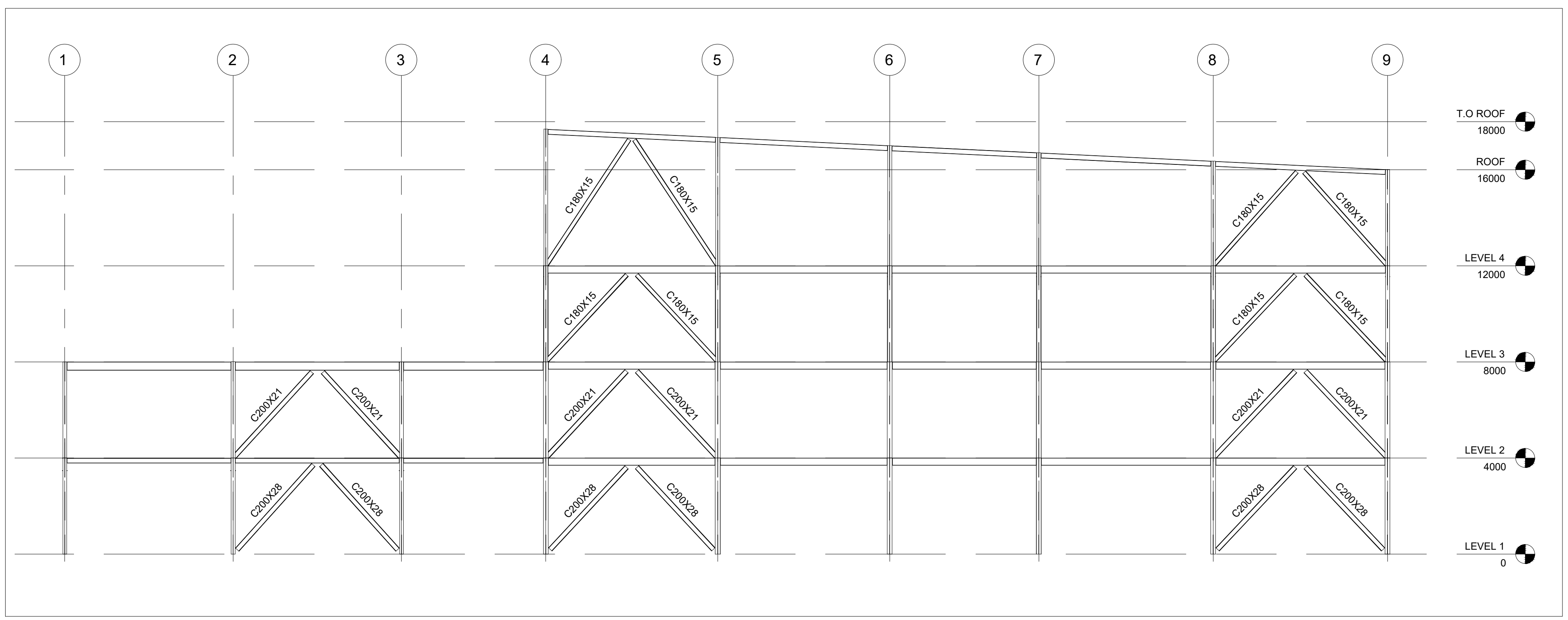


1 S7 EAST ELEVATION 1:250



1 S7 WEST ELEVATION 1:350

CONTRACTOR MUST CHECK AND VERIFY ALL DIMENSIONS AND JOB SITE CONDITIONS AND REPORT ANY DISCREPANCIES TO THE ENGINEER PRIOR TO COMMENCING CONSTRUCTION.

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FRUITLAND VERTICAL FARM AND MARKETPLACE

DRAWING TITLE

EAST AND WEST ELEVATION

PROJECT No: 001

DRAWING No

DATE: 20 JANUARY 2019

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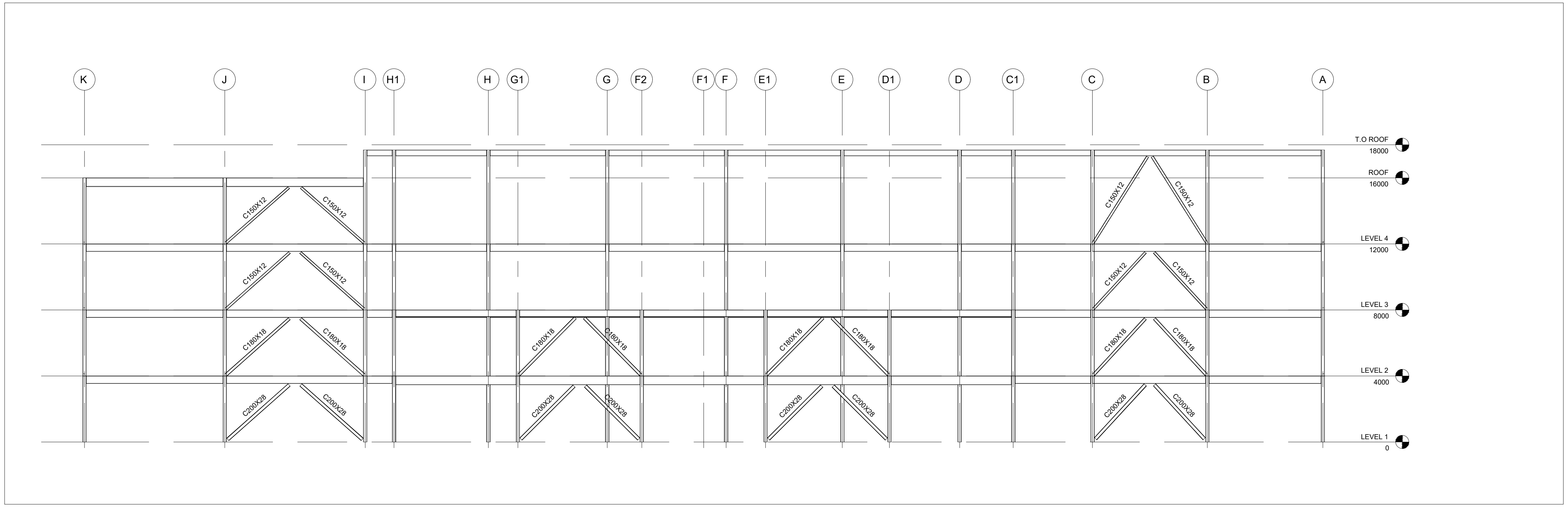
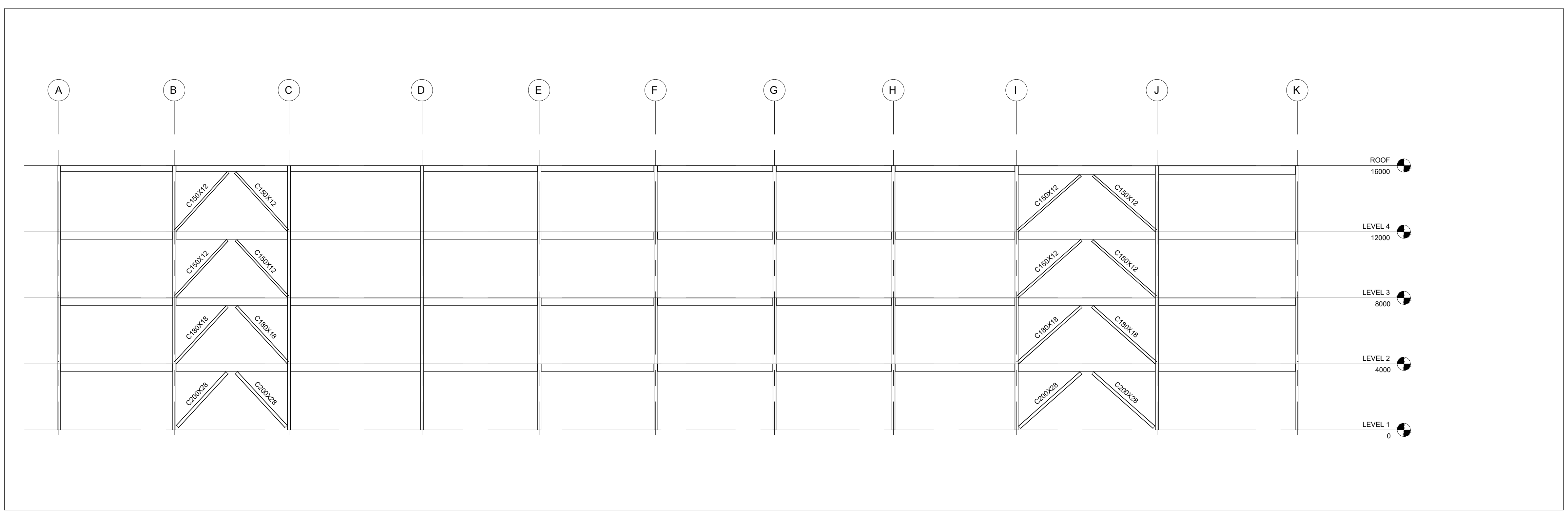


1 S8 NORTH ELEVATION
1:300



2 S8 SOUTH ELEVATION
1:300

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LOCATION
STONEY CREEK, ON

PROJECT
FRUITLAND VERTICAL FARM AND MARKETPLACE

DRAWING TITLE
NORTH AND SOUTH ELEVATION

PROJECT No: 001

DRAWING No

DATE: 20 JANUARY 2019

SCALE: –

DRAWING BY: KL & DM

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S8

JOIST SCHEDULE								
REGION	SECTION	LOAD TYPE	M_f (kNm)	M_r (kNm)	V_f (kN)	V_r (kN)	Δ_{max} (mm)	Δ_{limit} (mm)
FLOOR (MARKET PLACE)	W200x71	GRAVITY	141	249	81	542	13.8	19.4
FLOOR (MAIN BUILDING)	W310x45	GRAVITY	172	219	95	423	18.5	20.1
CURVED ROOF (MAIN BUILDING)	W200x31	GRAVITY	58	104	32	275	13.3	20.1
		UPLIFT	17	38	10	275	-	-
FLAT ROOF (MAIN BUILDING)	W360x39	GRAVITY	149	205	82	470	18.4	20.1
		UPLIFT	10	45	6	470	-	-
ROOF (MARKET PLACE)	W360x39	GRAVITY	123	205	70	170	14.0	19.4
		UPLIFT	16	49	9	170	-	-

GIRDER SCHEDULE								
REGION	SECTION	LOAD TYPE	M_f (kNm)	M_r (kNm)	V_f (kN)	V_r (kN)	Δ_{max} (mm)	Δ_{limit} (mm)
FLOOR (MARKET PLACE)	W460x89	GRAVITY	608	624	243	996	8.6	20.8
FLOOR (MAIN BUILDING)	W530x92	GRAVITY	663	732	234	1114	20.4	23.6
CURVED ROOF (MAIN BUILDING)	W360x51	GRAVITY	258	277	96	523	15.4	22.4
		UPLIFT	27	90	10	523	-	-
FLAT ROOF (MAIN BUILDING)	W530x92	GRAVITY	697	732	246	1114	20.9	23.6
		UPLIFT	30	262	11	1114	-	-
ROOF (MARKET PLACE)	W410x81	GRAVITY	525	534	210	931	20.5	20.8
		UPLIFT	23	251	9	931	-	-

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FRUITLAND VERTICAL FARM AND MARKETPLACE

DRAWING TITLE

JOIST AND GIRDER SCHEDULE

PROJECT No: 001

DRAWING No

DATE: 20 JANUARY 2019

SCALE: –

DRAWING BY: KL & DM

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S9

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T.O ROOF																	T.O ROOF
18000 ROOF																	18000 ROOF
16000 LEVEL 4	W200X42	W200X42	W200X42	W200X42	W200X42	W200X42	W200X42	W200X42	W200X42	W200X42		W200X42		W200X42	W200X42		16000 LEVEL 4
12000 LEVEL 3	W200X42	W200X42	W200X42	W200X42	W200X42	W200X42	W200X52	W200X52	W200X52	W200X52		W200X52		W200X42	W200X52		12000 LEVEL 3
8000 LEVEL 2	W200X52	W200X52	W200X52	W200X52	W200X52	W200X52	W200X71	W200X71	W200X71	W200X71	W200X42	W200X71	W200X42	W200X52	W200X71		8000 LEVEL 2
4000 LEVEL 1	W200X52	W200X52	W200X52	W200X52	W200X52	W200X52	W200X86	W200X86	W200X86	W200X86	W200X42	W200X86	W200X52	W200X52	W200X86		4000 LEVEL 1
0																	0
Column Locations	A-4, B-4, C-4, I-4	A-5	A-6	A-7	A-8	A-9, B-9, C-9, D-9, E-9, F-9, G-9, H-9, I-9	B-5, C-5, D-5, E-5, F-5, G-5, H-5, I-5	B-6, C-6, D-6, E-6, F-6, G-6, H-6, I-6	B-7, C-7, D-7, E-7, F-7, G-7, H-7, I-7	B-8, C-8, D-8, E-8, F-8, G-8, H-8, I-8	C1-1, C1-3, D1-1, E1-1, F2-1, G1-1, H1-1, H1-2, H1-3, C1-2	C1-4, D-4, E-4, F-4, G-4, H-4, H1-4	D1-2, D1-3, E1-2, E1-3, F2-2, F2-3, G1-2, G1-3	J-4, J-9, K-4, K-5, K-6, K-7, K-8, K-9	J-5, J-6, J-7, J-8		

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COLUMN SCHEDULE

PROJECT No: 001

DATE: 20 JANUARY 2019

SCALE: –

DRAWING BY: KL & DM

APPROVED BY: KL & DM

DRAWING No

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