Building Code Clause(s)B1, B2.....................

PRODUCER STATEMENT – PS1 – DESIGN

(Guidance notes on the use of this form are printed on page 2)

ISSUED BY:.Carter Holt Harvey Limited trading as Carter Holt Harvey Woodproducts New Zealand ....
(Design Firm)

Auckland Council
16/02/2018
BCO10260425
BUILDING CONSENT

TO: Three Blokes Limited........................
(Owner/Developer)

TO BE SUPPLIED TO: Auckland Council ....
(Building Consent Authority)

IN RESPECT OF: hyFRAME Building Systems for Replacement Planing Shed...
(Description of Building Work)

AT: 362 Matakana Valley road, Matakana, Auckland ....
(Address)

.... **LOT**Part Allotment 51 Parish of Matakana and Section 1 Survey Office Plan 400726... **DP** ... **SO** ....

We have been engaged by the owner/developer referred to above to provide Design and drawings of proposed LVL Portal Frame building system and Secondary wall and roof framing components.................. services in respect of the requirements of
(Extent of Engagement)

Clause(s) B1, B2...of the Building Code for All ☐ or Part only ☒ (as specified in the attachment to this statement), of the proposed building work.

The design carried out by us has been prepared in accordance with:

☒ Compliance Documents issued by the Ministry of Business, Innovation & Employment...Acceptable Solution for Durability, in Accordance with NZS 3602 Ref No. 1D.14, All other exterior wall framing (Acceptable Solutions B2/AS1, 3.2.1(a) ....or
(verification method / acceptable solution)

☒ Alternative solution as per the attached schedule B1/VM1 for determination of Structural Design actions, with Structural Timber Design completed in accordance with AS 1720.1 Timber Structures Part 1: Design Methods as an Alternative Solution......

The proposed building work covered by this producer statement is described on the drawings titled Proposed 12.0m x 4.8m x 30.0m hyFRAME Building..and numbered HF0014 S1-S6..;
together with the specification, and other documents set out in the schedule attached to this statement.

On behalf of the Design Firm, and subject to:
(i) Site verification of the following design assumptions Foundations and walls installed to Hutchinson Consulting Engineers Ltd. Specifications
(ii) All proprietary products meeting their performance specification requirements;

I **believe on reasonable grounds** that a) the building, if constructed in accordance with the drawings, specifications, and other documents provided or listed in the attached schedule, will comply with the relevant provisions of the Building Code and that b), the persons who have undertaken the design have the necessary competency to do so. I also recommend the following level of construction monitoring/observation:
☐CM1 ☒CM2 ☐CM3 ☐CM4 ☐CM5 (Engineering Categories) or ☐ as per agreement with owner/developer (Architectural)

I, Jeremy van Diepen........................... am: ☒CPEng ...1020778..........................#
(Name of Design Professional)

☐Reg Arch ....#

I am a Member of : ☒ IPENZ ☐NZIA and hold the following qualifications:..BE (Civil)(Hons), CMEngNZ.................
The Design Firm issuing this statement holds a current policy of Professional Indemnity Insurance no less than $200,000*.
The Design Firm is a member of ACENZ: ☐

SIGNED BY Jeremy van Diepen .... ON BEHALF OF ...CHH Woodproducts New Zealand ......
(Design Firm)

Date...19/12/2017........................ (signature)...

Note: This statement shall only be relied upon by the Building Consent Authority named above. Liability under this statement accrues to the Design Firm only. The total maximum amount of damages payable arising from this statement and all other statements provided to the Building Consent Authority in relation to this building work, whether in contract, tort or otherwise (including negligence), is limited to the sum of $200,000.*

This form is to accompany **Form 2 of the Building (Forms) Regulations 2004** for the application of a Building Consent.

12.0 x 4.8 x 30.0 hyFRAME Design

Soil Class C

CHH Woodproducts

This design has been prepared to provide computations for a hyFRAME LVL portal frame system for compliance with clauses B1, Structure & B2, Durability of the New Zealand Building Code

Table of Contents

Designed by: CHH Woodproducts
December 2017

Enquires : Free call 0800 808 131

Web : www.chhwoodproducts.co.nz

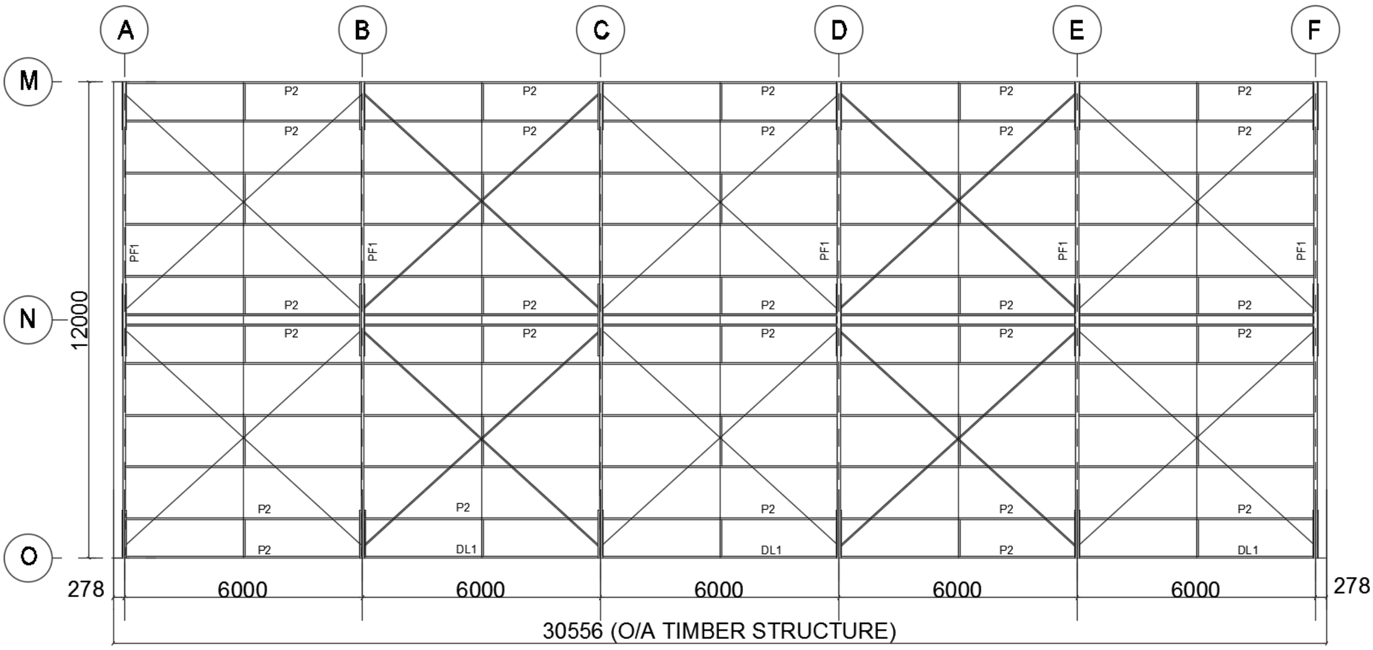
1.0 Introduction

This design has been developed as a design solution for a standard range of LVL Portal Frames developed by CHH Woodproducts NZ. The development of loadings and footings have been prepared based on standardised designs where internal pressures are considered to be $^{+}0.7$, $^{-}0.65$ and directional multipliers are equal to 1. The design has been prepared assuming the building is proposed in open terrain as described by Terrain Category 2 in Wind Region A6 or A7, and is subject to the following site information:

Building Span	12.0 m
Building length	30.0 m, consisting of 3 x 5.0 m bays
Building Clear Height	4.8 m
Dominant openings	3/ 4.8 x 4.8 m roller doors in one side wall
Cladding	Roof & Walls - Pierce fixed sheeting of weight 6.5 kg/m^2
Region	A6 or A7, v_{500} = 45 m/s, v_{20} = 37 m/s
Terrain Category	2
Directional Multipliers	1.0
Ground Snow load	not exceeding 0.9 kPa
Building Importance Level	2
Soil Class	C (to be confirmed by external consultant)

This example has been based on relevant current design standards as detailed below:

- AS/NZS 1170.0:2002 Structural design actions. Part 0: General principles
- AS/NZS 1170.1:2002 Structural design actions. Part 1: Permanent, imposed and other actions
- AS/NZS 1170.2:2011 Structural design actions. Part 2: Wind actions
- AS/NZS 1170.3:2003 Structural design actions. Part 3: Snow and ice actions
- AS/NZS 1170.5:2004 Structural design actions. Part 4: Earthquake actions – New Zealand
- AS 1720.1-2010 Timber structures. Part 1:Design Methods



Other Referenced Design Documents:

- Futurebuild LVL Specific Design Guide. November 2012
- Lumberlock Timber Connectors Characteristic Loadings Data. 08/2014
- Pryda Product Catalogue. October 2014

1.1 Frame loading

Frame loading is developed an based on light weight roofing, snow, wind and live loads where earthquake loads are such that they do not exceed the base shear experienced from terrain category wind loading. Effective load width is based on bay spacing of 6.0 m.

1.1.1 Dead loading

Assume roof sheeting mass of 6.5 kg/m² plus a miscellaneous load of 8.5 kg/m²

$$w_g^* = \frac{15.0 \times 6.0 \times 9.81}{1000} + self\_weight$$
$$\therefore w_g^* = 0.90 kN/m$$

1.1.2 Live loading

Live load of 0.25 kPa applied in accordance AS/NZS 1170.1:2002 Table 3.2.

$$w_q^* = 0.25 \times 6.0 = 1.50 kN/m$$

1.1.3 Snow loading

Ground snow load of 0.9 kPa (sub alpine) applied in accordance AS/NZS 1170.3:2003.

$$S_u = F_{SN}$$ AS/NZS 1170.3 Cl 4.2.3(a)

$$S = S_g C_e u_i$$ AS/NZS 1170.3 Eqn 4.2(1)

$$C_e = 1.0$$
$$u_i = \frac{0.7(60-\alpha)}{50}$$
$$\therefore u_i = \frac{0.7(60-15)}{50} = 0.63$$

$$w_{S_u}^* = (0.9 \times 0.63) \times 6.0 = 3.40 kN/m$$

1.1.4 Wind loading

h_{eave} = 5.26 m, h_{ridge} = 6.87 m, therefore h = 6.07 m

For Terrain Category 2.0, $m_{z,cat}$ = 0.93

$$\therefore q_u = \frac{1.2 \times 0.5 \times (0.93 \times 45)^2}{1000}$$
$$\therefore q_u = 1.06 kPa$$

Determine k_a

Area supported by rafters for lateral wind, A = (12.0x6.0) = 72.0 m², therefore k_a = 0.84

Area supported by frames for longitudinal wind, A = ((12.0+2 x 5.26)x 6.0) =135.1 m², therefore k_a = 0.8

Determine k_c

Critical load cases for wind are subject to four effective surfaces.

$k_{ce} = k_{ci} = 0.8$

Determine k_l

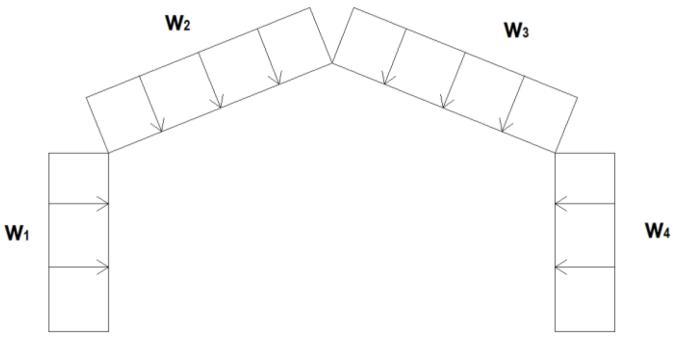
Local pressure factors are not applicable to frames.

$k_l = 1.0$

Consider Wind loading

$$w_i^* = q_u.spacing.(k_a.k_l.k_{c,e}.c_{pe} - k_{c,i}.c_{pi})$$

Lateral Wind



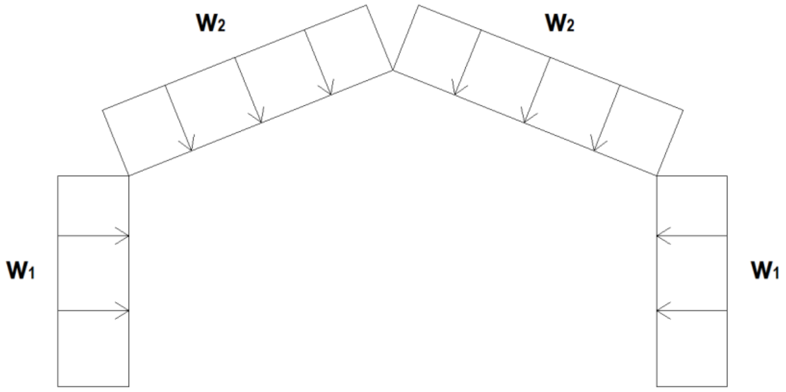
Lateral wind +cpi A

	qu (kPa)	E.S (m)	k_a	$k_{c,e}$	$k_a.k_c$	c_{pe}	$k_{c,i}$	c_{pi}	kN/m
w_1	1.06	6.00	1.00	0.8	0.80	0.7	0.8	0.70	0.00
w_2	1.06	6.00	0.84	0.8	0.80	-0.7	0.8	0.70	-7.12
w3	1.06	6.00	0.84	0.8	0.80	-0.5	0.8	0.70	-6.11
w4	1.06	6.00	1.00	0.8	0.80	-0.3	0.8	0.70	-5.09

Lateral wind -cpi A

	qu (kPa)	E.S (m)	k_a	$k_{c,e}$	$k_a.k_c$	c_{pe}	$k_{c,i}$	c_{pi}	kN/m
w_1	1.06	6.00	1.00	0.8	0.80	0.7	0.8	-0.65	6.87
w_2	1.06	6.00	0.84	0.8	0.80	-0.30	0.8	-0.65	1.78
w3	1.06	6.00	0.84	0.8	0.80	-0.5	0.8	-0.65	0.76
w4	1.06	6.00	1.00	0.8	0.80	-0.3	0.8	-0.65	1.78

Longitudinal Wind



Longitudinal wind +cpi

	kPa	E.S (m)	k_a	$k_{c,e}$	$k_a.k_c$	c_{pe}	$k_{c,i}$	c_{pi}	kN/m
w1	1.06	6.00	0.80	0.8	0.80	-0.58	0.8	0.70	-6.51
w2	1.06	6.00	0.80	0.8	0.80	-0.70	0.8	0.70	-7.12

Longitudinal wind -cpi

	kPa	E.S (m)	k_a	$k_{c,e}$	$k_a.k_c$	c_{pe}	$k_{c,i}$	c_{pi}	kN/m
w1	1.06	6.00	0.80	0.8	0.80	-0.2	0.8	-0.65	2.29
w2	1.06	6.00	0.80	0.8	0.80	0.2	0.8	-0.65	4.32

Refer to Appendix 2 for Design Action Effect Diagrams.

1.1.5 Earthquake loading

Since h ≤ 10.0 m adopt equivalent static method in accordance with AS/NZS 1170.5:2004.

Calculate Elastic Site Spectra

$$C(T) = C_h(T).Z.R.N(T,D)$$ AS/NZS 1170.5 Eqn 3.1(1)

Assume Soil Class C, and
Assume Period 0.4 s

$$\therefore C_h(T) = 2.36$$ AS/NZS 1170.5 Table 3.1

z = 0.13 (Matakana) AS/NZS 1170.5 Table 3.3

R_u = 1.0 (Return Period = 1/500 as per AS/NZS 1170.0)
R_s = 0.25 (Return Period = 1/25 as per AS/NZS 1170.0)

$N(T,D) = 1.0$ Since Annual Probability exceeding ≥ 1/250

$$C(T)_u = 2.36 \times 0.13 \times 1.0 \times 1.0$$
$$C(T)_u = 0.31$$
$$C(T)_s = 0.08$$

Calculate Horizontal Design Action Co-efficient and Design Spectra

$$C_d(T_1) = \frac{C(T_1).S_p}{k_u}$$ AS/NZS 1170.4 Eqn 5.2(1)

Calculate S_p

$$S_p = 1.3 - 0.3\mu$$ AS/NZS 1170.4 Cl 4.4.2

Assume μ=1.25 for portal frame design (nominally ductile):

$$S_p = 0.925$$ μ = 1.25

Calculate k_μ

$$k_u = \frac{(\mu - 1)T_1}{0.7} + 1$$ $T_1 < 0.7$ s

Since $T_1 = 0.4$

$$k_u = \frac{(1.25 - 1) \times 0.4}{0.7} + 1$$
$$\therefore k_u = 1.143$$

Therefore:

$$C_d(T_1)_{frames} = \frac{0.31 \times 0.925}{1.143}$$
$$\therefore C_d(T_1)_{frames} = 0.25$$

Assume μ=1.0 for longitudinal bracing (elastic):

$$S_p = 1.0$$ μ = 1.0

Calculate k_μ

$$k_u = \frac{(\mu - 1)T_1}{0.7} + 1$$ $T_1 < 0.7$ s

Since $T_1 = 0.4$

$$k_u = \frac{(1-1)\times 0.4}{0.7} + 1$$
$$\therefore k_u = 1.0$$

Therefore:

$$C_d(T_1)_{longitudinal\_bracing} = \frac{0.31\times 1.0}{1.0}$$
$$\therefore C_d(T_1)_{longitudinal\_bracing} = 0.31$$

Calculate Base Horizontal Shear

$$Base\_Horizontal\_Shear = w_g \times C_d(T_1)$$

Refer to Appendix 4 for full calculation

Lateral Load – Maximum primary and secondary framing plus cladding and mezzanine floor load

- **End Wall Frame critical – 4269 kg = 41.9 kN**
- $Base\_Horizontal\_Shear = 41.9\times 0.25$
- $\therefore Base\_Horizontal\_Shear = 10.4kN$

Longitudinal Load – Consider all primary and secondary framing plus cladding and mezzanine floor load

- **23911 kg = 234.6 kN**
- $Base\_Horizontal\_Shear = 234.6\times 0.31$
- $\therefore Base\_Horizontal\_Shear = 72.0kN$
- **Refer Section 7 for calculation of longitudinal bracing**

2.0 Purlin Design

Purlin Span	6000-75 =	5925 mm
Purlin Spacing		1350 mm (maximum)

Propose 240x45 hyFRAME for use as purlin

2.1 Dead load

Assume roof sheeting mass of 6.5 kg/m² plus a miscellaneous load of 8.5 kg/m²

$$w_g^* = \frac{15.0 \times 1.35 \times 9.81}{1000} + self\_weight$$

$$\therefore w_g^* = 0.27 kN/m$$

Serviceability

Timber components subjected to long term loads such as dead load require the consideration of creep effects. Table 2.4, AS 1720.1-2010 demonstrates the relationship between duration of load and creep. The j_2 factor is applied to elastic deflections. LVL products are considered dry at the time of supply and can be assumed to have a moisture content less than 15%. Note: NZS 3603 denoted k_2 as the creep modification factor

Section and Material Properties for the 240x45 hyFRAME have been applied, some properties may differ across markets.

$$\delta_T = j_2 . \delta$$

$$\delta = j_2 \left[\frac{5.w.l^4}{384.EI_x} \right] = 2.0 . \left[\frac{5 \times 0.27 \times 5925^4}{384 \times 476.9 \times 10^9} \right]$$

$$\therefore \delta_G = 18.2mm \quad or \quad Span/326$$

Serviceability limits for timber purlins are the same as those applied to other building products. For long term deflection of industrial purlins span/300 or 30.0 mm are deemed acceptable.

2.2 Live load

Live load of 0.35 kPa applied in accordance AS/NZS 1170.1:2002 Table 3.2.

$$w_q^* = 0.35 \times 1.35 = 0.47 kN/m$$

Serviceability

$$\delta_q = 1.0 . \left[\frac{5 \times 0.47 \times 5925^4}{384 \times 476.9 \times 10^9} \right]$$

$$\therefore \delta_q = 15.8mm \quad or \quad Span/375$$

Strength

Based on respective k_1 and load combination factors, combined dead and live load design actions will always be more critical for design than permanent loads where low roof masses (less than 20 kg/m²) are applied.

$$w^*_{1.2G+1.5Q} = 1.2 \times 0.27 + 1.5 \times 0.47$$
$$\therefore w^*_{1.2G+1.5Q} = 1.03 kN/m$$

$$M^*_{1.2G+1.5Q} = \frac{w.l^2}{8} = \frac{1.03 \times 5.9^2}{8}$$
$$\therefore M^*_{1.2G+1.5Q} = 4.48 kNm$$

2.3 Snow load

Ground snow load of 0.9 kPa (sub alpine) applied in accordance AS/NZS 1170.3:2003.

$$S_u = F_{SN}$$ AS/NZS 1170.3 Cl 4.2.3(a)

$$S = S_g C_e u_i$$ AS/NZS 1170.3 Eqn 4.2(1)

$$C_e = 1.0$$
$$u_i = \frac{0.7(60-\alpha)}{50}$$
$$\therefore u_i = \frac{0.7(60-15)}{50} = 0.63$$

$$w^*_{1.2G+S_u+\varphi Q} = 1.2 \times 0.27 + (0.9 \times 0.63) \times 1.35 + 0 \times 0.25 = 1.09 kN/m$$

$$M^*_{1.2G+S_u+\varphi Q} = \frac{w.l^2}{8} = \frac{1.09 \times 5.9^2}{8}$$
$$\therefore M^*_{1.2G+S_u+\varphi Q} = 4.74 kNm$$

Serviceability

$$\delta_q = 1.27 \left[\frac{5 \times 0.52 \times 5925^4}{384 \times 476.9 \times 10^9} \right]$$
$$\therefore \delta_q = 22.2mm \quad or \quad Span/266$$

Strength

Check Bending Capacity

Purlin design assumes the use of pierce fixed roof sheeting providing continuous lateral restraint to the top of the purlin. Since the top of the purlin is compression edge it is considered to be fully restrained k_{12}=1.0. *(Note: NZS 3603 denoted k_8 as the Stability factor).*

So for bending about XX axis
Calculate Bending Moment Capacity

$M_d \geq M^*$ — Eq. 3.2(1), AS 1720.1

$M_d = \phi.k_1.k_4.k_6.k_9.k_{12}.f_b^{'}.Z$ — Eq. 3.2(2), AS 1720.1

For solid section Futurebuild LVL members with depths greater than 95 mm, apply size factor of $\left(\frac{95}{D}\right)^{0.1538}$. For further information refer Futurebuild LVL Specific Design Guide.

where:

The selection of the capacity factor (termed strength reduction factor in NZ) for purlins can be based on Application Category 1, Secondary structural members in structures other than houses, for LVL apply Ø=0.95.

$$Ø = 0.95 \quad k_4 = k_6 = k_9 = 1.0 \quad f_b^{'} = 28.4.\left(\frac{95}{D}\right)^{0.1538} MPa$$

$$Z = \frac{d^2.b}{6} = \frac{240^2 \times 45}{6}$$

$$\therefore Z = 432.0 \times 10^3 mm^3$$

$$M_d = 0.95 \times k_1 \times 1.0 \times 1.0 \times 1.0 \times k_{12} \times \left(\frac{95}{240}\right)^{0.1538} \times 28.4 \times 432.0 \times 10^3$$

$$\therefore M_d = 10.1.k_1.k_{12} kNm$$

Duration of load factor for purlins is selected based on live load and snow load in Sub-Alpine area's not exceeding 5 days, k_1 =0.94

$$M_d = 9.50 kNm > M^*$$

Calculate Shear Capacity

$V_d \geq V*$ — Eq. 3.2(13), AS 1720.1 *(Eq. 3.3, NZS 3603 Similar)*

$V_d = \phi.k_1.k_4.k_6.f_s^{'}A_S$ — Eq. 3.2(14), AS 1720.1 *(Eq. 3.4 NZS 3603 Similar)*

where:

$$Ø = 0.95 \qquad k_1 = 0.94$$
$$k_4 = k_6 = 1.0 \quad f_s = 4.6 MPa$$

$$A_S = 2.b.d/3$$

$$\therefore A_S = \frac{2}{3}.240 \times 45 = 7200mm^2$$

$$\therefore V_d = 0.95 \times 0.94 \times 1.0 \times 4.6 \times 7200$$

$$\therefore V_d = 29.6kN > V^*$$

2.4 Wind loading

Note: Purlins have been designed for wind loading to match the highest and widest of the hyFRAME range, a 15.0 m span portal frame with a 4.8 m clear height. These purlins are detailed as such and the wind loads may be different to those applied in the frame analysis.

Design Parameters for purlin design (15.0x4.8 clear portal frame)

h_{eave} = 5.26 m, h_{ridge} = 7.27 m, therefore h = 6.27 m

For Terrain Category 2.0, $m_{z,cat}$ = 0.93

$$\therefore q_u = \frac{1.2 \times 0.5 \times (0.93 \times 45)^2}{1000}$$

$$\therefore q_u = 1.06kPa$$

θ = 0°, Lateral wind critical (by inspection)

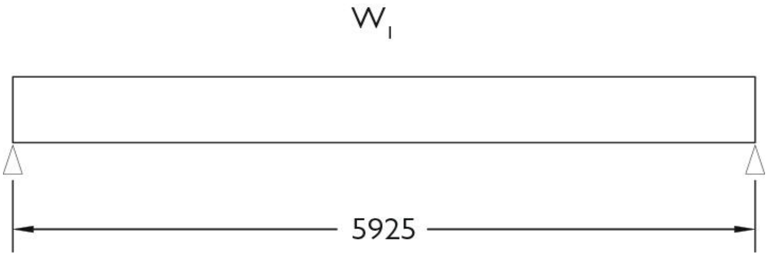
a = min(0.2b, 0.2d, h) = 3.0m

Area supported by purlins, A = (5.925x1.35) = 8.0 m² < 10 m², therefore k_a = 1.0

$k_{ce} = k_{ci} = 0.9$ Subject to two effective surfaces
$C_{pe} = -0.63$
$C_{pi} = +0.7$ Conservative

For $k_l = 1.5$

Area x $k_l = a^2 = 9.0$ m², within 0.5xa from the windward edge, hence apply k_l over whole purlin length



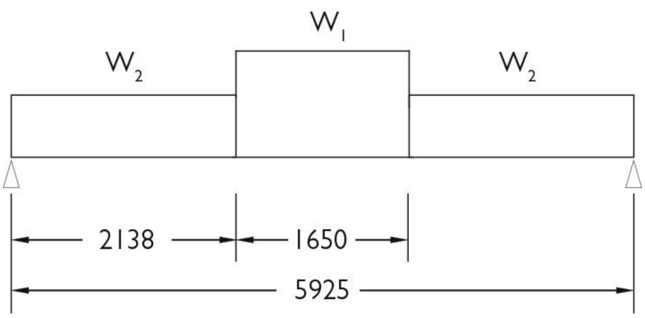
$$w_i^* = q_u.spacing.(k_a.k_l.k_{c,e}.c_{pe} - k_{c,i}.c_{pi})$$
$$w_1^* = 1.06 \times 1.35 \times (1.0 \times 1.5 \times 0.9 \times^{-} 0.63 - 0.9 \times^{+} 0.7)$$
$$\therefore w_1^* = -2.12\, kN/m$$
$$R^* = -2.12 \times 5.93/2$$
$$\therefore R^* = -6.29 kN$$

For k_l = 2.0

Area x k_l = 0.25a^2 = 2.25m^2

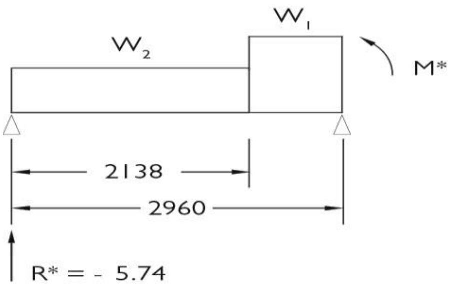
Since purlin spacing is 1.35 m

$\therefore$ apply over 2.25/1.35 = 1.65 m.



$$w_{i=}^* = q_u.spacing.(k_a.k_l.k_{c,e}.c_{pe} - k_{c,i}.c_{pi})$$
$$w_1^* = 1.06 \times 1.35 \times (1.0 \times 2.0 \times 0.9 \times^{-} 0.63 - 0.9 \times^{+} 0.7)$$
$$\therefore w_1^* = -2.52 kN/m$$
$$w_2^* = 1.06 \times 1.35 \times (1.0 \times 1.0 \times 0.9 \times^{-} 0.63 - 0.9 \times^{+} 0.7)$$
$$\therefore w_2^* = -1.71 kN/m$$

Calculate w_{eff}



Calculate Reactions

$$R^* = (^{-}2.52 \times 1.65 +^{-} 1.71 \times 2.14 \times 2)/2$$
$$\therefore R^* =^{-} 5.74 kN$$

Calculate Moment

$$M^* =^{-} 5.74 \times 2.96 - \left(^{-}2.52 \times 0.825^2/2 +^{-} 1.71 \times 2.14 \times 1.89\right)$$
$$\therefore M^* = -9.22 kNm$$

Calculate w_{eff}

$$w_{eff}^* = \frac{^{-}9.22 \times 8}{5.93^2}$$
$$\therefore w_{eff}^* =^{-} 2.10 kN/m$$

θ = 90˚, Longitudinal wind,

From previous, a=3.0 m

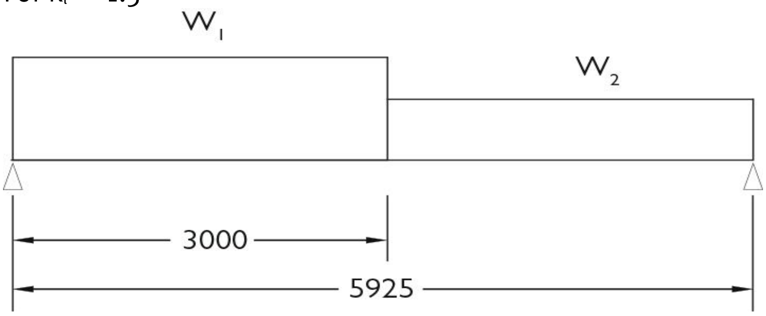
k_{ce} = k_{ci} = 0.9
C_{pe} = -0.63

Subject to two effective surfaces

$C_{pi} = +0.7$ Conservative

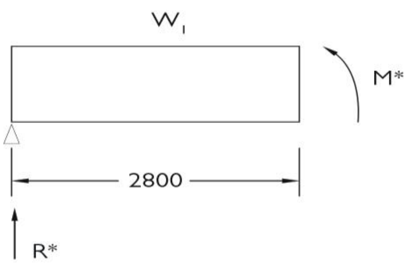
For k_l (For suction case)= 1.5 or 2.0 (See below)

For k_l = 1.5



$$w_{i=}^* = q_u.spacing.(k_a.k_l.k_{c,e}.c_{pe} - k_{c,i}.c_{pi})$$
$$w_1^* = 1.06 \times 1.35 \times (1.0 \times 1.5 \times 0.9 \times {}^- 0.9 - 0.9 \times {}^+ 0.7)$$
$$\therefore w_1^* = {}^- 2.64kN$$
$$w_2^* = 1.06 \times 1.35 \times (1.0 \times 1.0 \times 0.9 \times {}^- 0.9 - 0.9 \times {}^+ 0.7)$$
$$\therefore w_2^* = {}^- 2.06kN$$

Calculate Reactions



$$R^* = ({}^- 2.06 \times 2.93^2 / 2 + {}^- 2.64 \times 3.0 \times 4.43) / 5.93$$
$$\therefore R^* = {}^- 7.40kN$$

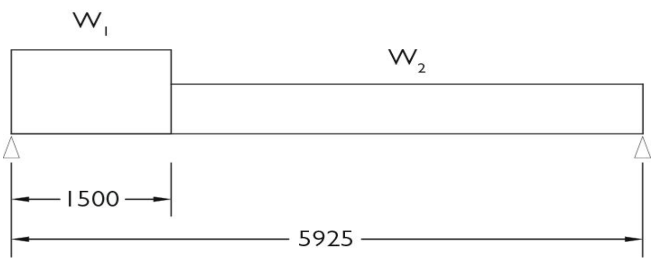
Calculate Moment

$$M^* = {}^- 7.40 \times 2.80 - {}^- 2.64 \times 2.80^2 / 2$$
$$\therefore M^* = {}^- 10.37kNm$$

Calculate w_{eff}

$$w_{eff} = \frac{{}^- 10.37 \times 8}{5.93^2}$$
$$\therefore w_{eff} = {}^- 2.36kN/m$$

Consider k_l = 2.0

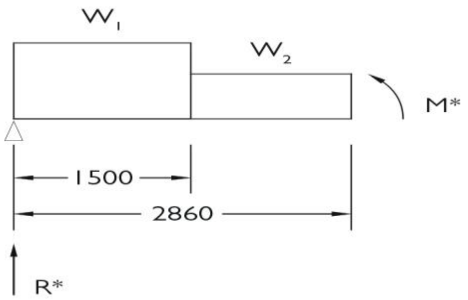


$$w_{i=}^{*} = q_u.spacing.(k_a.k_l.k_{c,e}.c_{pe} - k_{c,i}.c_{pi})$$

$$w_1^{*} = 1.06 \times 1.35 \times (1.0 \times 2.0 \times 0.9 \times {}^{-}0.9 - 0.9 \times {}^{+}0.7)$$

$$\therefore w_1^{*} = {}^{-}3.22 kN/m$$

$$w_2^{*} = {}^{-}2.06 kN/m$$



Calculate Reactions

$$R^{*} = ({}^{-}2.06 \times 4.43^2/2 + {}^{-}3.22 \times 1.5 \times 5.18)/5.93$$

$$\therefore R^{*} = {}^{-}7.63 kN$$

Calculate Moment

$$M^{*} = {}^{-}7.63 \times 2.86 + {}^{-}2.06 \times 1.36^2/2 + {}^{-}3.22 \times 1.5 \times 2.11$$

$$\therefore M^{*} = {}^{-}9.73 kNm$$

Calculate w_{eff}

$$w_{eff} = \frac{{}^{-}9.73 \times 8}{5.93^2}$$

$$\therefore w_{eff} = {}^{-}2.21 kN/m$$

Consider Maximum Positive loading

$$w_{+ev} = 1.06 \times 1.35 \times (1.0 \times 1.0 \times 0.9 \times {}^{+}0.2 - 0.9 \times {}^{-}0.65)$$

$$\therefore w_{+ev} = {}^{+}1.10 kN/m$$

$$R^{*} = 1.10 \times 5.93/2$$

$$\therefore R^{*} = 3.26 kN$$

Critical design wind loading, w_{wu} = ${}^{-}$2.36 kN/m, ${}^{+}$1.10 kN/m, R^{*}=8.13 kN

Serviceability

To obtain the serviceability wind load the ultimate uniform loads can be factored by the square of the ratio serviceability wind speed to ultimate wind speed.

$$w_s = \left(\frac{v_s}{v_u}\right)^2 \times w_s$$

$$w_s = \left(\frac{37}{45}\right)^2 \times {}^{-}2.36 = {}^{-}1.60 kN/m$$

$$\delta_w = 1.0\left[\frac{5 \times {}^{-}1.60 \times 5925^4}{384 \times 476.9 \times 10^9}\right]$$

$$\therefore \delta_w = 53.7mm \quad or \quad Span/110$$

On the basis of applied local pressure factors and the instantaneous nature of the wind gust span/100 is deemed acceptable.

Strength

Check Capacity

Consider negative wind pressures.

Calculate w_g

Assume roof sheeting mass of 6.5 kg/m² (do not apply miscellaneous load for uplift)

$$w_g^* = \frac{6.5 \times 1.35 \times 9.81}{1000} + self\_weight$$
$$\therefore w_g^* = 0.15 kN/m$$

$$w_{0.9G+w_u} = 0.9 \times w_g + w_u$$
$$w_{0.9G+w_u} = 0.9 \times 0.15 + {}^{-}2.36$$
$$w_{0.9G+w_u} = {}^{-}2.22 kN/m$$

$$M^* = \frac{w.l^2}{8} = \frac{{}^{-}2.22 \times 5.9^2}{8}$$
$$\therefore M^* = {}^{-}9.66 kNm$$

Calculate bending moment capacity

From previous:

$$M_d = 10.1.k_{12} kNm$$

Calculate k_{12}

Continuous lateral restraint is provided to the tension edge via pierce fixed sheeting.

Calculate S_1

$$S_1 = 2.25.\frac{d}{b}$$ Eq. 3.2(13), AS 1720.1

$$S_1 = 2.25 \times \frac{240}{45} = 12.0$$

Calculate p

The material constant, p, takes into account the material effects in the determination of the slenderness co-efficient whilst the slenderness coefficient, S, takes into account the geometrical parameters. The ratio 'r' is a function of the temporary design action effect divided by the total design action effect

$$\rho_b = 14.71\left(\frac{E}{f_b^{'}}\right)^{-0.408} r^{-0.061}$$ Eq. 8(1), AS 1720.1

where:

$$E = 9200MPa$$
$$f_b^{'} = 28.4\left(\frac{95}{D}\right)^{0.1538}$$
$$r = 1$$
$$\rho_b = 14.71\left(\frac{9200}{24.6}\right)^{-0.480} 1.0^{-0.061}$$
$$\therefore \rho_b = 0.85$$
$$\Rightarrow \rho_b S_1 = 10.3$$

Since $10 \leq \rho_b S \leq 20$

$$k_{12} = 1.5 - 0.05 \times \rho_b S_1$$ Eq. 3.2(11), AS 1720.1

$$k_{12} = 1.5 - 0.05 \times 10.3 = 0.98$$

$$\therefore M_d = 10.1 \times 0.98 = 9.95kNm > M^*$$

Calculate Shear Capacity

$$V_d \geq V^*$$ Eq. 3.2(13), AS 1720.1
(Eq. 3.3, NZS 3603 Similar)

$$V_d = \phi.k_1.k_4.k_6.f_s^{'}A_S$$ Eq. 3.2(14), AS 1720.1
(Eq. 3.4 NZS 3603 Similar)

where:

$$Ø = 0.95 \qquad k_1 = 1.0$$
$$k_4 = k_6 = 1.0 \qquad f_s = 4.6MPa$$ Futurebuild Specific Design Guide
$$A_S = 2.b.d/3$$
$$\therefore A_S = \frac{2}{3}.240 \times 45 = 7200mm^2$$

$$\therefore V_d = 0.95 \times 1.0 \times 1.0 \times 4.6 \times 7200$$
$$\therefore V_d = 31.5kN > V^*$$

2.5 Connection design

Propose JH47x190 to suit 240x45 hyFRAME. It is typical to apply a practical minimum number of nails for bracket and beam stability, for members around 240 mm deep we recommend a minimum of 6/12gx35 type17 screws i.e. 3/12g screws per tab.

Calculate minimum number of 12g type 17 Hex Head screws

Type 17 screws are preferred for timber connection as they are a self drilling screw through the timber.

$N_{d,j} \geq N*$ — Eq. 4.3(1), AS 1720.1 *(Eq. 4.5, NZS 3603 Similar)*

$N_{d,j} = \phi.k_1.k_{13}.k_{14}.k_{16}.k_{17}.n.Q_k$ — Eq. 4.3(2), AS 1720.1 *(Eq. 4.6, NZS 3603 Similar)*

where:

$\varnothing = 0.85 \quad Q_k = 2.153kN$ — AS 1720.1

$k_1 = 1.14$ *(critical load case is wind uplift, effective duration of peak action 5 seconds)*
$k_{13} = 1.0$ *(screws into side grain)*
$k_{14} = 1.0$ *(screws in single shear)*
$k_{16} = 1.0$ *(screws through LVL)*
$k_{17} = 1.0$ *(number of fasteners ≤ 4)*

From previous $V_d = 8.13kN$
$N_{d,j} = 0.85 \times 1.14 \times 1.0 \times 1.0 \times 1.0 \times 1.0 \times 6 \times 2.153$
$\therefore N_{d,j} = 12.52kN$

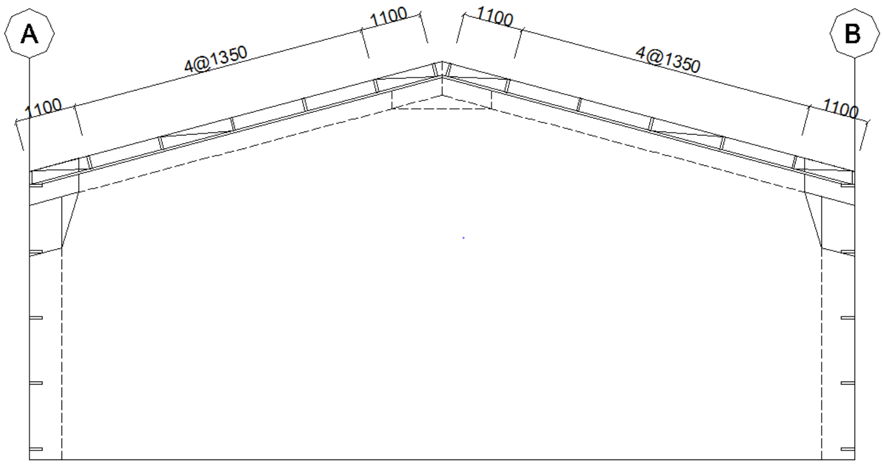
Say 6/12gx35 type 17 Hex Head screws, screwed through JH47190 joist hanger.

Also confirm Characteristic Strength Capacity of Bracket.

$\varnothing Q = 18.0kN > N^*$ — Mitek Literature

The 240x45 hyFRAME purlin to span 5.9 m at maximum 1350 mm spacing is adequate to support the design load.

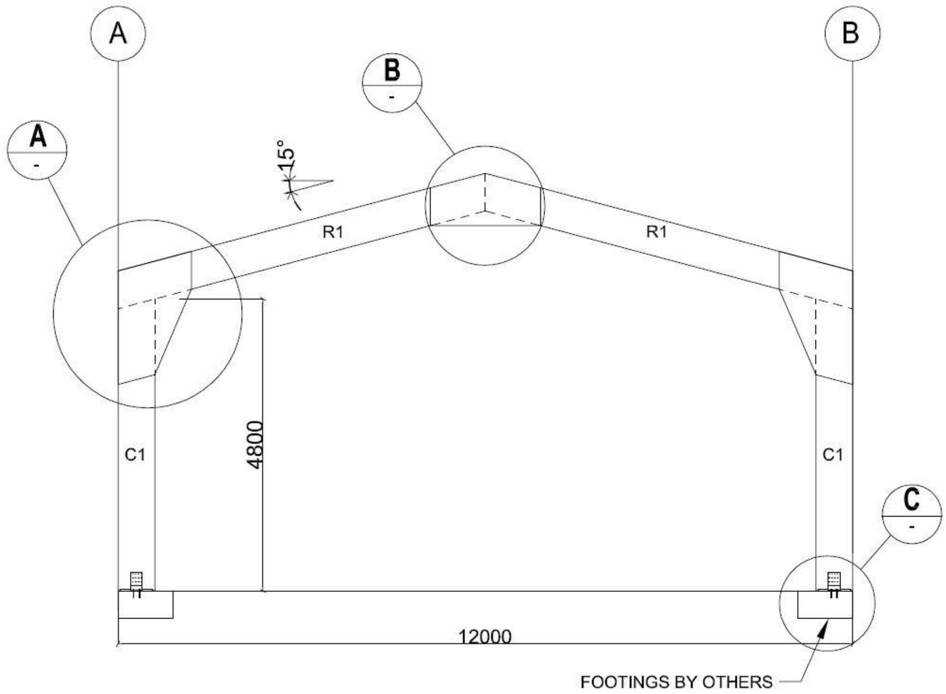
2.6 Proposed Purlin Layout


A
B
1100
4@1350
1100
1100
4@1350
1100

3.0 Portal Frame Design

The following portal frame has been analysed using elastic structural analysis with Microstran. For solid timber a five percent allowance for shear deflection is included in the average modulus of elasticity which removes any need for the separate consideration of shear deflection.

3.1 Proposed Portal Frame



3.2 Serviceability

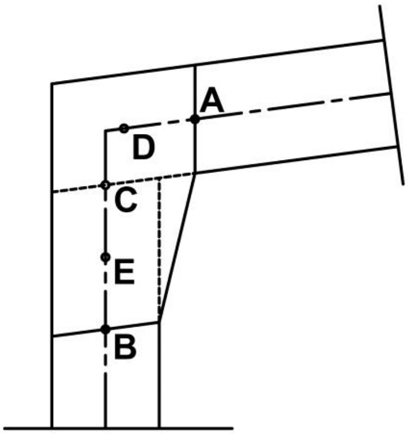
Serviceability design limits for timber and steel buildings are very similar where the consideration of cladding and absolute clearances need to be taken into account in the relative stiffness of the frame. Short term duration of loading for wind, live and earthquake loads may be calculated by applying a duration of load factor of 1, hence using the elastic deflection directly from analysis packages. For long term loads the effects of creep need to be taken into account. AS1720.1 Figure 2.1 defines j_2 *(NZS 3603 Table 2, k_2)* as 2.0 for loading of one year or more where the moisture content is less than 15%.

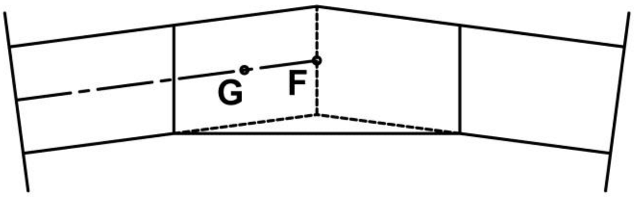
Serviceability – 600x75 hyFRAME portal frame

Load Case	j_2	Deflection	
		Vertical	Horizontal
Dead load	2.0	11.4 mm or span/1000	3.0 mm or height/1700
Live load	0.7	5.1 mm or span/2230	1.3 mm or height/3800
Snow load	1.27	14.3 mm or span/800	3.7 mm or height/1360
Seismic Load	1.0	n/a	7.3 mm or height/690
Wind loading			
Lateral wind$_1$	1.0	16.8 mm or span/680	30.7 mm or height/160
Lateral wind$_2$	1.0	1.3 mm or span/8780	36.1 mm or height/140
Longitudinal wind$_1$	1.0	13.6 mm or span/830	3.3 mm or height/1530
Longitudinal wind$_2$	1.0	10.2 mm or span/1110	2.5 mm or height/1970

3.3 Strength

The selection of design moments is important in the design of timber portal frames. The nature of the interaction of gussets provide specific locations for the selection of critical design actions for the design of rafters, columns gussets and nail rings. Hutchings and Bier [2000] provide guidance on the design moment locations as shown below.





Location A – Rafter design actions at knee
Location B – Column design actions
Location C – Knee gusset design actions
Location D – Gusset to rafter at knee connection actions
Location E – Gusset to column connection actions
Location F – Ridge gusset design actions
Location G – Ridge gusset to rafter design actions

A further check along the rafter is require where the critical design actions may not to be at the gusseted location and should be taken as the maximum along the rafter.

3.4 Rafter and Column Design

A check of the capacity of main frame members of a timber portal frame involves a check of combined bending and buckling action, both in plane and out of plane, and a check of combined bending and tension.

This analysis has been completed using the computeIT toolkIT Structural Analysis program. The resulting critical design action effects have been selected based on the above criteria and been combined in accordance with the requirements AS/NZS 1170.0 and AS 1720.1, where the slenderness co-efficient has been applied based on discrete restraint to the tension edge. The analysis and results can be viewed in Appendix 2.

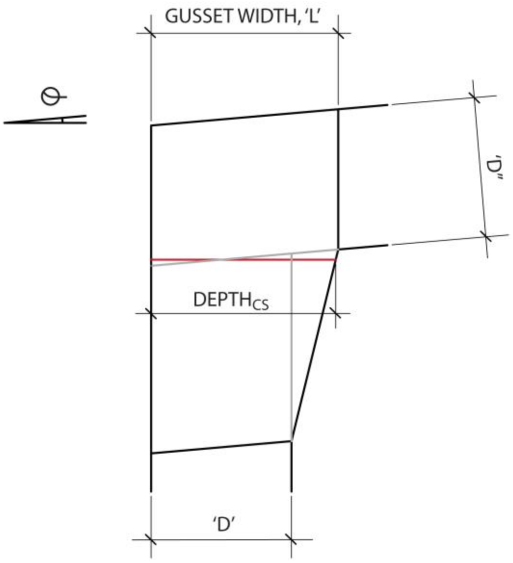
Use 600x75 hyFRAME as column and rafter as detailed above.

3.5 Gusset Design

The knee and ridge connections of an LVL portal frame can be completed by using a plywood gusset. Plywood gussets allow an ease of fabrication and can be readily fixed using machine driven nails. Plywood or minimum 4 x-band gussets are recommended for use in heavily nailed rigid connections because the x-band plies help reduce the tendency of the long band plies to split. This allows the nail spacing to be governed by the grain direction of the rafter or column which ever the gusset is being fastened to.

3.5.1 Knee gusset design

The capacity of a plywood gusset is based on the critical depth at which the gusset bends, which is a horizontal line across the centroid of the rafter and column intersection as shown below.

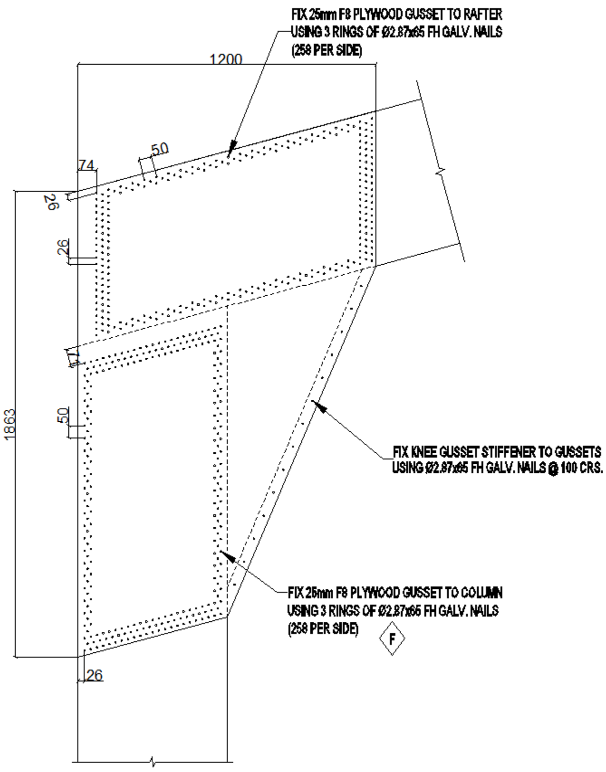


Geometrically, assuming the rafter depth and column depth are equal, the critical section for the knee connection may be calculated by:

$$Depth_{cs} = D + \frac{L - D}{1 + \left(1 - \frac{D}{2L}\right)\tan\theta}$$

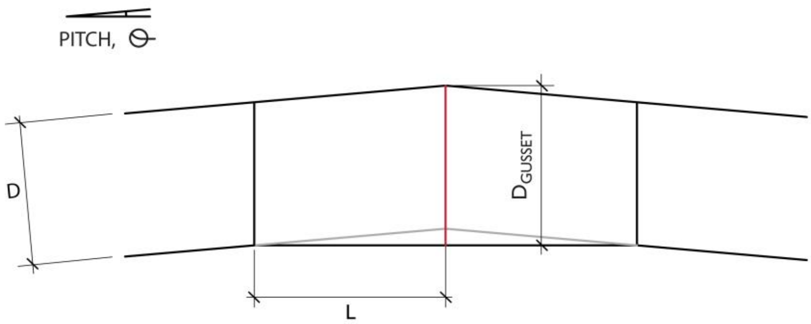
The analysis of the knee gussets has been completed using the computeIT toolkIT Structural Analysis program. The resulting critical design action effects have been entered and analysed for load combinations in accordance with the requirements AS/NZS 1170.0 and AS 1720.1 The analysis and results can be viewed in Appendix 2.

Use 25 mm F8 Ecoply Plywood gussets shaped and detailed as per below.



3.5.2 Ridge Gusset Design

The design of the ridge gusset is similar to the knee gusset where the design capacity is based on the moment resistance offered by the ridge gusset section. Typically a mitre type joint is considered. Hutchings [1989] proposes a 0.9 factor be applied to the critical section as defined below.

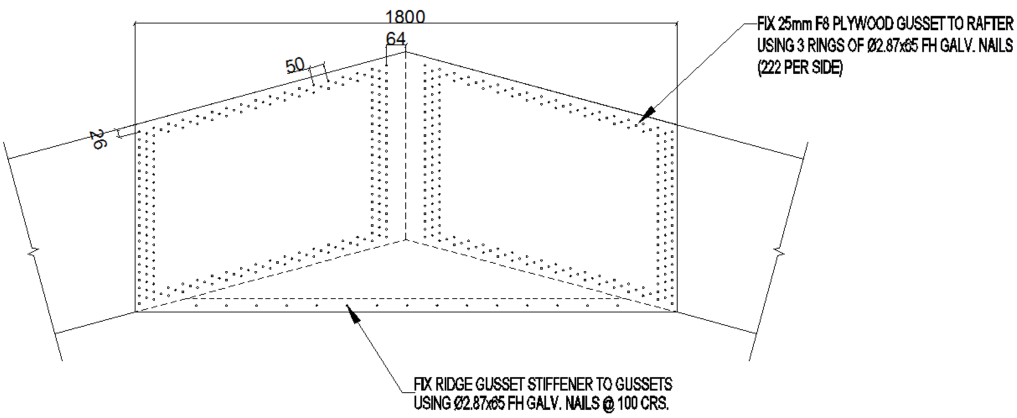


$$D_{gusset} = \frac{D}{Cos\theta} + L.\tan\theta$$

$$Depth_{cs} = 0.9.D_{gusset}$$

The analysis of the knee gussets has been completed using the computeIT toolkIT Structural Analysis program. The resulting critical design action effects have been entered and analysed for load combinations in accordance with the requirements AS/NZS 1170.0 and AS 1720.1. The analysis and results can be viewed in Appendix 2.

Use 25 mm F8 Ecoply Plywood gussets shaped and detailed as per below.



3.5.3 Nail ring design

The design of nail groups associated with rigid moment connections are often subjected to combined actions including bending, axial and shear forces. Whilst the bending and axial forces contributions are minor they need to be taken into account. It is normally most efficient to calculate the proportion of force remaining in the nails after the contribution to the design moment affect is taken out.

The complexity of calculations for the nail ring mean hand calculations can be time consuming and conservative. For this reason computer packages are often employed to develop design solutions. The design methodology, including k factors, from AS1720.1 has been applied to create nail ring capacities for a number of section sizes and gusset widths.

3.5.3.1 Knee nail ring design

The analysis of the nail ring for the knee gussets has been completed using the computeIT toolkIT Structural Analysis program. The resulting critical design action effects have been entered and analysed for load combinations in accordance with the requirements AS/NZS 1170.0 and AS 1720.1 The analysis and results can be viewed in Appendix 2.

Use 3 nail rings of 2.87x65 FH to pattern marked as detailed above.

3.5.3.2 Ridge nail ring design

The analysis of the nail ring for the knee gussets has been completed using the computeIT toolkIT Structural Analysis program. The resulting critical design action effects have been entered and analysed for load combinations in accordance with the requirements AS/NZS 1170.0 and AS 1720.1. The analysis and results can be viewed in Appendix 2.

Use 3 nail rings of 2.87x65 FH to pattern marked as detailed above

3.6 Column to footing connection design

Connection of portal frame columns to footings can be achieved by base brackets that are suitably sized and fixed directly to the LVL columns. A similar design philosophy is applied to the design and specification of hold down anchors and base plates as would normally be applied to steel where the buckling of the plate under tension needs to be considered.

Screws are ideal for most base bracket connections due to their ease of application, where the screw pattern is staggered for both sides of the column so that splitting of the LVL does not occur.

Consider Design Reactions

		PF1			
Load Case	k_1	R_x kN	R_y kN	$(R_x^2+R_y^2)^{0.5}$ kN	Angle
1.35G	0.57	2.59	11.21	11.51	77.0
1.2G+1.5Q	0.77	5.99	23.24	24.00	75.5
G+Eu (Lateral)	1.00	-7.12	13.59	15.34	62.3
0.9G+Wu (Lateral)	1.00	-13.52	-36.53	38.95	69.7
1.2G+Wu (Lateral)	1.00	-20.98	22.19	30.54	46.6
G+Eu (Longitudinal)	1.00	-2.14	-14.06	14.22	81.4
0.9G+Wu (Long)	1.00	8.41	-42.02	42.85	78.7
1.2G+Wu (Long)	1.00	2.72	34.59	34.70	85.5

k1 adjusted values

Load Case	k_1	PF1 R_x kN	R_y kN	$(R_x^2+R_y^2)^{0.5}$ kN	Angle
1.35G	0.57	4.54	19.67	20.18	77.0
1.2G+1.5Q	0.77	7.78	30.18	31.17	75.5
G+Eu (Lateral)	1.00	-7.12	13.59	15.34	62.3
0.9G+Wu (Lateral)	1.00	-13.52	-36.53	38.95	69.7
1.2G+Wu (Lateral)	1.00	-20.98	22.19	30.54	46.6
G+Eu (Longitudinal)	1.00	-2.14	-14.06	14.22	81.4
0.9G+Wu (Long)	1.00	8.41	-42.02	42.85	78.7
1.2G+Wu (Long)	1.00	2.72	34.59	34.70	85.5

It is typical in Timber structures to provide a moisture barrier at the base of the columns to eliminate the column from getting wet and staying wet during the construction period. This can be typically achieved by using H3.2 treated Plywood and melthoid at both the LVL column end and ground as detailed in the structural drawings. Downwards loads may be considered to be taken out in bearing so for the design of connections only uplift loads need be considered.

Calculate minimum number of 14g type 17 Hex Head screws

Type 17 screws are preferred for timber connection as they are a self drilling screws through the timber.

$N_{d,j} \geq N*$ Eq. 4.3(1), AS 1720.1 *(Eq. 4.5, NZS 3603 Similar)*

$N_{d,j} = \phi.k_1.k_{13}.k_{14}.k_{16}.k_{17}.n.Q_k$ Eq. 4.3(2), AS 1720.1 *(Eq. 4.6, NZS 3603 Similar)*

where:

$\varnothing = 0.8 \quad Q_k = 3.130kN$ AS 1720.1

$k_1 = 1.14$ *(critical load case is wind uplift, effective duration of peak action 5 seconds)*

$k_{13} = 1.0$ *(screws into side grain)*

$k_{14} = 1.0$ *(screws in single shear)*

$k_{16} = 1.0$ *(screws through LVL)*

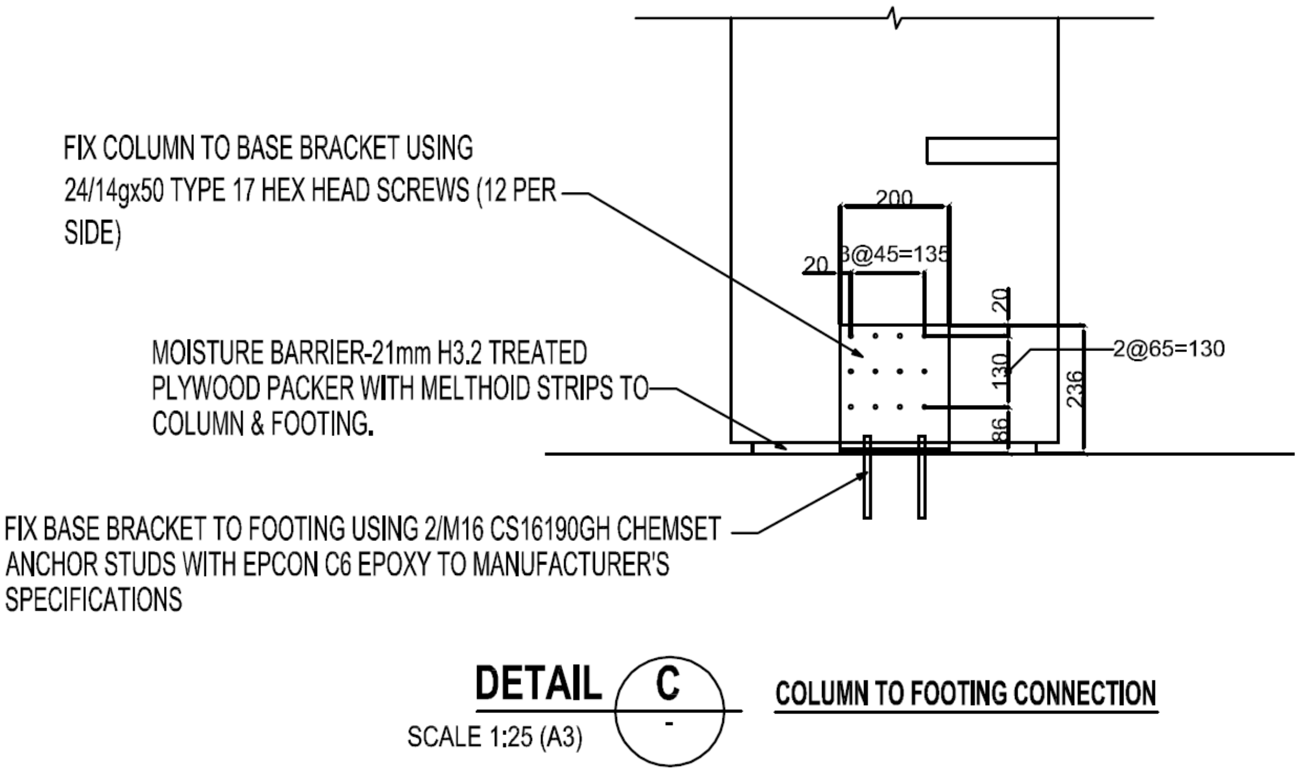
$k_{17} = 1.0$ *(number of fasteners ≤ 4) - conservative*

From previous $N^* = 42.85kN$ (downward forces are transferred directly in bearing)

$$N_{d,j} = 0.8 \times 1.0 \times 1.0 \times 1.0 \times 1.0 \times 1.0 \times 24 \times 3.130$$
$$\therefore N_{d,j} = 60.1kN$$

Say 24/14gx50 type 17 Hex Head screws, screwed through base plate sides into column.

Proposed Connection



Refer Appendix 5 for bracket and hold-down design

4.0 Girt Design

Girt Span 6000-75 = 5925mm
Girt Spacing (maximum) = 1400mm

Propose 240x45 hyFRAME.

4.1 Wind loading

The capacity of solid timber girts is also dependant on the nature of lateral torsional buckling restraint and the critical edge to which the loading and restraint is provided. It is therefore important to consider both positive and negative wind pressures.

Note: Girts have been designed for wind loading to match the highest and widest of the hyFRAME range, a 15.0 m span portal frame with a 4.8 m clear height. These girts are detailed as such and the wind loads may be different to those applied in the frame analysis.

Design Parameters for girt design (15.0x4.8 clear portal frame)

h_{eave} = 5.26 m, h_{ridge} = 7.27 m, therefore h = 6.27 m

For Terrain Category 2.0, $m_{z,cat}$ = 0.93

$$\therefore q_u = \frac{1.2 \times 0.5 \times (0.93 \times 45)^2}{1000}$$
$$\therefore q_u = 1.06kPa$$

Consider worst affected girt:

<u>Worst +ve pressure case:</u>

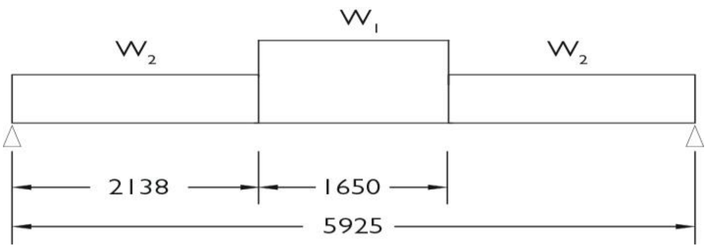
a = min(0.2b, 0.2d, h) = 3.0m

Area supported by girt, A = (5.925x1.35) = 8.0 m² < 10 m², therefore k_a = 1.0

$k_{ce} = k_{ci}$ = 0.9 Subject to two effective surfaces
$C_{pe} = {}^{+}0.7$
C_{pi} = -0.65 Conservative

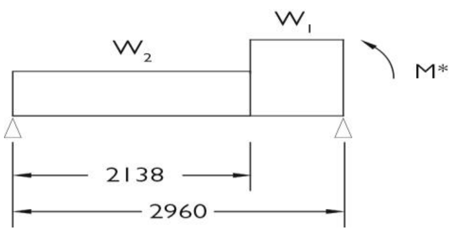
For k_l = 1.5

Area x $k_l = 0.25a^2$ = 2.25 m², within 0.5xa from the windward edge. Girt spacing is 1.4 m ∴ apply over 2.25/1.4 = 1.6 m length.



$$w_{i=}^{*} = q_u.spacing.(k_a.k_l.k_{c,e}.c_{pe} - k_{c,i}.c_{pi})$$
$$w_1^{*} = 1.06 \times 1.4 \times (1.0 \times 1.5 \times 0.9 \times^{+} 0.7 - 0.9 \times^{-} 0.65)$$
$$\therefore w_1^{*} =^{+} 2.27 kN/m$$
$$w_2^{*} = 1.06 \times 1.4 \times (1.0 \times 1.0 \times 0.9 \times^{+} 0.7 - 0.9 \times^{-} 0.65)$$
$$\therefore w_2^{*} =^{+} 1.80 kN/m$$

For Critical M*:



Calculate Reaction:
$$R^{*} = (^{+}2.27 \times 1.6 +^{+} 1.80 \times 2.16 \times 2)/2$$
$$\therefore R^{*} =^{+} 5.71 kN$$

Calculate w_{eff}

$$M^{*} =^{+} 5.71 \times 2.96 -^{+} 2.27 \times 0.8^2/2 -^{+} 1.80 \times 2.16 \times 1.88$$
$$\therefore M^{*} =^{+} 8.88 kNm$$
$$w_{eff} =^{+} 8.88 \times 8/5.93^2$$
$$\therefore w_{eff} =^{+} 2.02 kN/m$$

Critical reaction (+ve case) when:

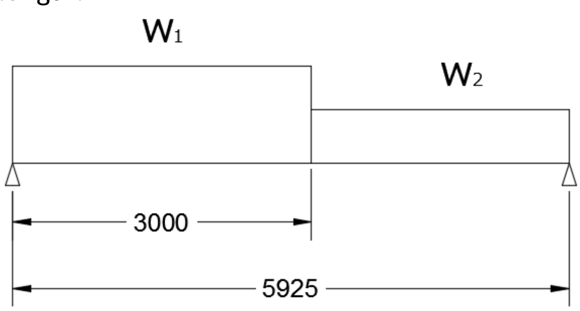
$$R^{*} = (^{+}1.80 \times 4.33^2/2 +^{+} 2.27 \times 1.60 \times 5.13)/5.93$$
$$\therefore R^{*} = 5.99 kN$$

<u>Worst -ve pressure case:</u>

$k_{ce} = k_{ci} = 0.9$ Subject to two effective surfaces
$C_{pe} = -0.65$
$C_{pi} = {}^{+}0.7$ Conservative

For $k_l = 1.5$

Area x $k_l = a^2 = 9.0\ m^2$, within a (3 m) from the windward edge, hence apply local pressure factor over 3 m length:



$$w_{i=}^{*} = q_u.spacing.(k_a.k_l.k_{c,e}.c_{pe} - k_{c,i}.c_{pi})$$
$$w_1^{*} = 1.06 \times 1.4 \times (1.0 \times 1.5 \times 0.9 \times {}^{-}0.65 - 0.9 \times {}^{+}0.7)$$
$$\therefore w_1^{*} = {}^{-}2.24kN/m$$
$$w_2^{*} = 1.06 \times 1.4 \times (1.0 \times 1.0 \times 0.9 \times {}^{-}0.65 - 0.9 \times {}^{+}0.7)$$
$$\therefore w_2^{*} = {}^{-}1.80kN/m$$

Calculate Reaction:

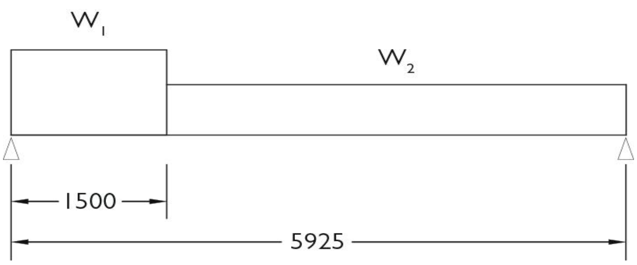
$$R^{*} = ({}^{-}1.80 \times 2.93^2 / 2 + {}^{-}2.24 \times 3.0 \times 4.43) / 5.93$$
$$\therefore R^{*} = {}^{-}6.32kN$$

Calculate w_{eff}

$$M^{*} = {}^{-}6.32 \times 2.82 - {}^{-}2.24 \times 2.82^2 / 2$$
$$\therefore M^{*} = {}^{-}8.85kNm$$
$$w_{eff} = {}^{-}8.85 \times 8 / 5.93^2$$
$$\therefore w_{eff} = {}^{-}2.01kN/m$$

For k_l = 2.0

Max. distance from edge = 0.5a = 1.5m.



$$w_{i=}^{*} = q_u.spacing.(k_a.k_l.k_{c,e}.c_{pe} - k_{c,i}.c_{pi})$$
$$w_1^{*} = 1.06 \times 1.4 \times (1.0 \times 2.0 \times 0.9 \times {}^{-}0.65 - 0.9 \times {}^{+}0.7)$$
$$\therefore w_1^{*} = {}^{-}2.67kN/m$$
$$w_2^{*} = {}^{-}1.80kN/m$$

Calculate Reaction:

$$R^{*} = ({}^{-}1.80 \times 4.43^2 / 2 + {}^{-}2.67 \times 1.5 \times 5.18) / 5.93$$
$$\therefore R^{*} = {}^{-}6.48kN$$

Calculate w_{eff}

$$M^* = {}^-6.48 \times 2.88 - {}^-1.80 \times 1.38^2 / 2 - {}^-2.67 \times 1.5 \times 2.13$$
$$\therefore M^* = {}^-8.42kNm$$
$$w_{eff} = {}^-8.42 \times 8 / 5.93^2$$
$$\therefore w_{eff} = {}^-1.92kN/m$$

Critical design wind loading, w_{wu} = $^-$2.01 kN/m, $^+$2.02 kN/m, R^*=$^-$6.48 kN

Serviceability

To obtain the serviceability wind load the ultimate uniform loads can be factored by the square of the ratio serviceability wind speed to ultimate wind speed.

$$w_s = \left(\frac{v_s}{v_u}\right)^2 \times w_s$$
$$w_s = \left(\frac{37}{45}\right)^2 \times {}^+2.02 = {}^+1.37kN/m$$
$$\delta_w = 1.0\left[\frac{5 \times {}^+1.37 \times 5925^4}{384 \times 476.9 \times 10^9}\right]$$
$$\therefore \delta_w = 46.1mm \quad or \quad Span/129$$

On the basis of applied local pressure factors and the instantaneous nature of the wind gust span/100 is deemed acceptable.

Strength

Consider positive wind pressures.

Check Bending Capacity

Girt design assumes the use of pierce fixed wall sheeting providing continuous lateral restraint to the outside face of the girt. Since compression edge is fully restrained k_{12}=1.0. *(Note: NZS 3603 denoted k_8 as the Stability factor).*

So for bending about XX axis
Calculate Bending Moment Capacity

$$M_d \geq M^* \qquad \text{Eq. 3.2(1), AS 1720.1}$$
$$M_d = \phi.k_1.k_4.k_6.k_9.k_{12}.f_b^{'}.Z \qquad \text{Eq. 3.2(2), AS 1720.1}$$

For solid section Futurebuild LVL members with depths greater than 95 mm, apply size factor of $\left(\frac{95}{D}\right)^{0.1538}$. For further information refer Futurebuild LVL Specific Design Guide.

where:

The selection of the capacity factor (termed strength reduction factor in NZ) for girts can be based on Application Category 1, Secondary structural members in structures other than houses, for LVL apply Ø=0.95.

$$\text{Ø} = 0.95 \quad k_4 = k_6 = k_9 = 1.0 \quad f_b^{'} = 28.4.\left(\frac{95}{D}\right)^{0.1538} MPa$$

$$Z = \frac{d^2.b}{6} = \frac{240^2 \times 45}{6}$$

$$\therefore Z = 432.0 \times 10^3 mm^3$$

$$M_d = 0.95 \times k_1 \times 1.0 \times 1.0 \times 1.0 \times k_{12} \times \left(\frac{95}{240}\right)^{0.1538} \times 28.4 \times 432.0 \times 10^3$$

$$\therefore M_d = 10.1.k_1.k_{12} kNm$$

Duration of load factor for girts is selected based on a single wind gust, not exceeding 5 seconds, k_1 =1.0

Compression edge of girt has continuous restraint from pierce fixed sheeting, therefore k_{12} =1.0

$$M_d = 10.1kNm > M^*$$

Calculate Shear Capacity

$$V_d \geq V^*$$

Eq. 3.2(13), AS 1720.1
(Eq. 3.3, NZS 3603 Similar)

$$V_d = \phi.k_1.k_4.k_6.f_s^{'}A_S$$

Eq. 3.2(14), AS 1720.1
(Eq. 3.4 NZS 3603 Similar)

where:

$$\text{Ø} = 0.95 \qquad k_1 = 1.0$$

$$k_4 = k_6 = 1.0 \quad f_s = 4.6MPa$$

Futurebuild Specific Design Guide

$$A_S = 2.b.d/3$$

$$\therefore A_S = \frac{2}{3}.240 \times 45 = 7200mm^2$$

$$\therefore V_d = 0.95 \times 1.0 \times 1.0 \times 4.6 \times 7200$$

$$\therefore V_d = 31.5kN > V^*$$

Consider negative wind pressures.

$$w_{w_u} = {}^{-}2.01kN/m$$

$$M^* = \frac{w.l^2}{8} = \frac{^-2.01 \times 5.9^2}{8}$$
$$\therefore M^* =^- 8.9kNm$$

Calculate bending moment capacity

From previous:

$$M_d = 10.1.k_{12}kNm$$

Calculate k_{12}

Continuous lateral restraint is provided to the tension edge via pierce fixed sheeting.

Calculate S_1

$$S_1 = 2.25.\frac{d}{b}$$ Eq. 3.2(13), AS 1720.1

$$S_1 = 2.25 \times \frac{240}{45} = 12.0$$

Calculate *p*

The material constant, p, takes into account the material effects in the determination of the slenderness co-efficient whilst the slenderness coefficient, S, takes into account the geometrical parameters. The ratio 'r' is a function of the temporary design action effect divided by the total design action effect

$$\rho_b = 14.71\left(\frac{E}{f_b^{'}}\right)^{-0.408} r^{-0.061}$$ Eq. 8(1), AS 1720.1

where:

$$E = 9200MPa$$
$$f_b^{'} = 28.4\left(\frac{95}{D}\right)^{0.1538}$$
$$r = 1$$
$$\rho_b = 14.71\left(\frac{9200}{24.6}\right)^{-0.480} 1.0^{-0.061}$$
$$\therefore \rho_b = 0.85$$
$$\Rightarrow \rho_b S_1 = 10.3$$

Since $10 \le \rho_b S \le 20$

$k_{12} = 1.5 - 0.05 \times \rho_b S_1$ Eq. 3.2(11), AS 1720.1

$k_{12} = 1.5 - 0.05 \times 10.3 = 0.98$

$\therefore M_d = 10.1 \times 0.98 = 9.95kNm > M^*$

4.2 Connection design

Propose JH47x190 to suit 240x45 hyFRAME. It is typical to apply a practical minimum number of nails for bracket and beam stability, for members around 240 mm deep we recommend a minimum of 6/12gx35 type17 screws ie. 3/12g screws per tab.

Calculate minimum number of 12g type 17 Hex Head screws

Type 17 screws are preferred for timber connection as they are a self drilling screw through the timber.

$N_{d,j} \geq N^*$ Eq. 4.3(1), AS 1720.1
(Eq. 4.5, NZS 3603 Similar)

$N_{d,j} = \phi.k_1.k_{13}.k_{14}.k_{16}.k_{17}.n.Q_k$ Eq. 4.3(2), AS 1720.1
(Eq. 4.6, NZS 3603 Similar)

where:

$Ø = 0.85 \quad Q_k = 2.153kN$ AS 1720.1

$k_1 = 1.14$ *(critical load case is wind uplift, effective duration of peak action 5 seconds)*
$k_{13} = 1.0$ *(screws into side grain)*
$k_{14} = 1.0$ *(screws in single shear)*
$k_{16} = 1.0$ *(screws through LVL)*
$k_{17} = 1.0$ *(number of rows of fasteners ≤ 4)*

From previous $V_d = 6.48kN$

$N_{d,j} = 0.85 \times 1.14 \times 1.0 \times 1.0 \times 1.0 \times 1.0 \times 6 \times 2.153$
$\therefore N_{d,j} = 12.52kN > V_d$

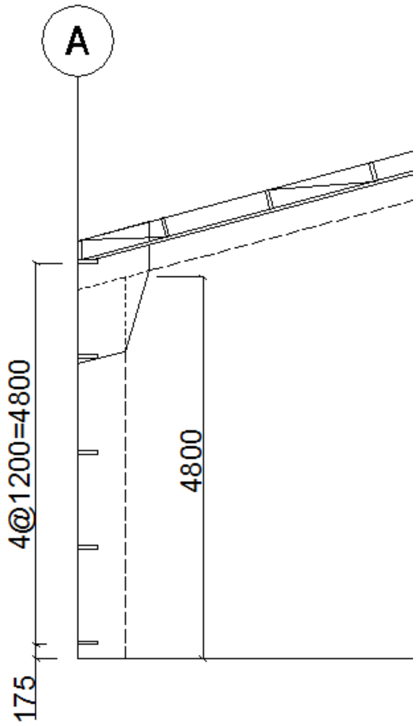
Say 6/12gx35 type 17 Hex Head screws, screwed through JH47190 joist hanger.

Also confirm Characteristic Strength Capacity of Bracket.

$ØQ = 18.0kN > N^*$ Mitek Literature

The 240x45 hyFRAME girt to 5.9 m at maximum 1400 mm spacing is adequate to support the design load as a girt.

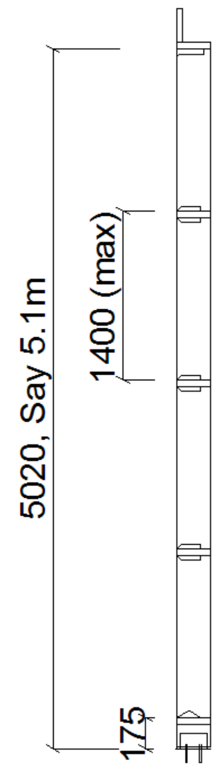
4.3 Proposed Girt Layout (Current hyFRAME shown) Girts spaced at maximum 1400 c/c.



5.0 Mullion design, side wall

The mullion is best calculated as a vertical member supporting a series of point loads that share a common spacing, which is typical of mullions. Standard beam formulae have been adapted to best provide accurate but easy to calculate equations. Refer Appendix 3 for beam equations, where n is the number of girts.

Consider side wall mullion



Side wall Mullion Span 5.1 m

Girt Spacing 1200 mm
(Note for capacity based on maximum 1400 mm spacing)

Propose 240x63 hySPAN for use as side wall mullion.

5.1 Wind loading

Side wall girts are to be of the same section size as side wall mullions and as such provide lateral restraint to the compression edge for both and negative wind pressures. Girt reactions have been recalculated excluding the local pressure factors as the mullion does not directly support the cladding.

Determine local pressure factor independent reaction at girt ends (mullions are not subjected to local pressure factors:

Side wall mullion load width (max.) = 3.0 m

θ = 0°, Lateral wind

Girt loading (positive pressure)

$$P_{+ve} = 1.06 \times 1.4 \times \left(0.9 \times^{-} 0.65 - 0.9 \times^{+} 0.7\right) \times 3.0 =^{+} 5.41kN$$

θ = 90°, Longitudinal wind

Girt loading (negative pressure)

$$P_{-ve} = 1.06 \times 1.4 \times \left(0.9 \times^{+} 0.7 - 0.9 \times^{-} 0.65\right) \times 3.0 =^{-} 5.41kN$$

Serviceability

Apply maximum wind pressure for serviceability

$$P_s = \left(\frac{37}{45}\right)^2 \times^{+} 4.63 =^{+} 3.66kN$$

$$n = 4$$

Appendix 3

$$\delta_w = k_2 . \frac{P.L^3}{192.EI} . n . \left[3 - \frac{1}{2}\left(1 + \frac{4}{n^2}\right)\right]$$

$$\delta_w = 1.0 \times \frac{3660 \times 5100^3}{192 \times 13200 \times 72.5 \times 10^6} \times 4 \times \left[3 - \frac{1}{2}\left(1 + \frac{4}{4^2}\right)\right]$$

$$\therefore \delta_w = 25.0mm \quad or \quad Span/200$$

Strength

Calculate bending moment and shear

Positive wind pressures

$$M^* = n.\frac{P.l}{8} = 4 \times \frac{{}^{+}5.41 \times 5.1}{8}$$

$$\therefore M^* = 17.2kNm$$

Negative wind pressures

$$M^* = n.\frac{P.l}{8} = 4 \times \frac{{}^{-}5.41 \times 5.1}{8}$$

$$\therefore M^* =^{-} 17.2kNm$$

Consider shear and support reaction for wind load

$$V^* = (n-1).\frac{P}{2} = 4 \times \frac{{}^{+}5.41}{2}$$

$$\therefore N^* = V^* =^{+} 10.8kN$$

Calculate Bending Moment Capacity

Positive Pressure.

Calculate Slenderness Co-efficient, k_{12}

Compression edge restrained by girts at 1400 mm spacing (conservative – to suit 3.6 m high frame).

$$S_1 = 1.25\frac{d}{b}\left[\frac{L_{ay.}}{d}\right]^{0.5}$$ Eq. 3.4(4), AS 1720.1

$$S_1 = 1.25 \times \frac{240}{63}\left[\frac{1400}{240}\right]^{0.5}$$

$$\therefore \mathrm{S}_1 = 11.50$$

Calculate *p* factor

$$\rho_b = 14.71\left(\frac{E}{f_b^{'}}\right)^{-0.408} r^{-0.061}$$ Eq. 8(1), AS 1720.1

where:

$$E = 13200MPa$$

$$f_b^{'} = 50\left(\frac{95}{D}\right)^{0.1538}$$

$$r = 1$$

$$\rho_b = 14.71\left(\frac{13200}{43.4}\right)^{-0.480} 1.0^{-0.061}$$

$$\therefore \rho_b = 0.95$$

$$\Rightarrow \rho_b S_1 = 10.9$$

Since $10 \le \rho_b S \le 20$

$$k_{12} = 1.5 - 0.05\rho_b S$$

$$\therefore k_{12} = 1.5 - 0.05 \times 10.9 = 0.95$$ Eq. 3.2(10), AS 1720.1

$$Ø = 0.95 \quad k_4 = k_6 = k_9 = 1.0 \quad f_b^{'} = 50.\left(\frac{95}{D}\right)^{0.1538} MPa$$

$$Z = \frac{d^2.b}{6} = \frac{240^2 \times 63}{6}$$

$$\therefore Z = 604.8 \times 10^3 mm^3$$

$$M_d = 0.9 \times k_1 \times 1.0 \times 1.0 \times 1.0 \times k_{12} \times \left(\frac{95}{240}\right)^{0.1538} \times 50 \times 604.8 \times 10^3$$

$$\therefore M_d = 23.6.k_1.k_{12} kNm$$

Duration of load factor for mullion is selected based on wind load not exceeding 5 seconds, k_1 =1.0

$$M_d = 22.4kNm > M^*$$

Calculate Shear Capacity

$V_d \geq V*$ Eq. 3.2(13), AS 1720.1
(Eq. 3.3, NZS 3603 Similar)

$V_d = \phi.k_1.k_4.k_6.f_s^{'}A_S$ Eq. 3.2(14), AS 1720.1
(Eq. 3.4 NZS 3603 Similar)

where:

$\O = 0.9 \qquad k_1 = 1.0$
$k_4 = k_6 = 1.0 \quad f_s = 4.6MPa$ Futurebuild Specific Design Guide
$A_S = 2.b.d/3$
$\therefore A_S = \frac{2}{3}.240 \times 63 = 10080mm^2$

$\therefore V_d = 0.9 \times 1.0 \times 1.0 \times 4.6 \times 10080$
$\therefore V_d = 41.7kN > V^*$

5.2 Connection design

Connection to the ground is proposed using Mitek CF2x brackets whilst the connection to the eaves beam can be performed using two Mitek N21 Diagonal Cleats. The design capacities expressed in the Mitek literature are based on fully nailing out the holes. We can calculate a reduced number of fasteners for ease of installation whilst maintaining the structural integrity. It is recommended that the reduced number of fasteners are evenly distributed across the tab/bracket area.

Propose use of proprietary Ø3.15x30 FH nails for connection to mullion.

$N_{d,j} \geq N*$ Eq. 4.2(1), AS 1720.1
$N_{d,j} = \phi.k_1.k_{13}.k_{14}.k_{16}.k_{17}.n.Q_k$ Eq. 4.2(2), AS 1720.1

where:

$\O = 0.8 \quad Q_k = 0.810kN$ AS 1720.1

$k_1 = 1.14$ *(critical load case is wind uplift, effective duration of peak action 5 seconds)*
$k_{13} = 1.0$ *(Nails into side grain)*
$k_{14} = 1.0$ *(nails in single shear)*
$k_{16} = 1.0$ *(nails through LVL)*
$k_{17} = 1.0$ *(number of fasteners ≤ 4)*

From previous $N^* = 10.8kN$

$$N_{d,j} = 0.8 \times 1.14 \times 1.0 \times 1.0 \times 1.0 \times 1.0 \times 20 \times 0.810$$
$$\therefore N_{d,j} = 14.77kN$$

Say 10/Ø3.15x30 FH nails per tab, per bracket. Stagger nails.

From previous try 6/12gx35 type 17

$$N_{d,j} = 0.8 \times 1.14 \times 1.0 \times 1.0 \times 1.0 \times 1.0 \times 6 \times 2.153$$
$$\therefore N_{d,j} = 11.78kN > V_d$$

Say 6/12gx35 type 17 Hex Head screws, screwed through FB65170 joist hanger.

Also check Characteristic Strength Capacity of Brackets:

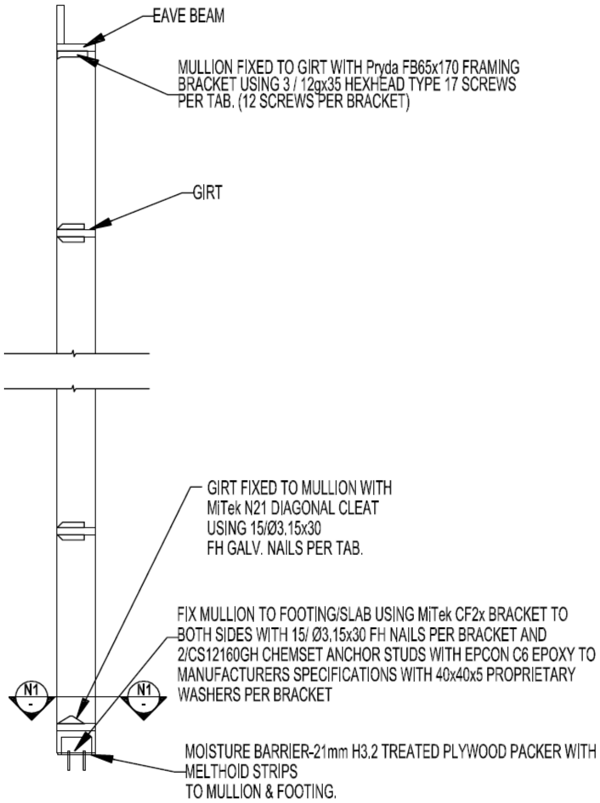
FB65170 joist hanger short term load case capacity
$\varnothing Q = 0.8 \times 15.5kN > N^*$ Pryda Literature

One pair of CF2x brackets with 2/ M12 Chemset CS12160GH anchors – Epcon C6 epoxy
Load in shear only.

$\varnothing Q = 0.8 \times 48.0kN > N^*$ Mitek Literature

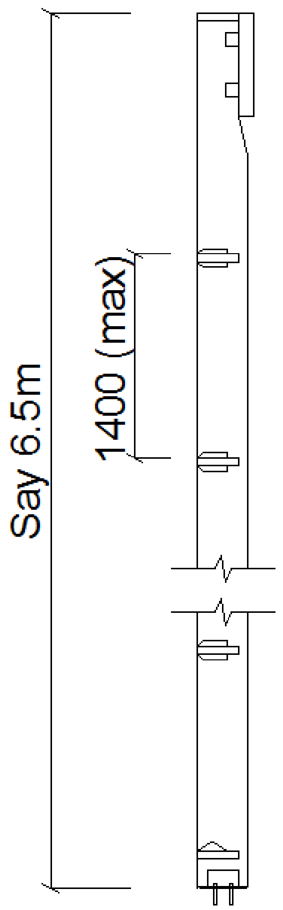
$\varnothing Q = 4 \times 7 = 28kN > N^*$ Reid literature – ref appendix 5

Proposed Mullion Connection



6.0 Mullion design, end wall

Consider end wall mullion



Mullion Span 6.5 m

Girt Spacing 1200 mm
(note: max. 1400 used for stability calculation)

Propose 300x90 hy90 for use as end wall mullion.

6.1 Wind loading

Girts provide lateral restraint to the compression edge for positive pressures and to the tension edge for negative wind pressures. Girt loads have been recalculated excluding local pressure factors.

θ = 0°, Lateral wind

Girt loading (negative pressure)

$$P_{-ve} = 1.06 \times 1.2 \times \left(0.9 \times^{-} 0.65 - 0.9 \times^{+} 0.7\right) \times 6.0 =^{-} 9.28kN$$

θ = 90°, Longitudinal wind

Girt loading (positive pressure)

$$P_{+ve} = 1.06 \times 1.2 \times \left(0.9 \times^{-} 0.65 - 0.9 \times^{+} 0.7\right) \times 6.0 =^{+} 9.28kN$$

Serviceability

Apply maximum wind pressure for serviceability

$$P_s = \left(\frac{37}{45}\right)^2 \times {}^{+}9.28 = {}^{-}6.27kN$$

$$n = 5$$

$$\delta_w = j_2 . \frac{P.L^3}{192.EI} . \left[n - \frac{1}{n}\right] . \left[3 - \frac{1}{2}\left(1 - \frac{1}{n^2}\right)\right]$$

Appendix 3

$$\delta_w = 1.0 \times \frac{6270 \times 6500^3}{192 \times 9500 \times 202.5 \times 10^6} \times \left[5 - \frac{1}{5}\right] \times \left[3 - \frac{1}{2}\left(1 - \frac{1}{5^2}\right)\right]$$

$$\therefore \delta_w = 56.4mm \quad or \quad Span/115$$

Strength

Calculate bending moment and shear

Positive wind pressures

$$M^* = \left(n^2 - 1\right) . \frac{P.l}{8.n} = \left(5^2 - 1\right) \times \frac{{}^{+}9.28 \times 6.5}{8 \times 5}$$

$$\therefore M^* = {}^{+}36.2kNm$$

Negative wind pressures

$$M^* = \left(n^2 - 1\right) . \frac{P.l}{8.n} = \left(5^2 - 1\right) \times \frac{{}^{-}9.28 \times 6.5}{8 \times 5}$$

$$\therefore M^* = {}^{-}36.2kNm$$

Consider shear and support reaction for wind load

$$V^* = (n - 1) . \frac{P}{2} = (6 - 1) \times \frac{{}^{-}9.28}{2}$$

$$\therefore N^* = V^* = 23.2kN$$

Calculate Bending Moment Capacity

Since positive and negative pressures are equal, determine capacity for negative pressures where girts provide discrete restraint to the tension edge.

Calculate k_{12}

Tension edge restrained by girts at 1400 mm spacing (maximum).

Consider Stability equation for Discrete Restraint to Tension Edge from AS 1720.1.

$$S_1 = \left(\frac{d}{b}\right)^{1.35}\left(\frac{L_{ay}}{d}\right)^{0.5}$$ Eq. 3.2(5) AS 1720.1

$$S_1 = \left(\frac{300}{90}\right)^{1.35}\left(\frac{1400}{300}\right)^{0.5}$$

$$\therefore \mathrm{S}_1 = 10.97$$

Calculate *p* factor

$$\rho_b = 14.71\left(\frac{E}{f_b^{'}}\right)^{-0.408} r^{-0.061}$$ Eq. 8(1), AS 1720.1

where:

$$E = 9500MPa$$

$$f_b^{'} = 35\left(\frac{95}{D}\right)^{0.1538}$$

$$r = 1$$

$$\rho_b = 14.71\left(\frac{9500}{29.32}\right)^{-0.480} 1.0^{-0.061}$$

$$\therefore \rho_b = 0.92$$

$$\Rightarrow \rho_b S_1 = 10.1$$

Since $\rho_b S > 10$

$$k_{12} = 1.5 - 0.05\rho_b S$$

$$\therefore k_{12} = 1.0$$ Eq. 3.2(10), AS 1720.1

$$Ø = 0.95 \quad k_4 = k_6 = k_9 = 1.0 \quad f_b^{'} = 35.\left(\frac{95}{D}\right)^{0.1538} MPa$$

$$Z = \frac{d^2.b}{6} = \frac{300^2 \times 90}{6}$$

$$\therefore Z = 1350.0 \times 10^3 mm^3$$

$$M_d = 0.95 \times k_1 \times 1.0 \times 1.0 \times 1.0 \times k_{12} \times \left(\frac{95}{300}\right)^{0.1538} \times 35 \times 1350.0 \times 10^3$$

$$\therefore M_d = 37.6.k_1.k_{12} kNm$$

Duration of load factor for mullion is selected based on wind load not exceeding 5 seconds, k_1 =1.0

$$M_d = 37.6kNm > M^*$$

Calculate Shear Capacity

$$V_d \geq V^*$$

Eq. 3.2(13), AS 1720.1
(Eq. 3.3, NZS 3603 Similar)

$$V_d = \phi.k_1.k_4.k_6.f_s^{'}A_S$$

Eq. 3.2(14), AS 1720.1
(Eq. 3.4 NZS 3603 Similar)

where:

$$\text{Ø} = 0.95 \qquad k_1 = 1.0$$
$$k_4 = k_6 = 1.0 \qquad f_s = 4.6MPa$$

$$A_S = 2.b.d/3$$
$$\therefore A_S = \frac{2}{3}.300 \times 90 = 18000mm^2$$

$$\therefore V_d = 0.95 \times 1.0 \times 1.0 \times 4.6 \times 18000$$
$$\therefore V_d = 78.6kN > V^*$$

Consider notching of mullion top.

The mullion notch to accommodate the rafter needs to be checked. The notch will only fracture due to an opening moment which would be caused by a positive wind pressure. Since the rafter acts as a support for the mullion the moment at the notch is zero, however the shear force needs to be considered as per equation 3.7, NZS 3603.

M* = 0 kNm, V* = N* = 23.2 kN

$$\frac{6M^*}{bdn^2} + \frac{6V^*}{bdn} \leq \phi \times g_{40} \times k_1 \times k_4 \times k_6 \times k_{12} \times f_{sj}$$

AS1720.1 EQN 9(1)

$$g_{40} = {}^{9}\!/_{300^{0.24}} = 2.29$$

$$\frac{0 \times 6}{90 \times 240^2} + \frac{6 \times 23.2 \times 10^3}{90 \times 240} \leq 0.9 \times 2.29 \times 1.0 \times 1.0 \times 1.0 \times 1.0 \times 4.6$$

$$6.4 \leq 9.5 \therefore O.K$$

$\therefore$ Notch as above O.K.

6.2 Connection design

Connection to the ground is proposed using Mitek CF2x brackets, similar to the side wall mullion. The connection to the rafter requires tension capable fixings. One system using proprietary brackets involves the use of three Mitek concealed purlin cleats (CP80) together with one suitable length strap or cyclone tie wrapped around the mullion.

$R^{*}top =^{+/-} 23.2kN$ See appendix B

Propose 3/mitek CPC80 cleats, fixed with 8/Ø3.15x30 FH nails and 4/14gx35 type 17 screws per bracket

$Capacity = 0.8 \times 24$

$\therefore Capacity = 19.2kN$ Mitek Literature

Pryda Maxi Strap, wrapped around mullion with 6 nails per end.

$ØQ = 6 \times 2 = 12kN^{*}$ Pryda Literature

$\therefore ØQ\_system = 19.2 + 12 = 31.2kN \geq R^{*}top \therefore O.K$

$R^{*}base =^{+/-} 23.2kN$ See appendix B

Propose mitek CF2x cleats fixed to manufacturers specs with 2/ M12 Chemset CS12160GH anchors – Epcon C6 epoxy

$Capacity = 0.8 \times 48$

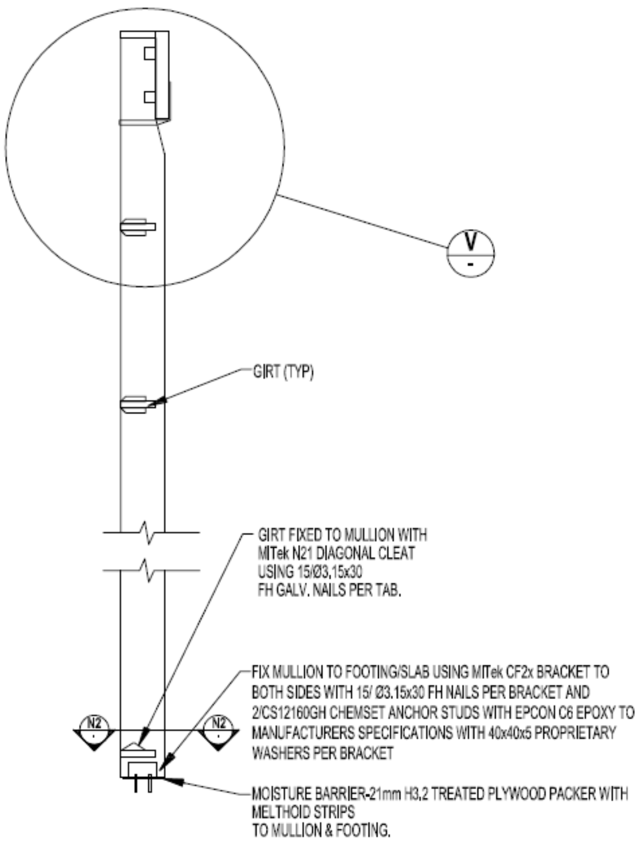
$\therefore Capacity = 38.4kN \geq R^{*}base \therefore O.K$

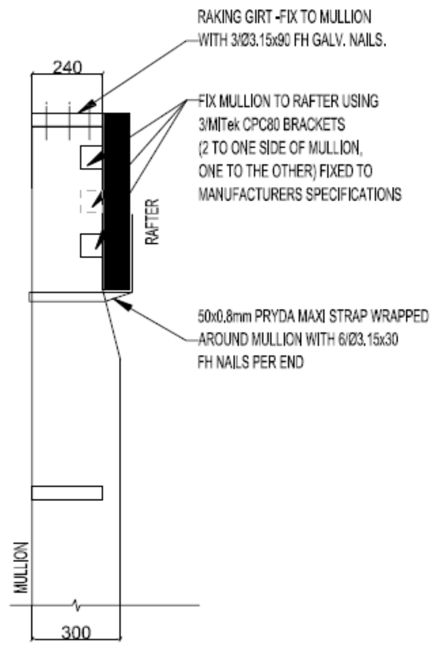
$Capacitybolts = 4 \times 7kN$

$\therefore Capacitybolts = 28kN \geq R^{*}base \therefore O.K$

Mitek/Reid Literature (ref appendix 5)

Proposed Connection





7.0 Longitudinal Bracing

Portal Frame action takes care of the lateral loads however Longitudinal Bracing is required to transfer the longitudinal forces to the ground. For spans up to 20m strap bracing is an easy way to achieve the required longitudinal bracing requirements. Essentially the braced bays will act as a truss transferring the loads through the purlins between braced bays.

θ = 90˚, Longitudinal wind

Consider critical loading

q_u=1.06 kPa $c_{p,e\ (Windward)}$ = $^{+}0.7$, $c_{p,e\ (Leeward)}$= $^{-}0.3$ (D/B=2.5)

Note: Internal pressures cancel each other.

$k_{c,e}$ = 0.9 (2 surfaces)

Calculate design loading

Calculate end wall force
(Note: Assume heights for 4.8 clear height building)

$$Area = \frac{1}{2}.\left(\frac{h_{eave} + h_{ridge}}{2}\right).\frac{b}{2}$$

$$Area = \frac{1}{2}.\left(\frac{5.26 + 6.87}{2}\right).\frac{12}{2}$$

$$Area = 18.2m^2 / side$$

$$P^*_{end\_wall} = q_u.A.k_c.\left(c_{p,e(W)} - c_{p,e(L)}\right)$$

$$P^*_{end\_wall} = 1.06 \times 18.2 \times 0.9 \times (^{+}0.7 - ^{-}0.3)$$

$$\therefore P^*_{end\_wall} = 17.4kN$$

Consider friction due to sheeting profile

$$\frac{d}{h} = \frac{30.0}{6.07} = 4.9$$

Cl 5.5 AS/NZS 1170.2:2002

$$\frac{d}{b} = \frac{30.0}{12.0} = 2.5$$

Since one of the building ratios exceed 4 the friction force acting on the building needs to be resisted.

Calculate Friction Force

Calculate Load Area for friction

$$Area = (b + 2h)(d - 4h)$$

AS/NZS 1170.2 Eqn. 5.5(a)

$$Area = (12 + 2 \times 6.07)(30 - 4 \times 6.07)$$

$$\therefore Area = 138.1m^2$$

For large buildings the area may be broken up into its roof and wall contribution to take advantage of the lower loads to be resisted however for this example the loads do not reach levels where any significant advantage may be gained.

$c_f = 0.04$ - Ribs across the wind direction AS/NZS 1170.2 Table 5.9

$$P^*_{friction} = q_u.A_{friction}.K_c c_f$$

$$P^*_{friction} = 1.06 \times 138.1 \times 0.9 \times 0.04$$

$$\therefore P^*_{friction} = 5.3kN$$

Total load to be resisted by bracing per side

$$P^*_{Total,w_u} = P^*_{end\_wall} + \frac{P^*_{friction}}{2}$$

$$P^*_{Total,w_u} = 17.4 + 5.3$$

$$\therefore P^*_{Total,w_u} = 22.7kN$$

Consider the Earthquake Design Action Effects from Section 1.1.5 and Appendix 4

- $Base\_Horizontal\_Shear = 72.0kN$

Total load to be resisted by bracing per side

$$P^*_{Total,Eq} = \frac{P^*_{Base\_horizontal\_shear}}{2}$$

$$P^*_{Total,E_q} = \frac{72.0}{2}$$

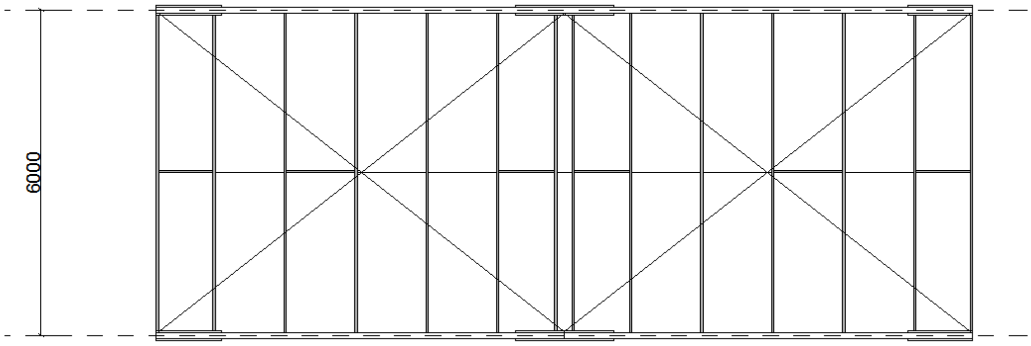
$$\therefore P^*_{Total,E_q} = 36.0kN$$

i.e. Seismic is critical for longitudinal bracing design

For long buildings it is good practice to have a bracing bay each end of the building rather than relying on the building to transfer all bracing loads from one end to the other, for this example we propose two braced bays.

Therefore the horizontal load to be transferred in each braced bay is 18.0 kN.

Consider Roof Bracing Layout - tension only (wall layout similar)



Calculate the number of cross braces required in each braced bay:

Roof Bracing

Roof bracing angle = 44°

$$R_{brace} = \frac{18}{Cos44} = 25.0kN$$

$$\#cross\_braces = \frac{R_{brace}}{Maxistrap\_Capacity}$$

$$\#cross\_braces = \frac{25.0}{13.2}$$

$$\therefore \#cross\_bracess = 1.9$$

Say 2 bays of roof bracing using double Pryda Maxistrap

Wall brace

Wall bracing angle = 38°

$$R_{brace} = \frac{18}{Cos38} = 22.8kN$$

$$\#cross\_braces = \frac{R_{brace}}{Maxistrap\_Capacity}$$

$$\#cross\_braces = \frac{22.8}{13.2}$$

$$\therefore \#cross\_braces = 1.7$$

Say 2 bays of wall bracing using double Pryda Maxistrap

7.1 Purlins subject to axial loads

As above, purlins are used to transfer longitudinal loads down the length of the building in tension or compression. It is good practice to apply continuity strap at connections to transfer tension between purlins across the portal frame. Loads are generally low however can be significant close to the eave and/or ridge in braced bays where tension-only brace loads are transferred through the purlin in compression.

In this case assume 50% of P* is transferred through a single purlin (conservative).

For wind combined check: $N_T^* = N_C^* = 0.5.P^* = 0.5 \times 11.4 \approx 6kN$

For seismic axial check: $N_T^* = N_C^* = 0.5.P^* = 0.5 \times 18 \approx 9kN$

Critical design actions for combined actions check (from section 2 earlier):

$M^* = -9.66kNm \qquad k_{12} = 0.98 \qquad M_d = 9.95kNm$

Combined bending and compression checks:

$$\left(\frac{M_x^*}{M_{d,x}}\right)^2 + \frac{N_c^*}{N_{d,cy}} \leq 1.0$$ Eq. 3.5(1) AS 1720.1

$$\frac{M_x^*}{M_{d,x}} + \frac{N_c^*}{N_{d,cx}} \leq 1.0$$ Eq. 3.5(2) AS 1720.1

Calculate capacity in compression:

$$N_{d,c} = \phi.k_1.k_4.k_6.k_{12}.f'_c.A_c$$ Eq. 3.3(2) AS 1720.1

Calculate k_{12}

Consider stability equations for lateral buckling under compression from AS 1720.1.

Major axis buckling, S3 is lesser of:

$$S_3 = \frac{L_{ax}}{d}$$ Eq. 3.3(5) AS 1720.1

or

$$S_3 = \frac{g_{13}.L}{d}$$ Eq. 3.3(6) AS 1720.1

where:

$$L = L_{ax} = 5925mm$$
$$g_{13} = 1.0$$ Table 3.2 AS 1720.1
$$d = 240$$

$$\therefore S_3 = \frac{5925}{240} = 24.7$$

Minor axis buckling, assuming top edge continuously restrained by roofing, S4 is:

$$S_4 = 3.5.\frac{d}{b}$$ Eq. 3.3(10) AS 1720.1

$$\therefore S_4 = 3.5 \times \frac{240}{45} = 18.7$$

Calculate *p* factor

$$\rho_b = 11.39\left(\frac{E}{f_c^{'}}\right)^{-0.408} r^{-0.074}$$ Eq. 8(2), AS 1720.1

where:

$$E = 9200MPa$$
$$f_c^{'} = 30MPa$$
$$r = 1.0$$
$$\rho_c = 11.39\left(\frac{9200}{30}\right)^{-0.408} 1.0^{-0.074}$$
$$\therefore \rho_c = 1.10$$
$$\Rightarrow \rho_c S_3 = 1.10 \times 24.7 = 27.2$$
$$\Rightarrow \rho_c S_4 = 1.10 \times 18.7 = 20.6$$

Since $\rho_c S > 20$

$$k_{12} = \frac{200}{(\rho_c S)^2}$$
$$\therefore major\_axis\_k_{12} = 0.27$$ Eq. 3.3(11c), AS 1720.1
$$\therefore \min or\_axis\_k_{12} = 0.47$$

$$\phi = 0.9$$
$$k_1 = 1.0$$
$$k_4 . k_6 = 1.0$$
$$A_c = 240 \times 45 = 10800mm^2$$

$$N_{d,cx} = 0.9 \times 1.0 \times 1.0 \times 1.0 \times 0.27 \times 30 \times 10800 = 78.7kN > N^*$$
$$N_{d,cy} = 0.9 \times 1.0 \times 1.0 \times 1.0 \times 0.47 \times 30 \times 10800 = 137.0kN > N^*$$

Combined wind actions checks:

$$\left(\frac{M_x^*}{M_{d,x}}\right)^2 + \frac{N_c^*}{N_{d,cy}} = \left(\frac{9.66}{9.95}\right)^2 + \frac{6.0}{137} = 1.0$$
$$\frac{M_x^*}{M_{d,x}} + \frac{N_c^*}{N_{d,cx}} = \frac{9.66}{9.95} + \frac{6.0}{78.7} = 1.05$$

One of the combined checks is 5% over however M* has been calculated including local pressure coefficients (for purlins in the end bays). The probability of the maximum local pressure coefficient applying to a purlin at the same time as maximum axial load transfer is very low due to the instantaneous nature of wind gusts, location of the braced bays and the building length. On this basis 5% has been deemed acceptable.

Combined bending and tension checks:

$$\frac{k_{12}.M^*}{M_d}+\frac{N_T^*}{N_{d,t}}\leq 1.0$$ Eq. 3.5(3) AS 1720.1

$$\frac{M_x^*}{M_{d,x}}-\frac{Z}{A}.\frac{N_T^*}{M_{d,x}}\leq 1.0$$ Eq. 3.5(4) AS 1720.1

Calculate capacity in tension:

$$N_{d,t}=\phi.k_1.k_4.k_6.f'_t.A_t$$ Eq. 3.4(2) AS 1720.1

$$\phi=0.9$$
$$k_1=1.0$$
$$k_4.k_6=1.0$$
$$f_t'.=15.7.\left(\frac{150}{d}\right)^{0.167}=14.5MPa$$
$$A_c=240\times 45=10800mm^2$$

$$N_{d,t}=0.9\times 1.0\times 1.0\times 1.0\times 14.5\times 10800=140.9kN>N^*$$

Combined wind actions checks:

$$\frac{k_{12}.M^*}{M_d}+\frac{N_T^*}{N_{d,t}}=\frac{0.98\times 9.66}{9.95}+\frac{6.0}{140.9}=0.99$$

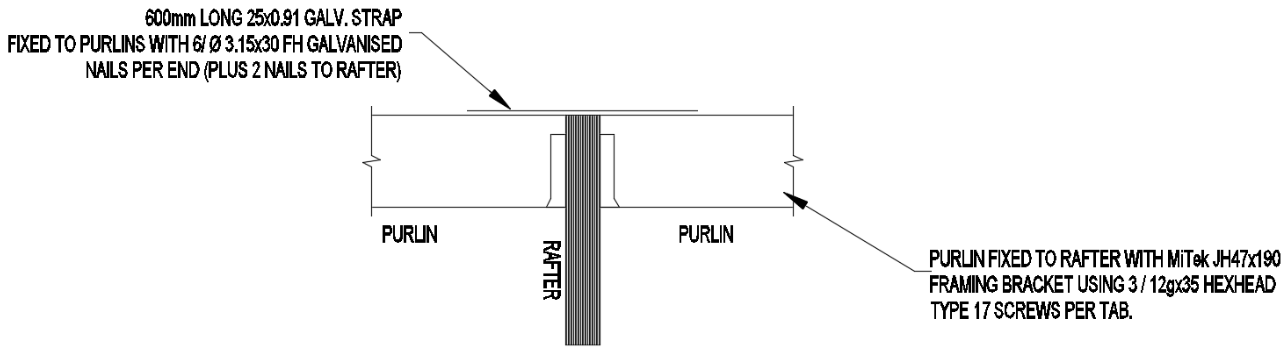
$$\frac{M_x^*}{M_{d,x}}-\frac{Z}{A}.\frac{N_T^*}{M_{d,x}}=\frac{9.66}{9.95}-\frac{432}{10800}.\frac{6.0}{9.95}=0.99$$

Both ok.

Connection requirements:

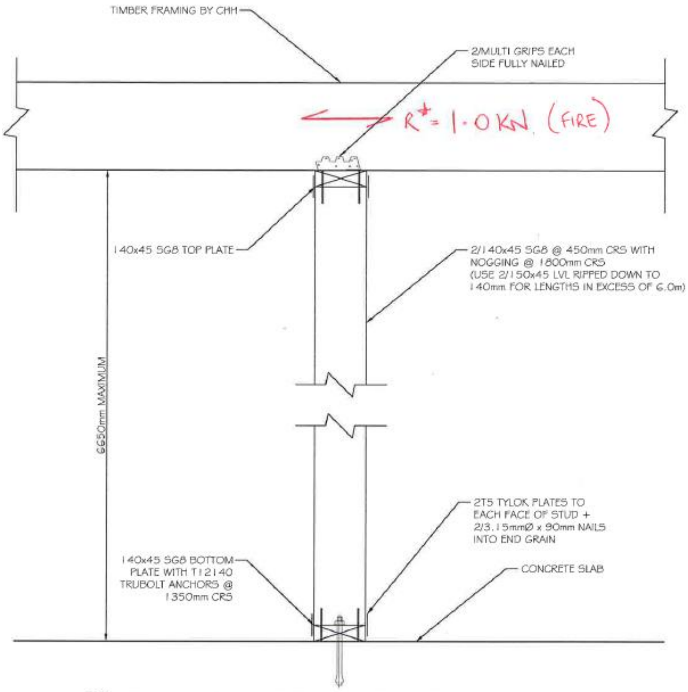
Generally apply 25x0.91mm sheet brace straps with 6/3.15 x 30 FH nails as continuity strap (6kN capacity – refer Pryda literature in Appendix 6).
Eave purlins to use either two of the above straps or 50mm Pryda Maxistrap with 10/3.15x30 FH nails.

Proposed continuity strap detail below:

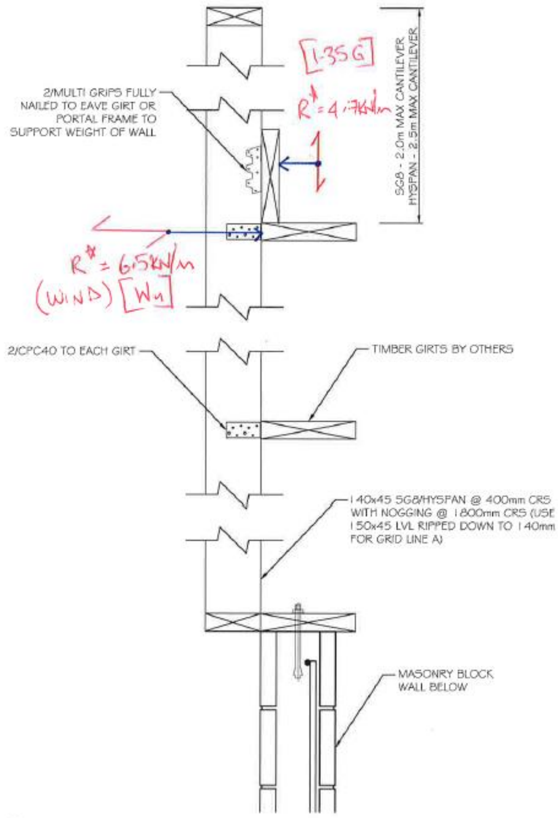


8.0 Parapet wall framing support

This project has a parapet wall in one corner which runs along grid A for 6 m and grid M for 3 m. The wall framing has been designed and specified by others however support is required by the LVL structure. Connection details and loads to allow for have been provided by the consultant who has designed the parapet wall framing, Hutchinson Consulting Engineers, refer below.
Loads imposed by the internal intertenancy wall between grids D and E on the LVL purlins have also been supplied. These loads are much lower than the axial forces considered previously for the purlins so no extra checks are required.



TYPICAL INTER TENNANCY WALL FRAMING DETAILS
SCALE 1:10



TYPICAL PARAPET WALL FRAMING DETAILS
SCALE 1:10

8.1 Horizontal load applied at eave height:

For lateral case, horizontal load to be transferred to portal frames by eave beam. Check 240x45 hyFRAME eave member is sufficient:

Eave member carries Wu* = 6.5kN/m for 3 m (refer above) and half normal girt load for other 3 m length; Wu* = 0.5 x -2.02 = 1.01kN/m. Analysis run using computeIT for beams software, refer Appendix 2.
Reactions at portal frames on grids A and B added to microstran analysis and design completed for increased design actions, refer appendix 1 and 2.

Adopt 240x90 hy90 member for eave girt between grids A & B (apply on both sides of building to avoid confusion on site) with Mitek SPH220 brackets.

For longitudinal case, horizontal load to be transferred to bracing by purlins. Horizontal load at portal frame rafter level is Wu* = 6.5kN/m therefore axial load per purlin is 6.5 kN/m x 1.35 m = 8.8kN.

Lower than axial loads checked earlier for seismic case (9 kN) so ok but longer straps required to facilitate load transfer. Use 50 mm Pryda Maxistrap with 10/3.15 x 30 FH nails per end in parapet wall area.

8.2 Vertical load:

Vertical load due to mass of wall framing is proposed to be transferred down the girt blocking to the masonry wall below. Provide blocking at 400 mm centres in line with parapet wall frame studs.

8.1 Effect on longitudinal bracing:

Check additional bracing load for wind case.

Calculate extra wall area:

$$A_{extra} \approx 0.5\times6.0\times1.6+0.5\times6\times1.8$$

$$A_{extra} \approx 11m^2$$

$$P^*_{extra} = q_u.A.k_c.\left(c_{p,e(W)} - c_{p,e(L)}\right)$$

$$\therefore P^*_{extra} \approx 1.06\times11\times0.9\times({}^{+}0.7 - {}^{-}0.3) = 10.5kN$$

$$\Rightarrow P^*_{total} = 17.4+10.5 = 27.9kN$$

P* is lower than seismic case so ok, refer above for continuity strap requirement for purlins.

9.0 Durability

Futurebuild LVL is manufactured to meet the requirements of the New Zealand Building Code Clause B2 Durability when treated to the requirements of Acceptable Solution B2/AS1 and/or NZS 3602, Timber and Wood-based Products for use in Building.
This building is intended to be partially lined on the inside with plasterboard and as such forms part of the Acceptable Solution B2/AS1, Clause 3.2.1(a), Section D, Members protected from the weather but with a risk of moisture penetration conducive to decay, Reference No. 1D.14, All other exterior wall framing.
Reference 1D.14 in NZS 3602 has no LVL entry therefore the Radiata Pine entry from Table 1A in B2/AS1 must be used; in this case noting minimum H1.2 treatment.

10.0 References

1. CHH Woodproducts New Zealand, *Futurebuild LVL Specific Design Guide, January 2014*.
2. Batchelor, M.L. (1984), *Improved Plywood Gussets for Timber Portal Frames*, Proceedings of the Pacific Timber Engineering Conference, Auckland 1984, Paper No. 185B.
3. Hutchings B.F (1989), *Moment Joist Design*, Design, Construct and Detailing in Timber Conference, 15-17 May, 1992, Timber Development Association (NSW) Ltd.
4. Hutchings B.F and Bier H (1999), *Timber Engineering Design Made More Accessible*, Australasian Structural Engineering Conference, Auckland, 1999.
5. Milner H.R (1987), *The Design and Construction of Timber Portal Frames*, Chisolm Institute of Technology
6. Milner H.P and Crozier D.A (2000), *Structural Design of Timber Portal Frame Buildings,* Engineers Australia Pty Ltd.
7. National Association of Forest Industries, Timber Datafile SS1, *Timber Portal Frames*, National Association of Forest Industries
8. Standards Australia, *AS 1720.1-2010 Timber Structures, Part 1: Design methods*
9. Standards New Zealand, *NZS 3603:1993 Timber structures standard*

Appendix 1 – Frame Design Action Effect Diagrams

SERVICEABILITY ANALYSIS

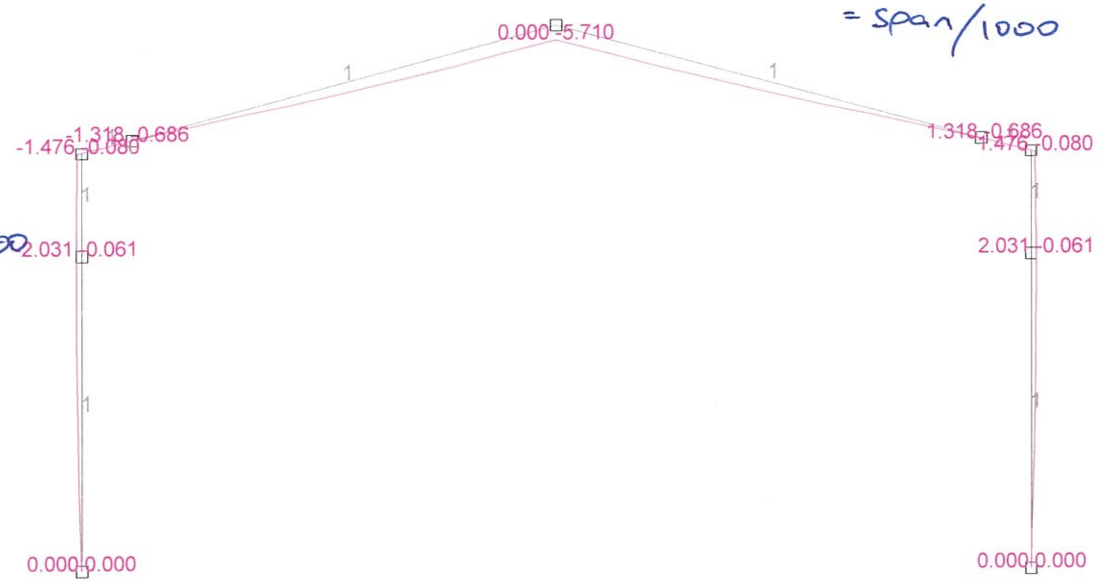
CHH Infortech NZ
Job: HF0014 12x4.8 all grids
12m x 4.8m hyFRAME

15 Dec 2017
03:12 p.m.

Load Cases:
1 P G (gy=-9.81)

Sections:
1 600x75

$j_2 = 2.0$ for permanent loads



$\Delta_G = 2 \times 5.7 = 11.4mm$
$= span/1000$

$\Delta_G = 2 \times 1.5 = 3.0mm$
$= height/1700$

theta: 270 phi: 0

Displaced Shape

serv.

CHH Infortech NZ
Job: HF0014 12x4.8 all grids
12m x 4.8m hyFRAME

15 Dec 2017
03:12 p.m.

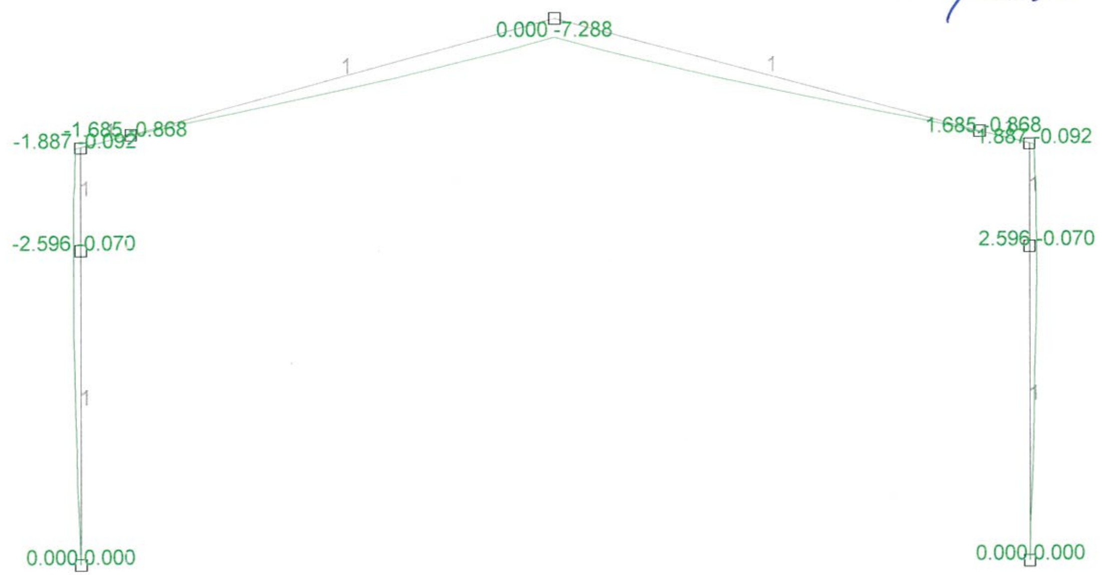
Load Cases:
2 P Q

Sections:
1 600x75

$\psi_s = 0.7$ for L.L.

$\Delta \psi_s Q = 0.7 \times 7.3 = 5.1$mm
$=$ span/2230

$\Delta \psi_s Q = 0.7 \times \cancel{}\ 1.9$
$= 1.3$mm
$=$ height/3800



theta: 270 phi: 0

Displaced Shape

serv.

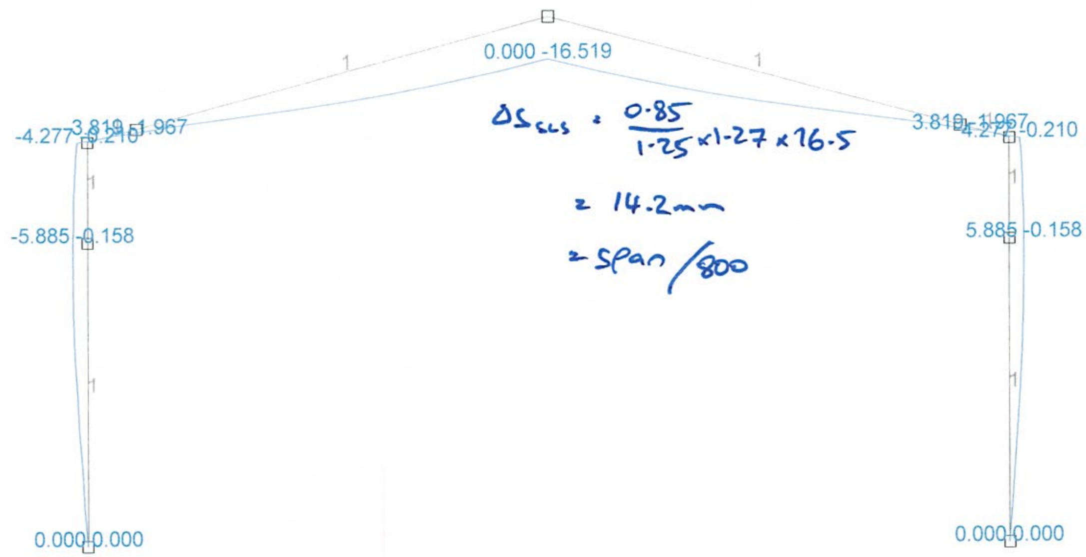
Microstran [V8.11x]

\\dnzchhnas\vandiej$\UserSettings\Documents\Current projects\HF0014 Matakana - Structural Design\HF0014 12x4.8 all grids

Load Cases:
3 P Su

Sections:
1 600x75

$j_2 = 1.27$ (for 5 days)
→ sub-alpine snow



$\Delta S_{sus} = \frac{0.85}{1.25} \times 1.27 \times 16.5$

$= 14.2mm$

$= span/800$

$\Delta S_{sus} = \frac{0.85}{1.25} \times 1.27 \times 4.3$

$= 3.7mm$

$= height/1354$

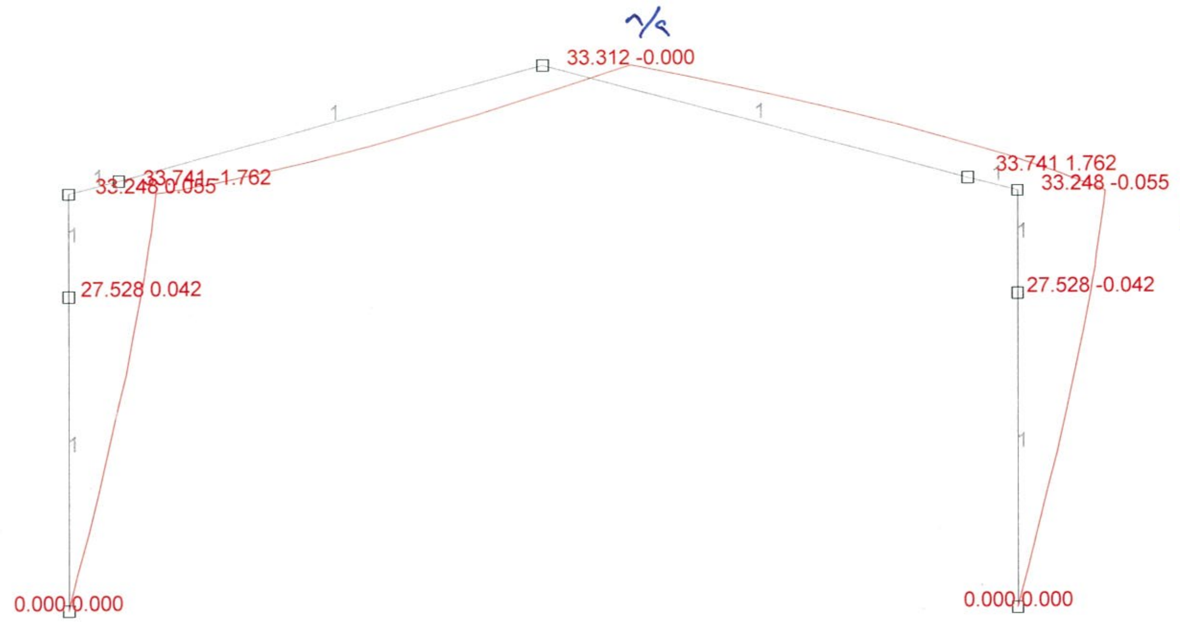
Displaced Shape

Serviceability

Load Cases:
4 P Eq

Sections:
1 600x75

$$\frac{C_d(T)_{SLS}}{C_d(T)_{ULS}} = 0.22 \text{ (ref. appendix 4)}$$



$\Delta E_{q\,SLS} = 0.22 \times 33.2$
$= 7.3\text{mm}$
$= \text{height}/690$

theta: 270 phi: 0

Displaced Shape

serv.

CHH Infortech NZ
Job: HF0014 12x4.8 all grids
12m x 4.8m hyFRAME

15 Dec 2017
03:12 p.m.

Load Cases:
5 P Wu - Lat

Sections:
1 600x75

$$\Delta_{wind\ SLS} = \left(\frac{V_{SLS}}{V_{500}}\right)^2 \times \Delta_{wind\ ULS}$$

$$= \left(\frac{37}{45}\right)^2 \times \Delta_{ULS}$$

$$= 0.676 \times \Delta_{ULS}$$

$$\Delta_{W_{SLS}} = 0.676 \times 24.9$$

$$= 16.8\ mm$$

(span/680)

$\Delta_{W_{SLS}} = 30.7\ mm$

(~~span~~ height/160)

theta: 270 phi: 0

serv.

Displaced Shape

Microstran [V8.11x]

\\dnzchhnas\vandiej$\UserSettings\Documents\Current projects\HF0014 Matakana - Structural Design\HF0014 12x4.8 all grids

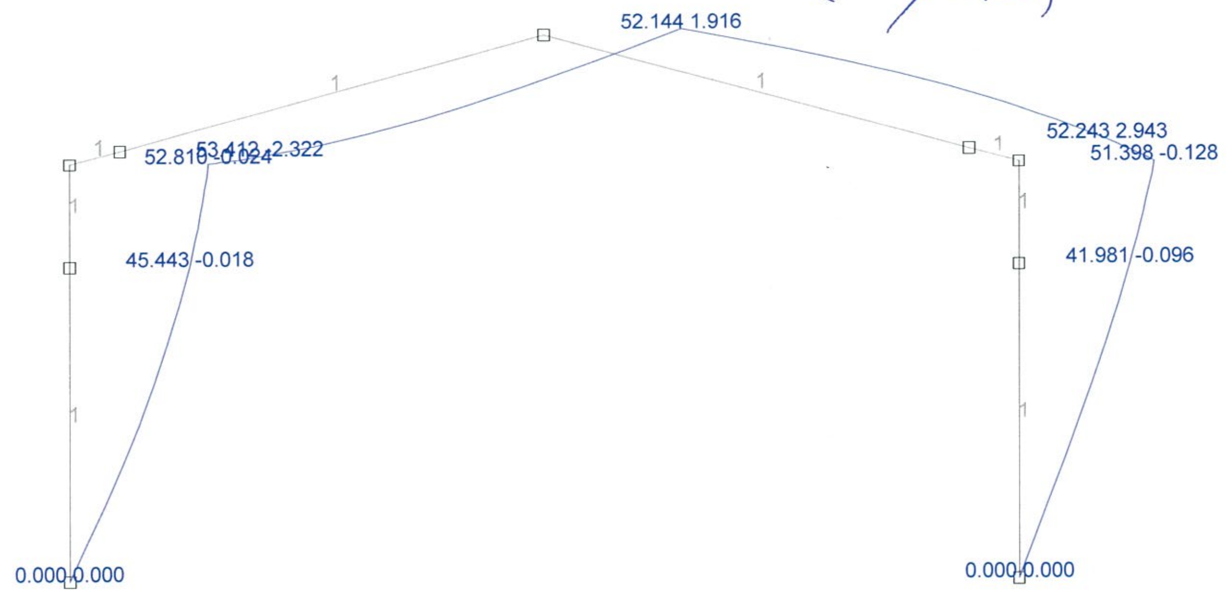
Load Cases:
6 P Wu - Lat

Sections:
1 600x75

$\Delta_{WSLS} = 0.676 \times 1.9 = 1.3$ mm
(span/8780)

$\Delta_{WSLS} = 36.1$ mm
= height/140
(critical)

OK



Y
Z X
theta: 270 phi: 0

serv.

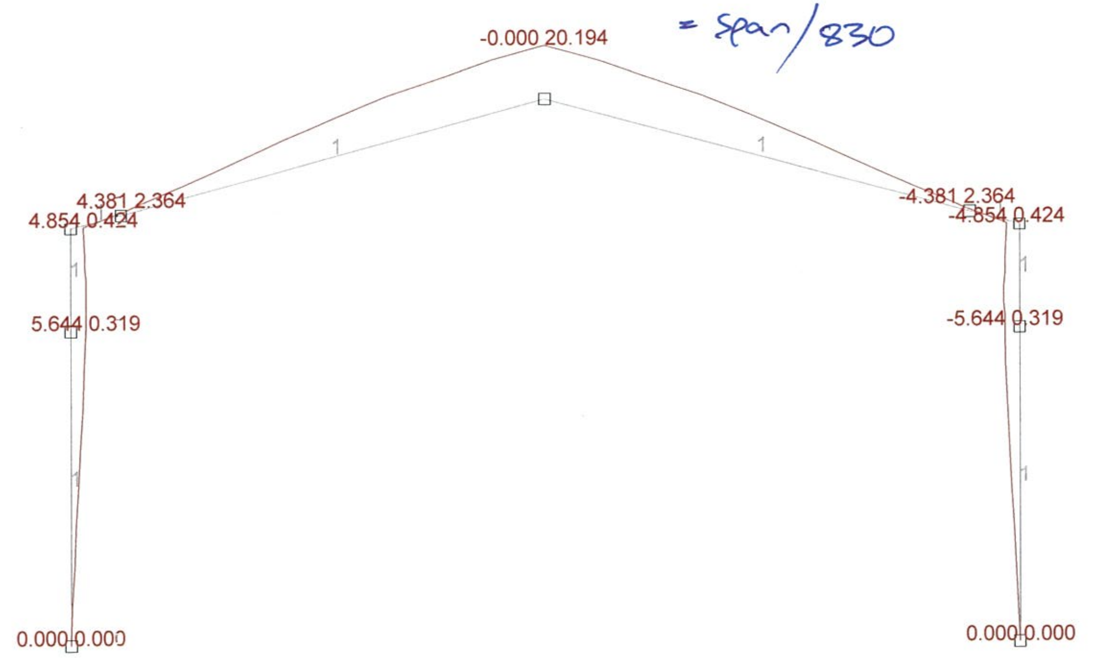
Displaced Shape

Load Cases:
7 P Wu - Long

Sections:
1 600x75

Δ_{wsls} = 13.6mm
= span/830

Δ_{wsls} = 3.3mm
= height/1530



theta: 270 phi: 0

serv.

Displaced Shape

CHH Infortech NZ
Job: HF0014 12x4.8 all grids
12m x 4.8m hyFRAME

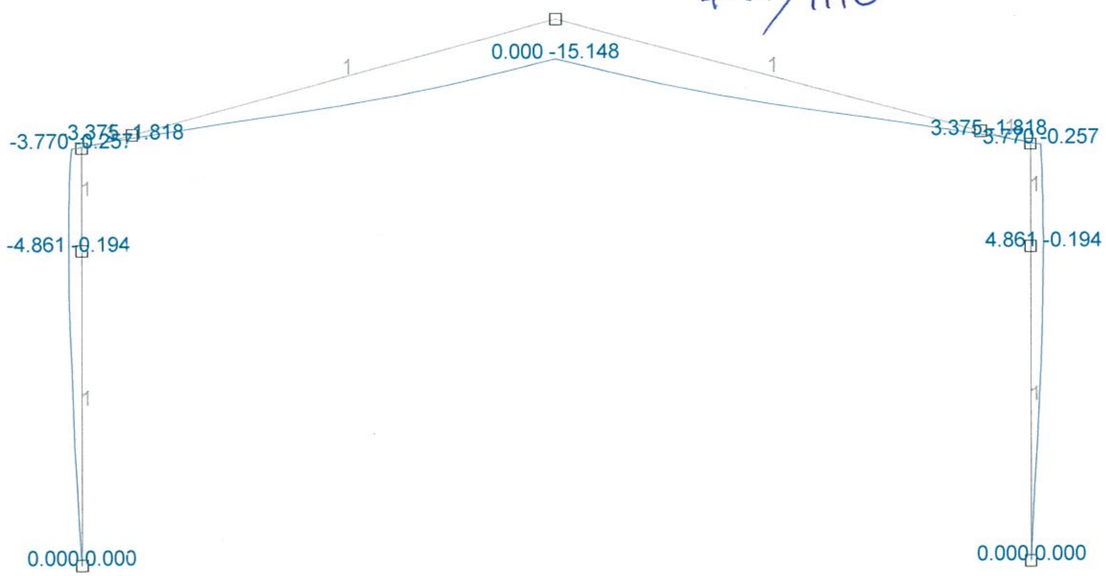
15 Dec 2017
03:12 p.m.

Load Cases:
8 P Wu - Long

Sections:
1 600x75

$\Delta W_{sls} = 10.2mm$
$= span/1110$

$\Delta W_{sls} = 2.5mm$
$= height/1970$



Y
Z X
theta: 270 phi: 0

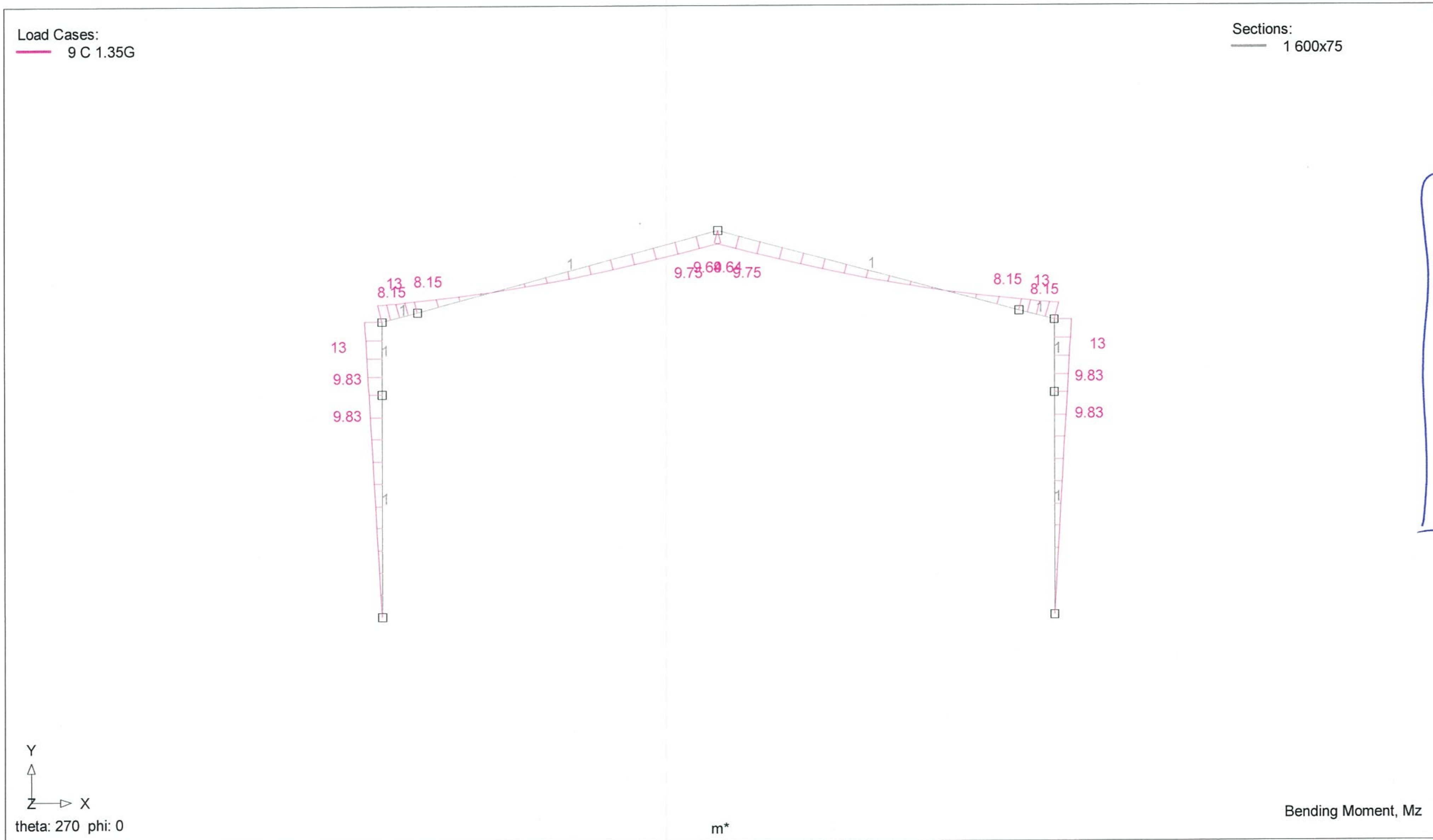
serv.

Displaced Shape

Microstran [V8.11x]

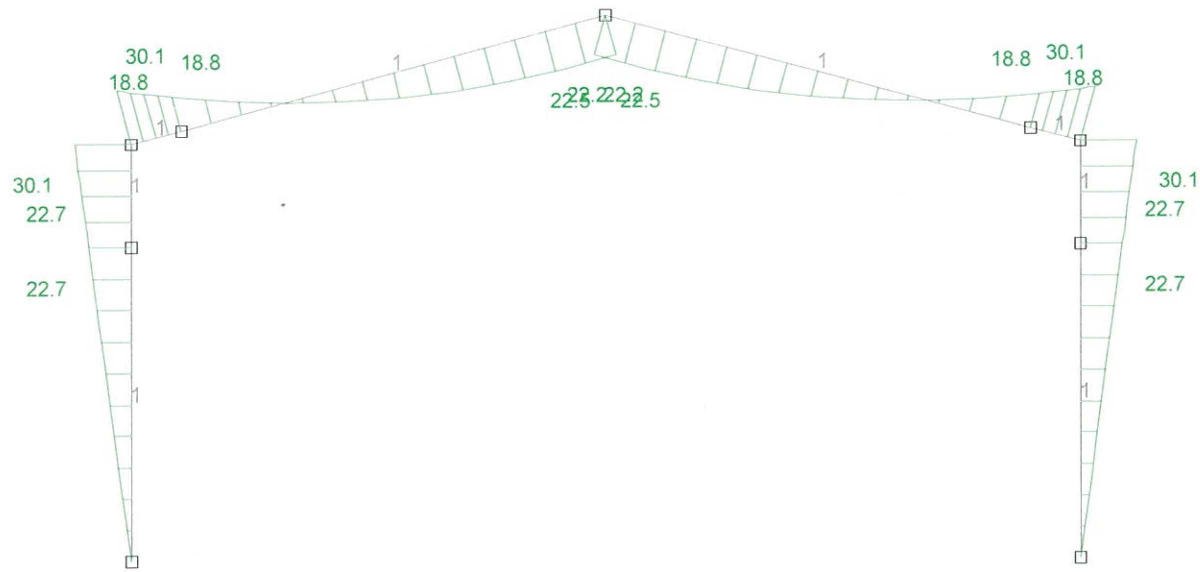
\\dnzchhnas\vandiej$\UserSettings\Documents\Current projects\HF0014 Matakana - Structural Design\HF0014 12x4.8 all grids

ULS ANALYSIS:



Load Cases:
10 C 1.2G+1.5Q

Sections:
1 600x75



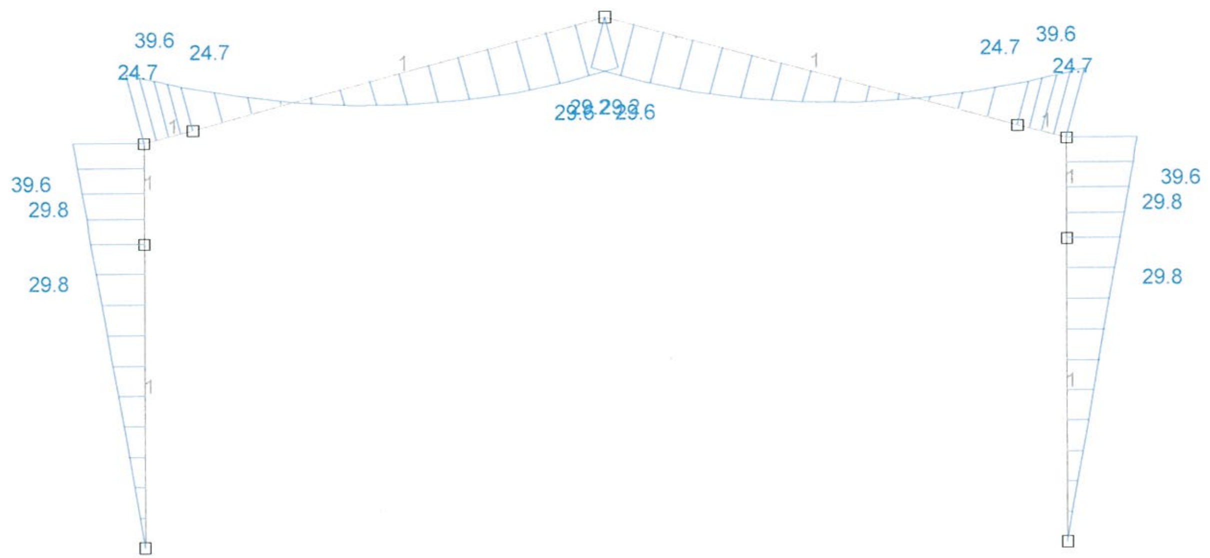
Y
Z X
theta: 270 phi: 0

m*

Bending Moment, Mz

Load Cases:
11 C 1.2G+shi.Q+Su

Sections:
1 600x75



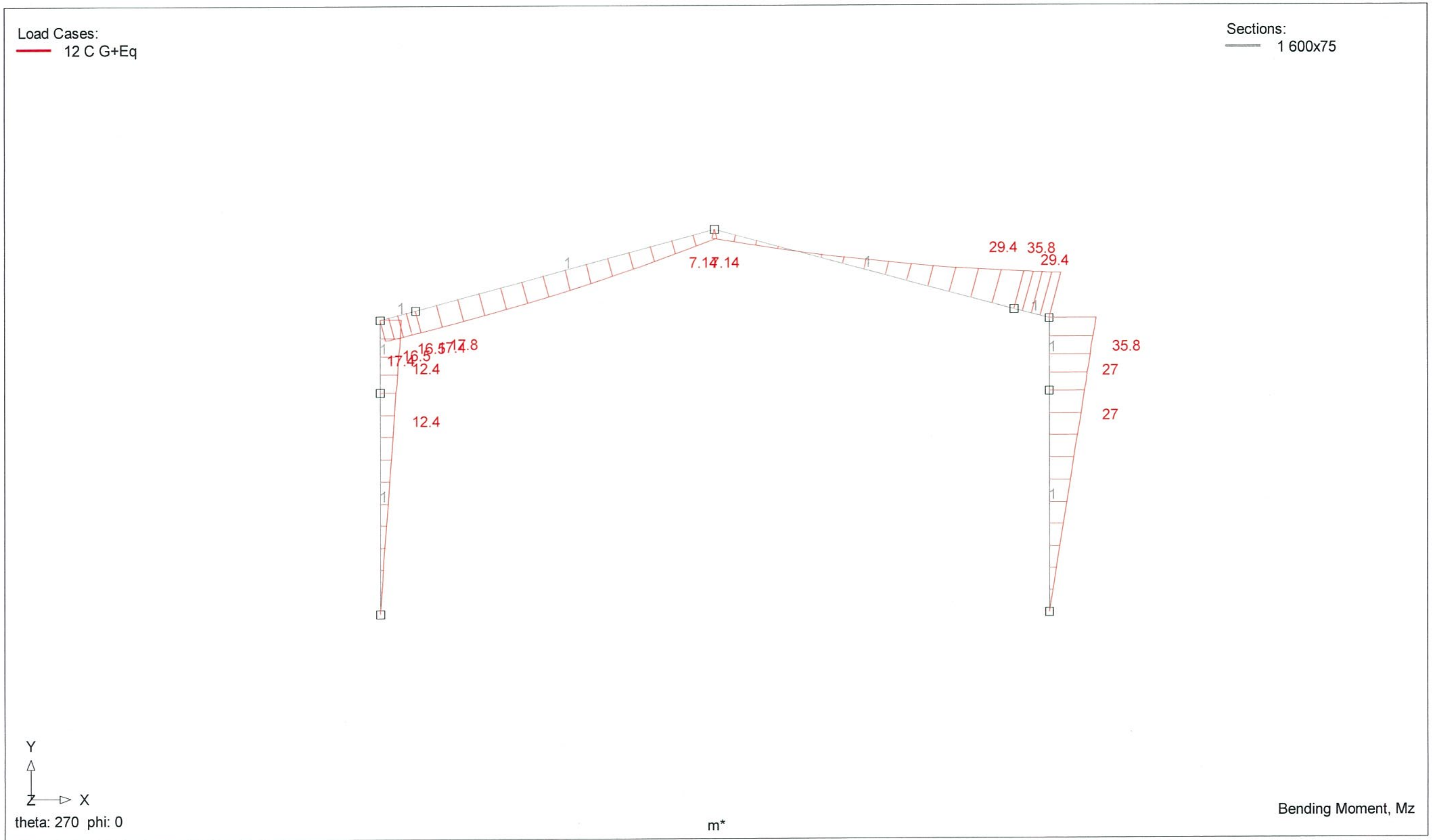
theta: 270 phi: 0

M*

Bending Moment, Mz

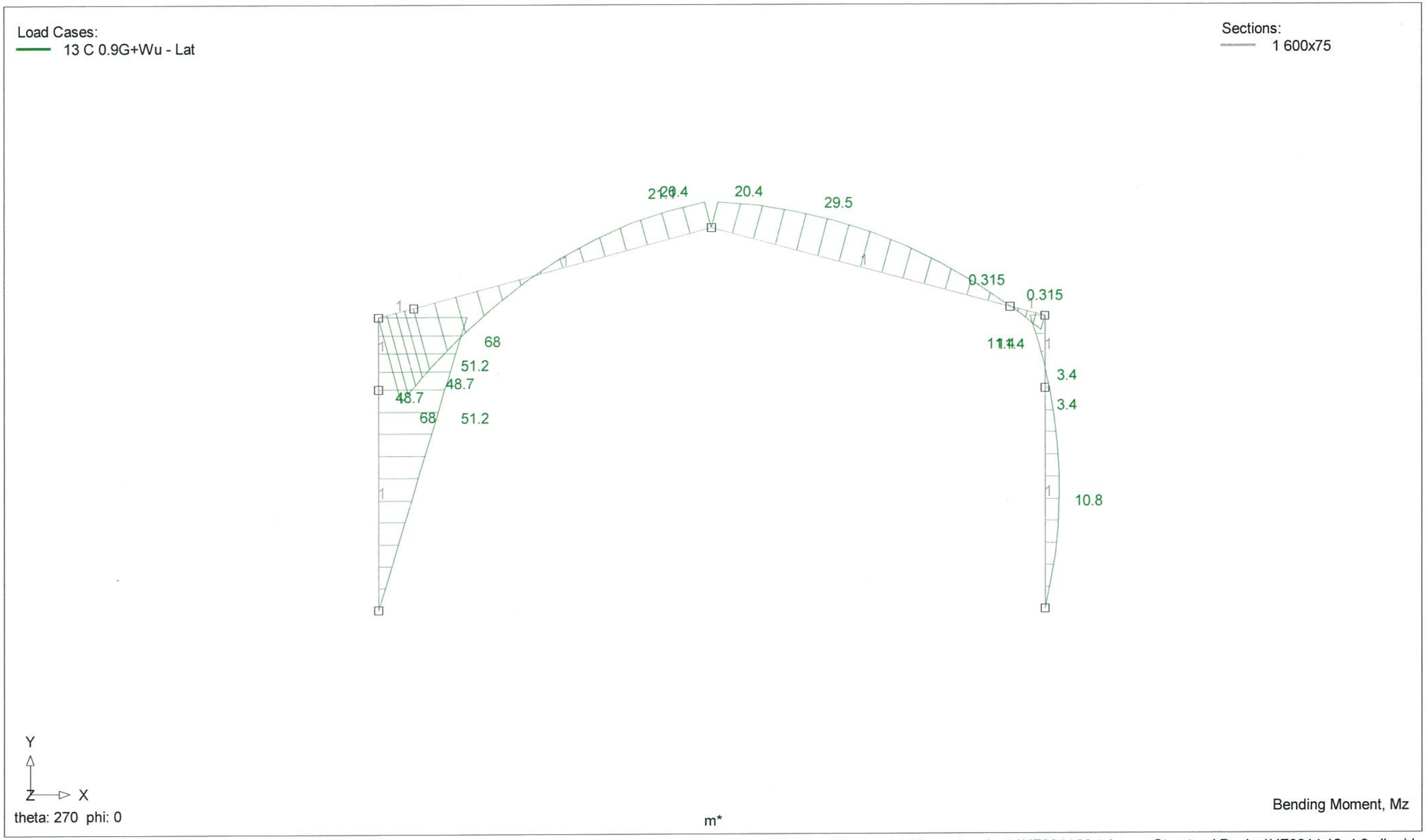
CHH Infortech NZ
Job: HF0014 12x4.8 all grids
12m x 4.8m hyFRAME

15 Dec 2017
03:10 p.m.



CHH Infortech NZ
Job: HF0014 12x4.8 all grids
12m x 4.8m hyFRAME

15 Dec 2017
03:11 p.m.

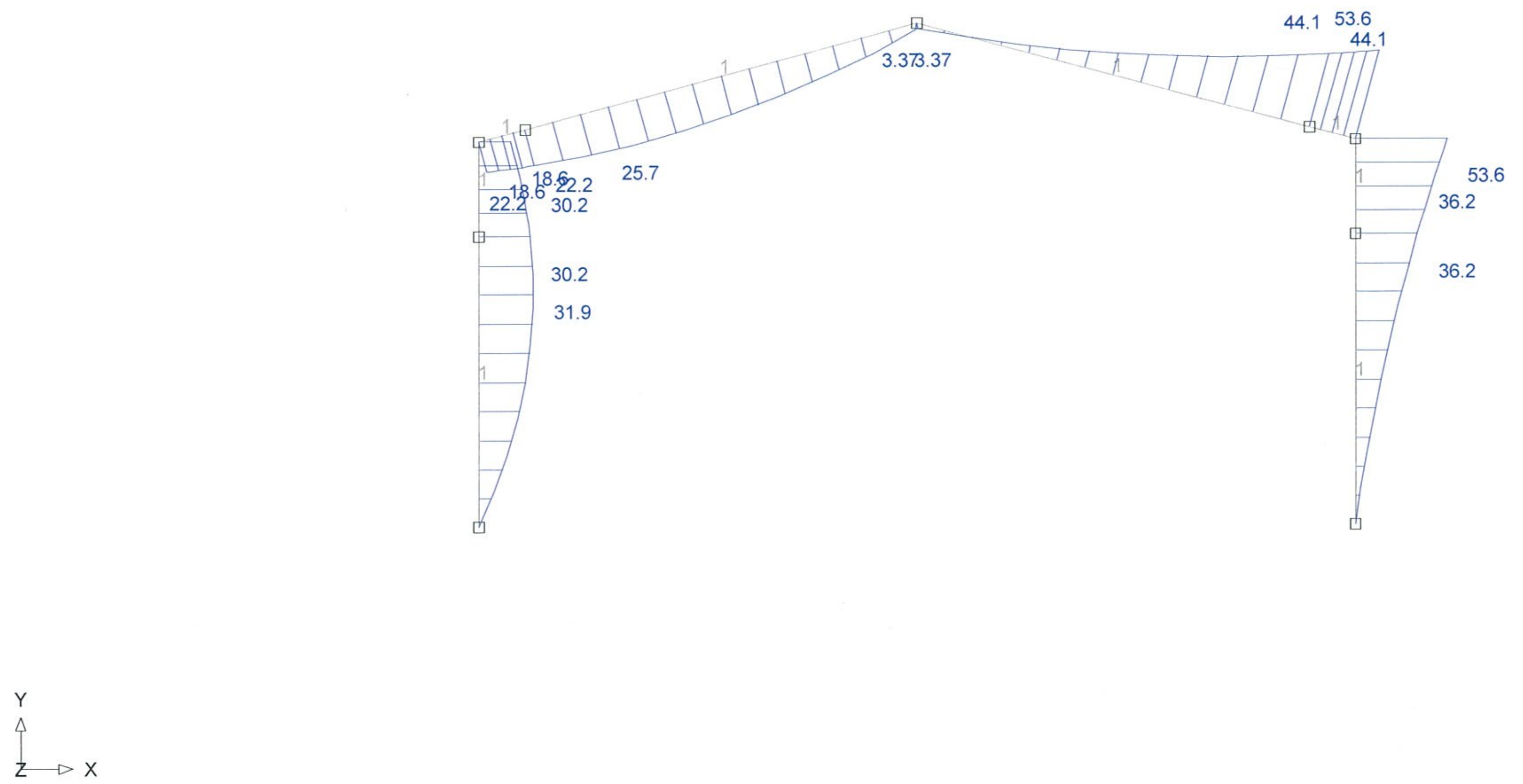


Microstran [V8.11x]

\\dnzchhnas\vandiej$\UserSettings\Documents\Current projects\HF0014 Matakana - Structural Design\HF0014 12x4.8 all grids

Load Cases:
14 C 1.2G+Wu - Lat

Sections:
1 600x75



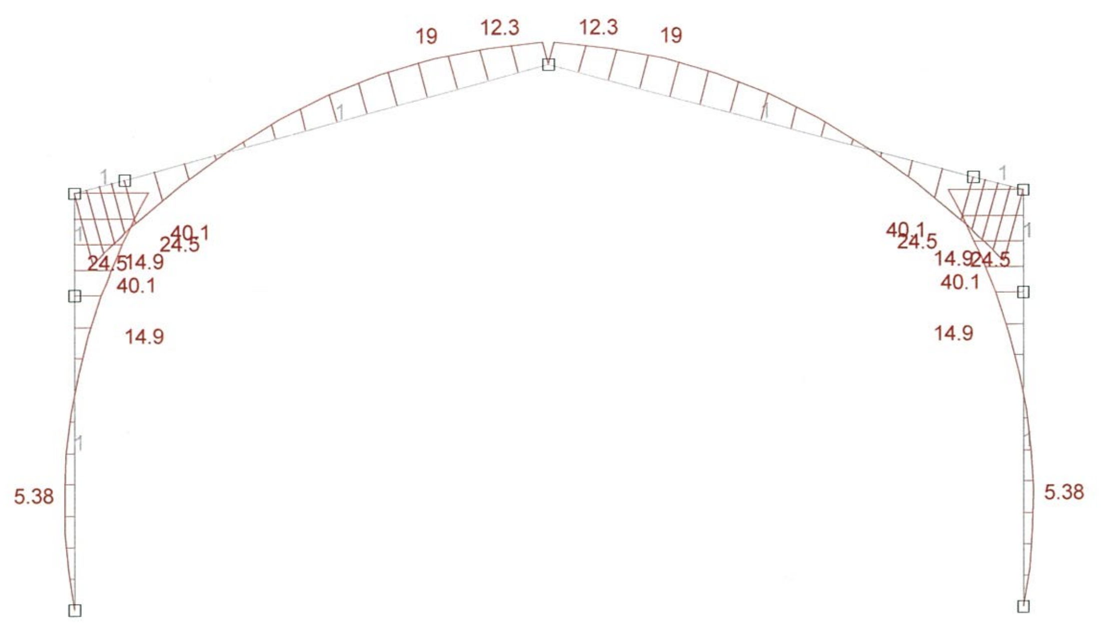
theta: 270 phi: 0

m*

Bending Moment, Mz

Load Cases:
15 C 0.9G+Wu - Long

Sections:
1 600x75



theta: 270 phi: 0

m*

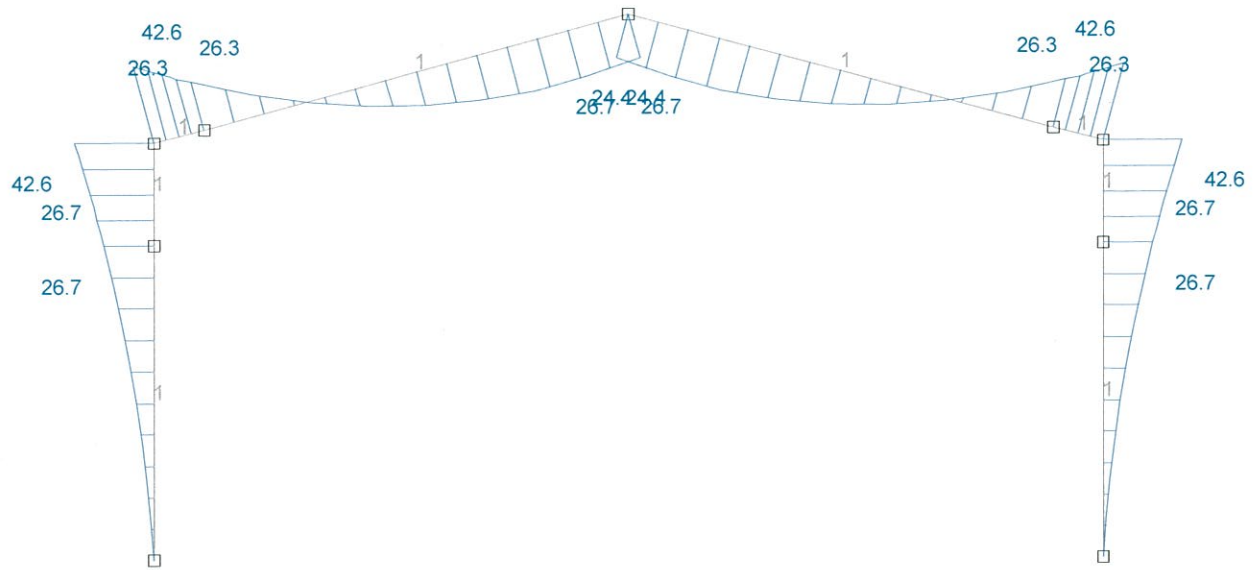
Bending Moment, Mz

CHH Infortech NZ
Job: HF0014 12x4.8 all grids
12m x 4.8m hyFRAME

15 Dec 2017
03:11 p.m.

Load Cases:
16 C 1.2G+Wu - Long

Sections:
1 600x75



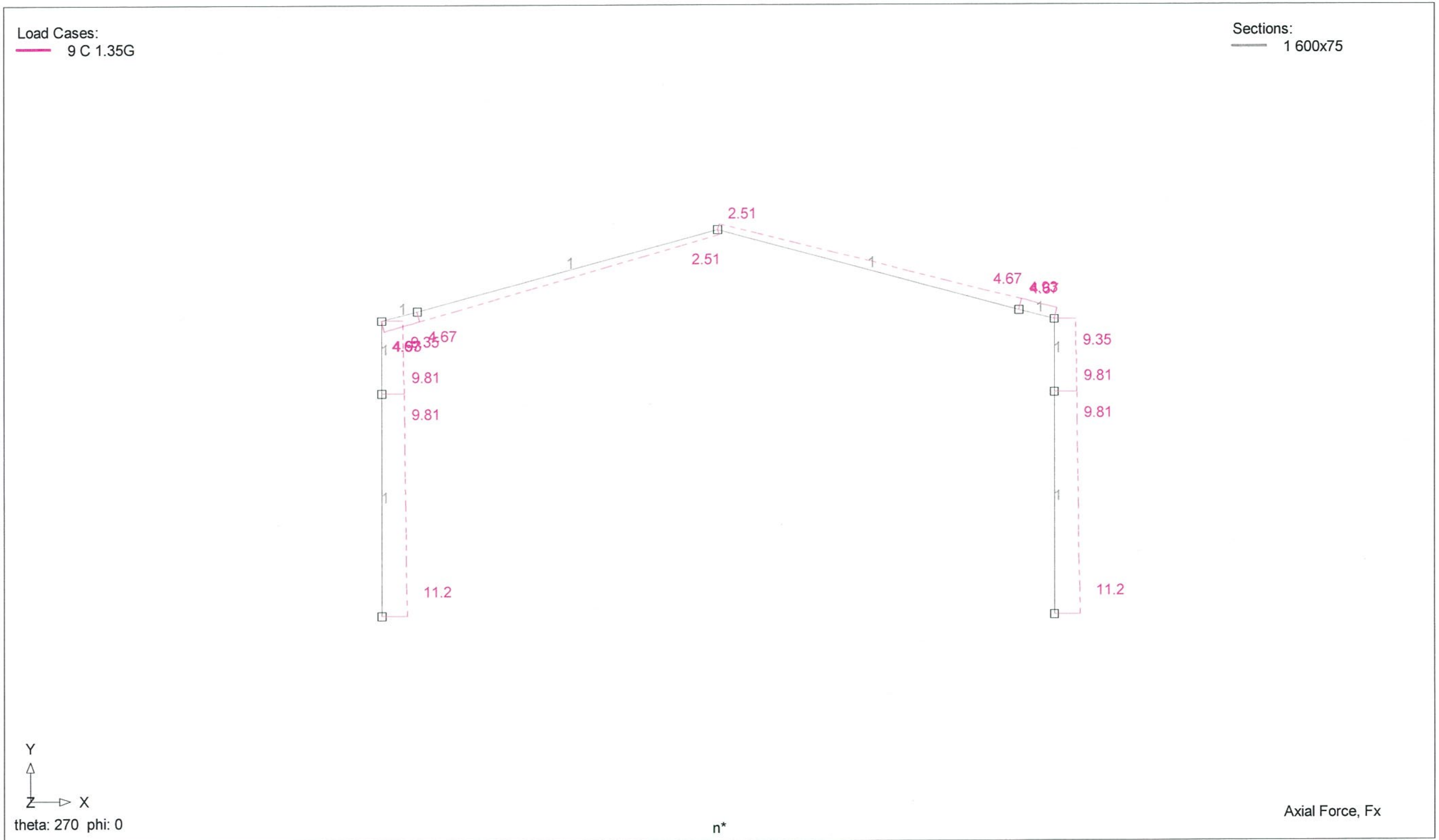
Y
Z X
theta: 270 phi: 0

m*

Bending Moment, Mz

Microstran [V8.11x]

\\dnzchhnas\vandiej$\UserSettings\Documents\Current projects\HF0014 Matakana - Structural Design\HF0014 12x4.8 all grids

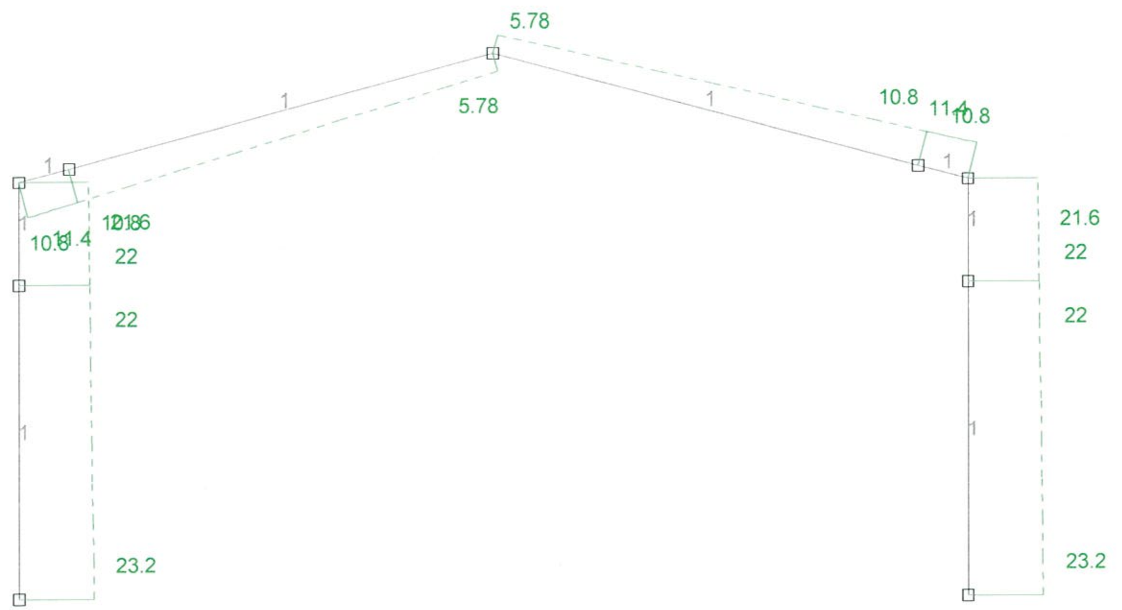

Load Cases:
9 C 1.35G
Sections:
1 600x75
2.51
2.51
4.67
4.67
9.35
9.35
9.81
9.81
9.81
9.81
11.2
11.2
Y
X
Z
theta: 270 phi: 0
n*
Axial Force, Fx

CHH Infortech NZ
Job: HF0014 12x4.8 all grids
12m x 4.8m hyFRAME

15 Dec 2017
03:11 p.m.

Load Cases:
10 C 1.2G+1.5Q

Sections:
1 600x75



Y
X
Z
theta: 270 phi: 0

Axial Force, Fx

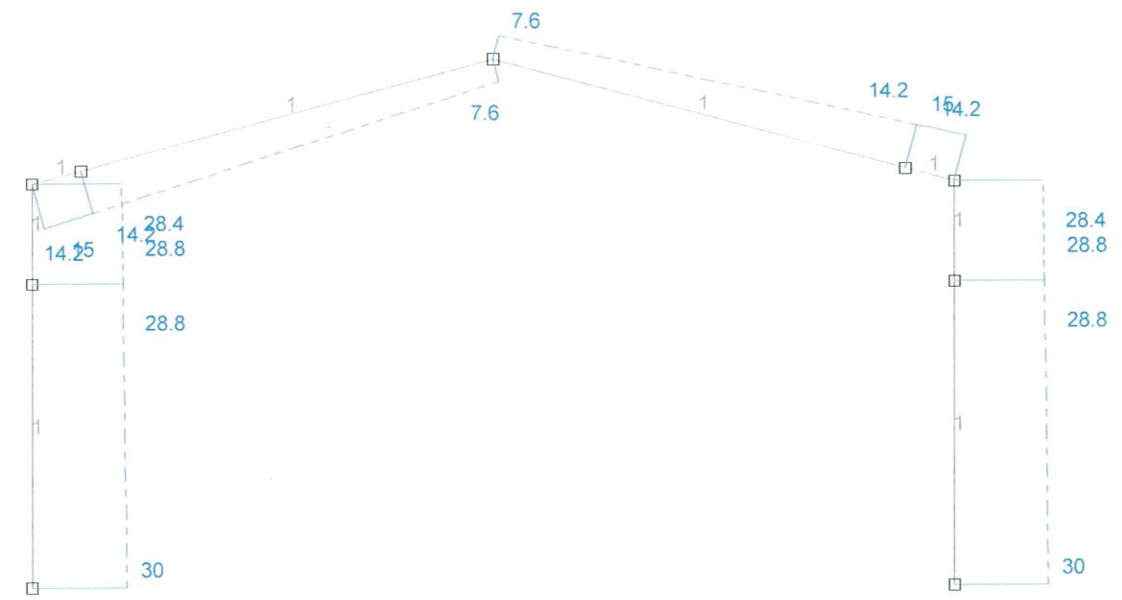
n*

Microstran [V8.11x]

\\dnzchhnas\vandiej$\UserSettings\Documents\Current projects\HF0014 Matakana - Structural Design\HF0014 12x4.8 all grids

Load Cases:
11 C 1.2G+shi.Q+Su

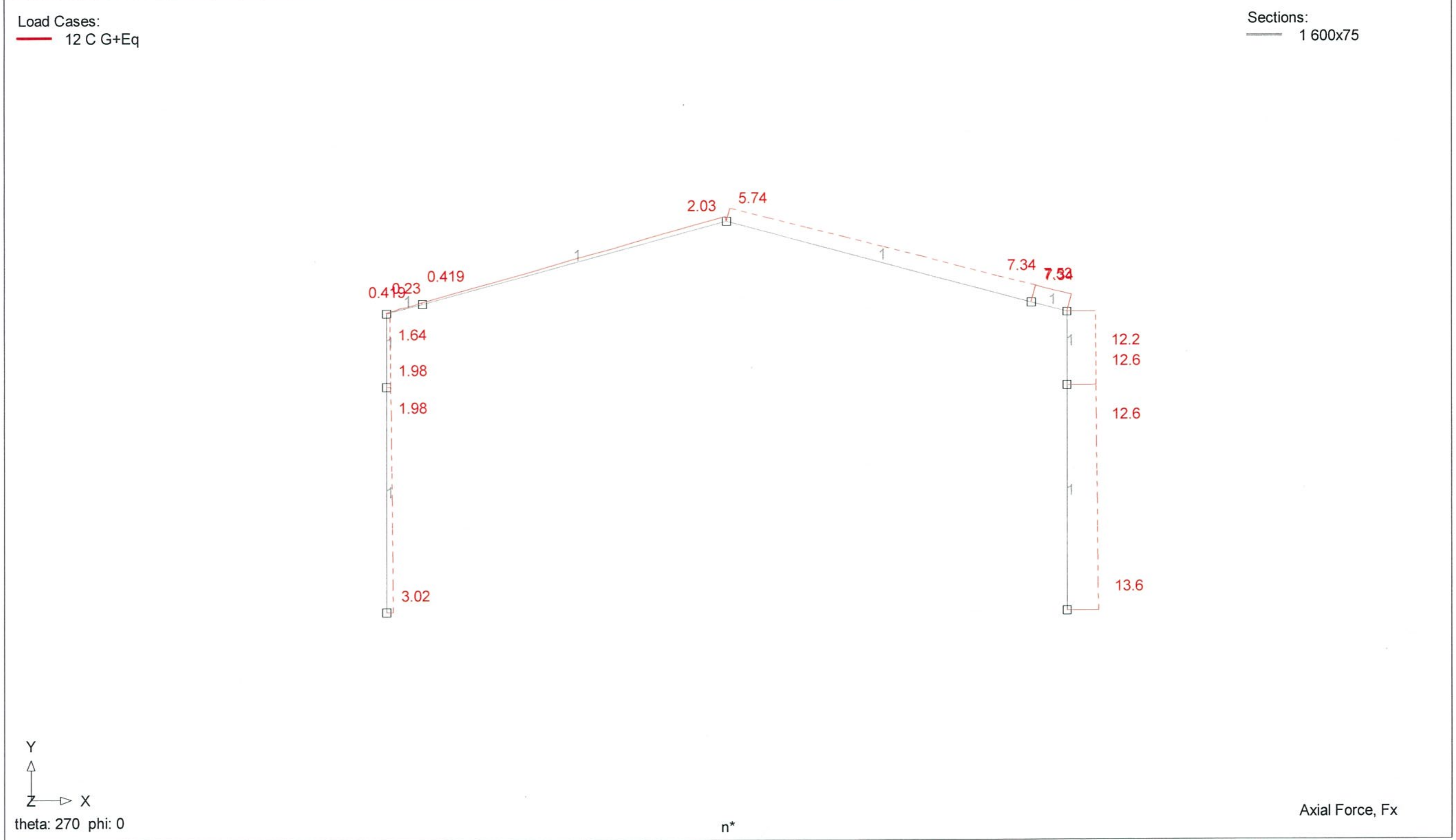
Sections:
1 600x75

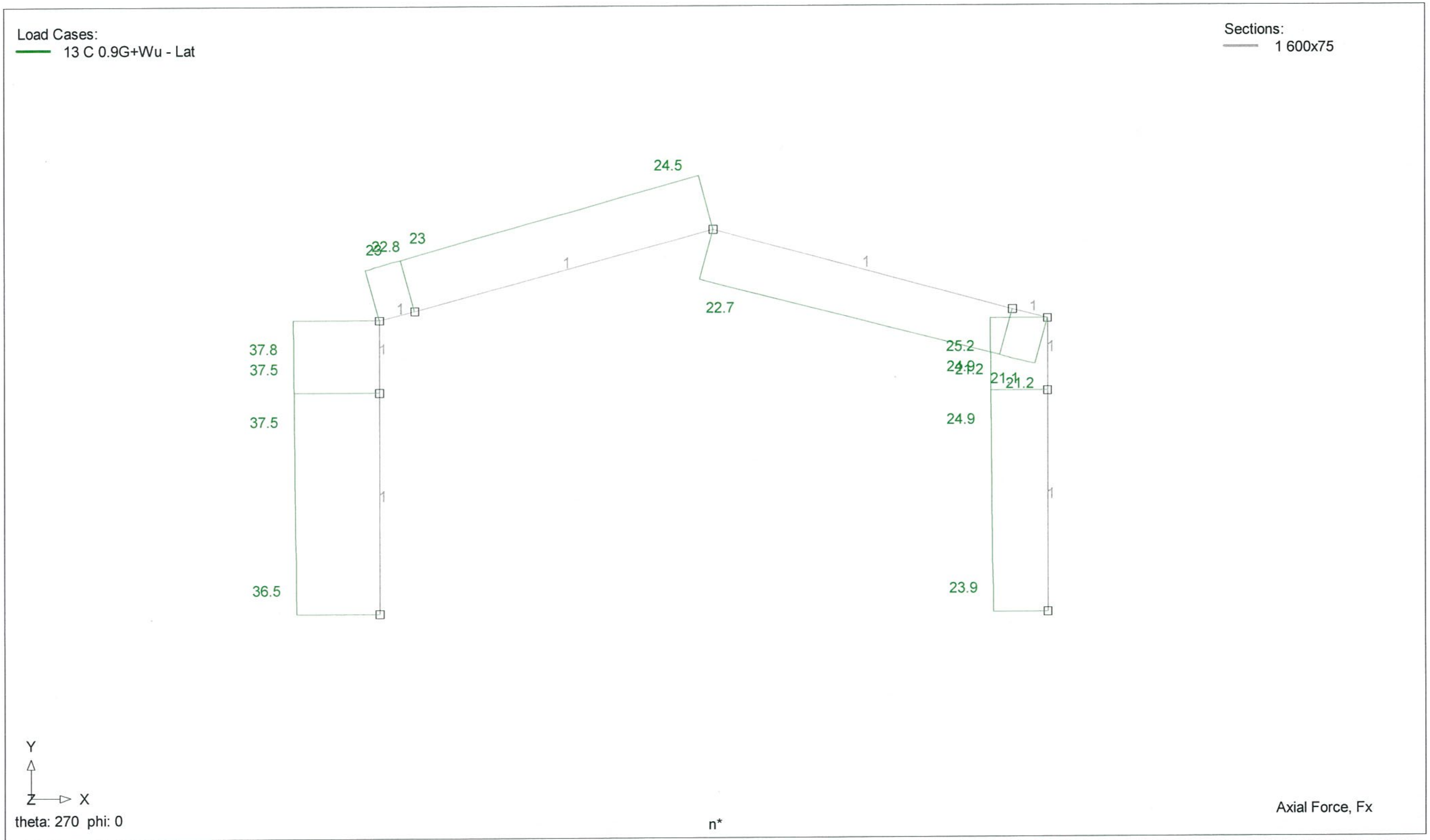


theta: 270 phi: 0

N*

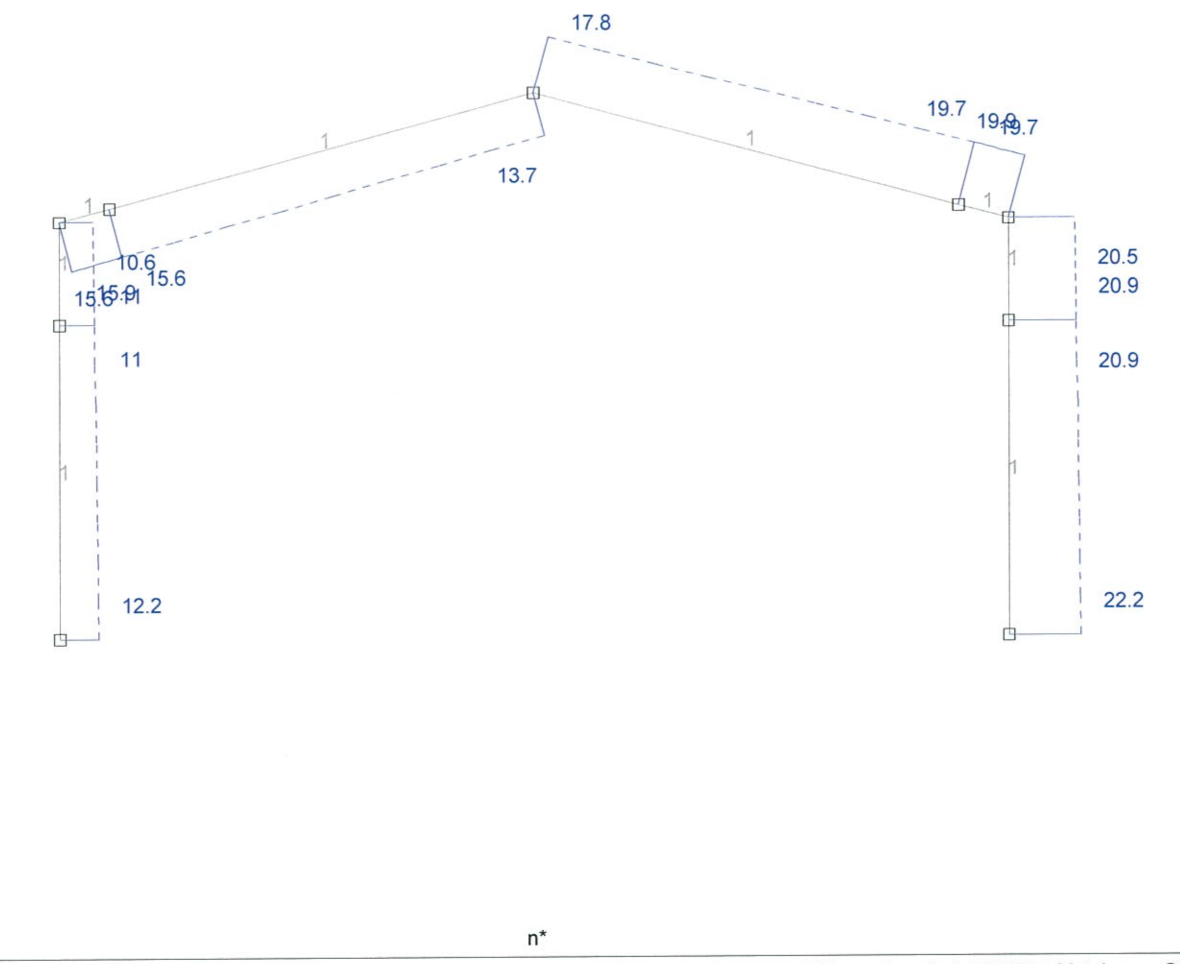
Axial Force, Fx


Load Cases:
12 C G+Eq
Sections:
1 600x75
2.03
5.74
0.419
0.419
0.23
7.34
7.53
7.34
1.64
1.98
1.98
3.02
12.2
12.6
12.6
13.6
Y
X
Z
theta: 270 phi: 0
n*
Axial Force, Fx

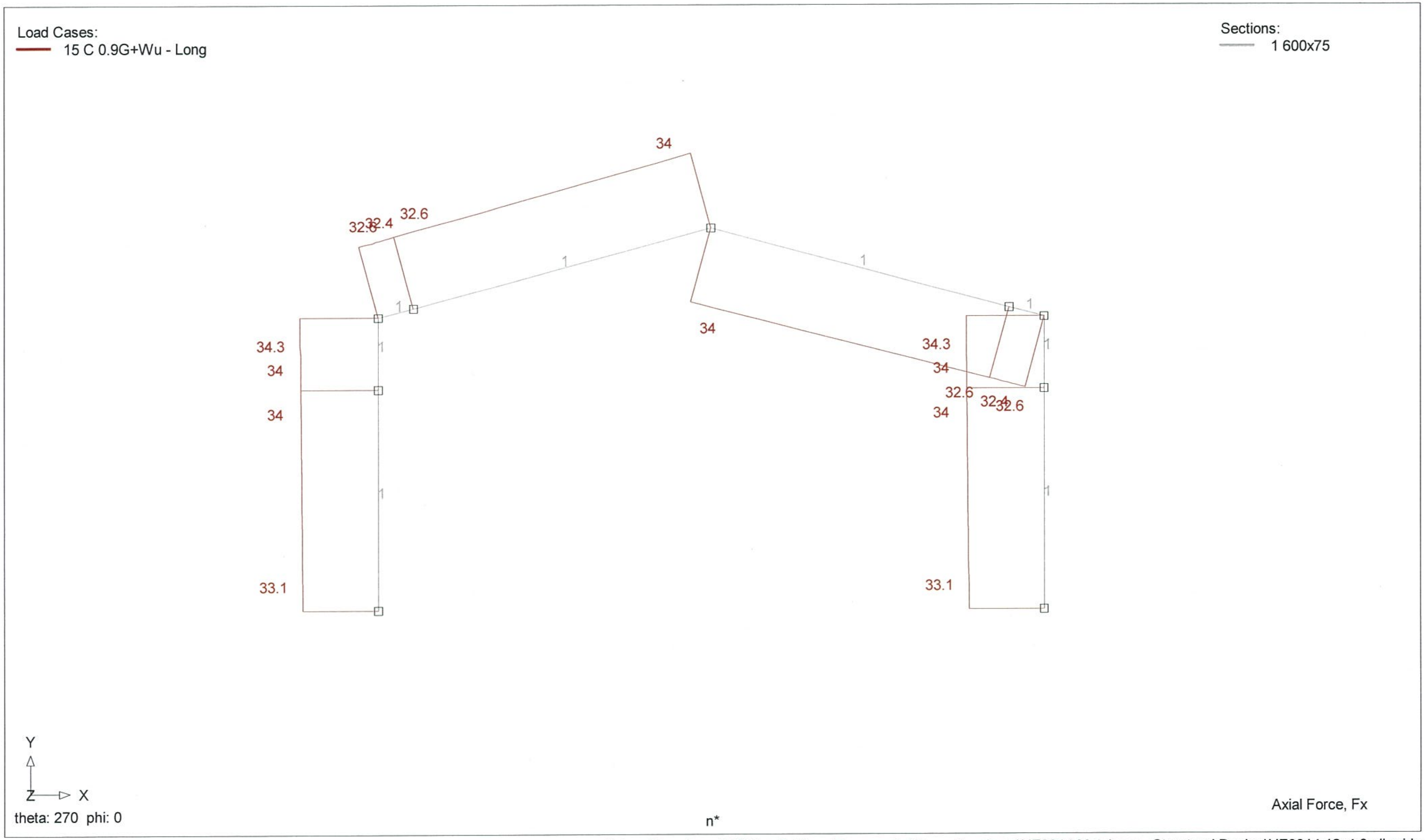

Load Cases:
13 C 0.9G+Wu - Lat
Sections:
1 600x75
24.5
22.8
23
23
22.7
25.2
24.9
21.2
21.1
21.2
37.8
37.5
37.5
36.5
24.9
23.9
Y
X
Z
theta: 270 phi: 0
Axial Force, Fx
n*

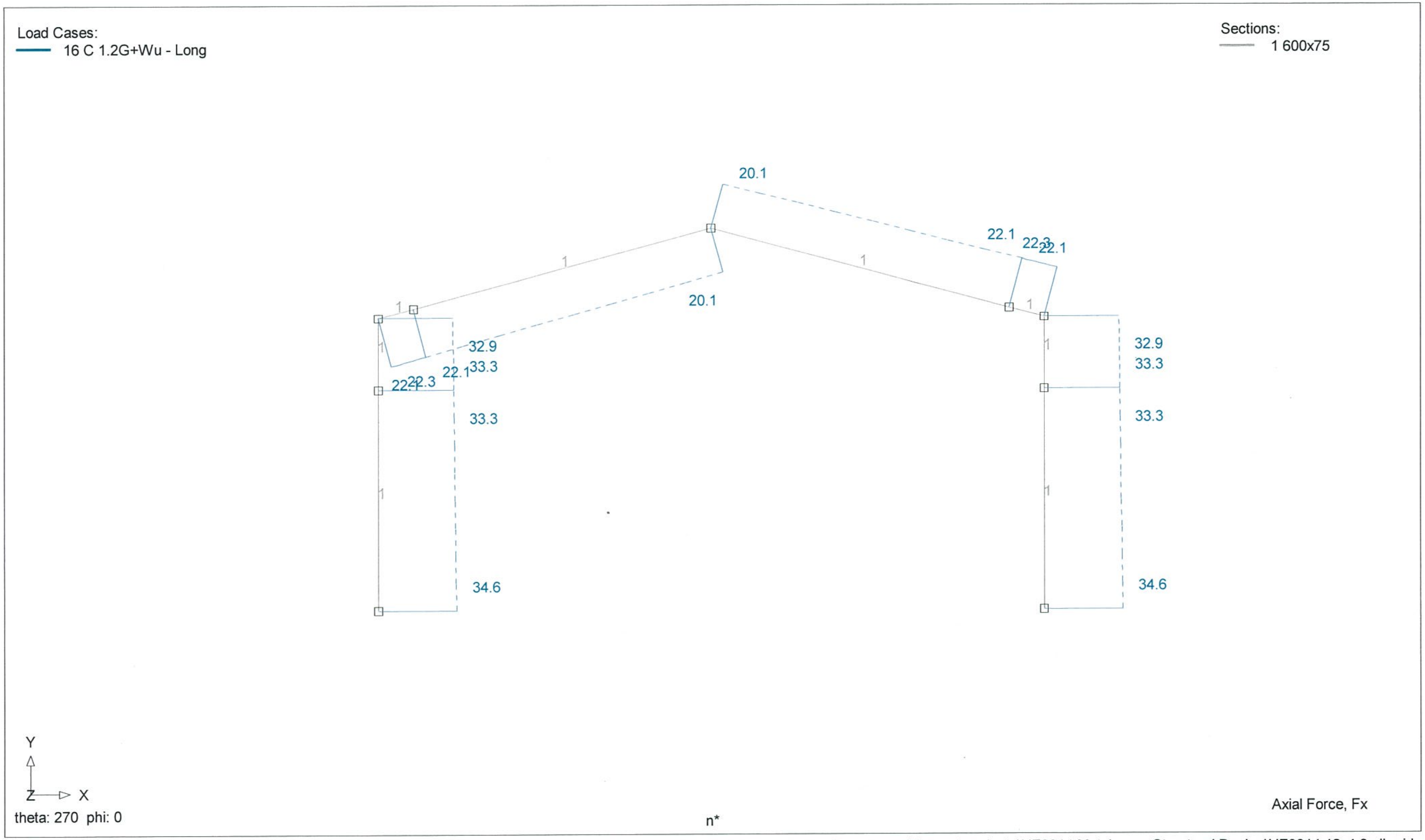
Load Cases:
14 C 1.2G+Wu - Lat

Sections:
1 600x75



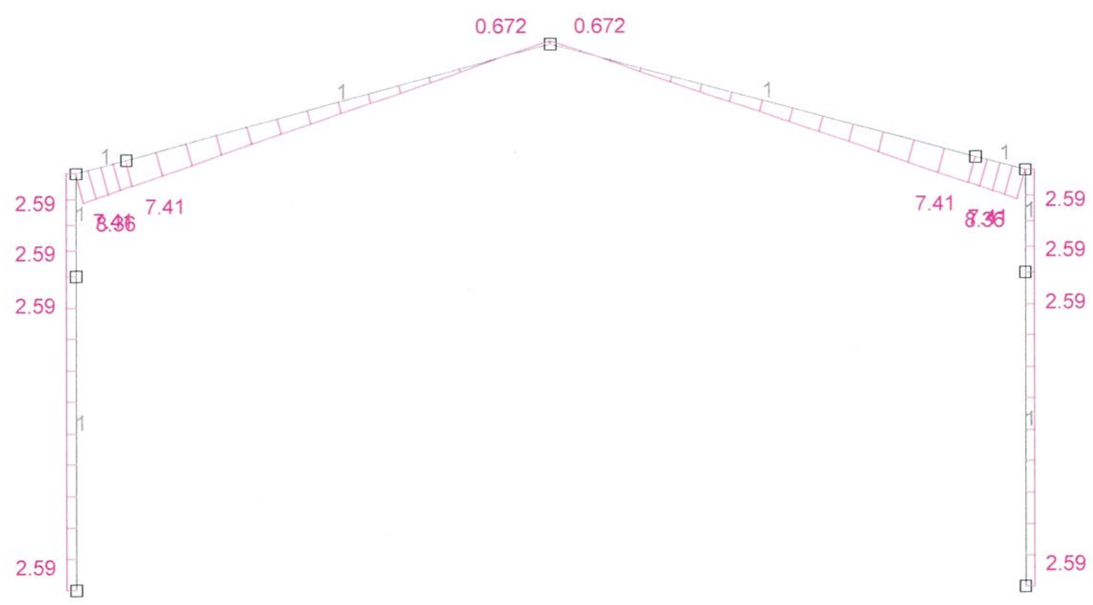
Axial Force, Fx


Load Cases:
15 C 0.9G+Wu - Long
Sections:
1 600x75
34
32.6
32.6
32.4
1
1
1
34.3
34
34
1
33.1
34
1
34.3
34
32.6
32.4
32.6
34
1
1
33.1
Y
Z
X
theta: 270 phi: 0
n*
Axial Force, Fx


Load Cases:
16 C 1.2G+Wu - Long
Sections:
1 600x75
20.1
20.1
22.1
22.3
22.1
32.9
33.3
33.3
34.6
22.1
22.3
22.1
32.9
33.3
33.3
34.6
1
Y
X
Z
theta: 270 phi: 0
n*
Axial Force, Fx

Load Cases:
9 C 1.35G

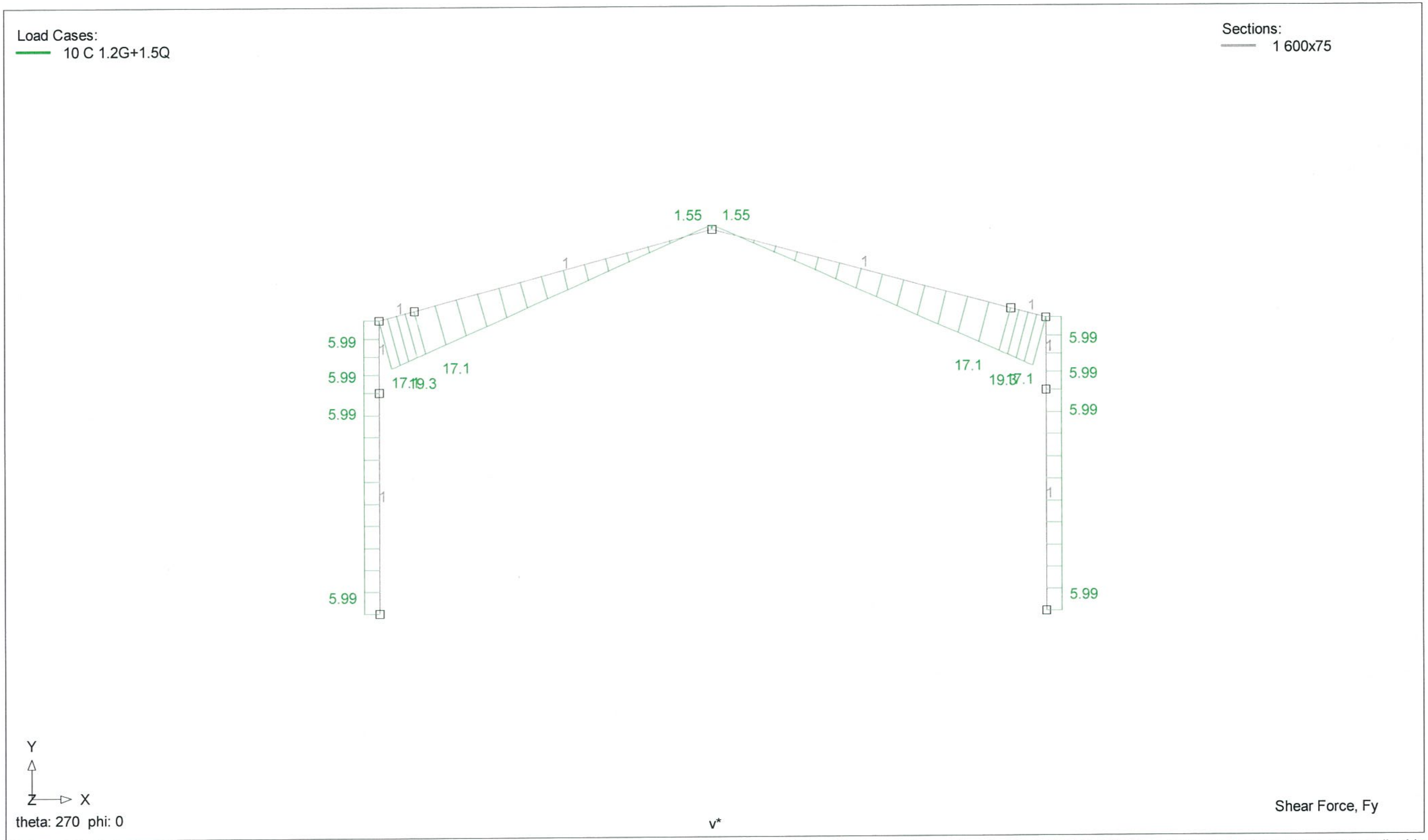
Sections:
1 600x75



theta: 270 phi: 0

v*

Shear Force, Fy

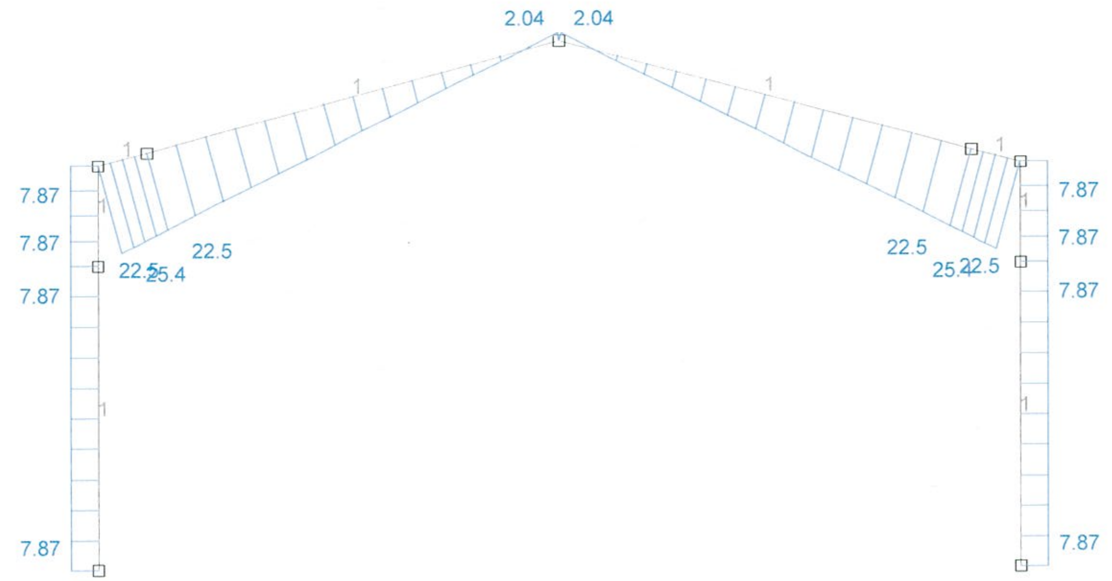

Load Cases:
10 C 1.2G+1.5Q
Sections:
1 600x75
1.55
1.55
1
1
1
1
1
1
5.99
5.99
5.99
5.99
5.99
5.99
5.99
5.99
17.1
17.1
17.19.3
19.17.1
Y
X
Z
theta: 270 phi: 0
v*
Shear Force, Fy

CHH Infortech NZ
Job: HF0014 12x4.8 all grids
12m x 4.8m hyFRAME

22 Nov 2017
05:07 p.m.

Load Cases:
11 C 1.2G+shi.Q+Su

Sections:
1 600x75



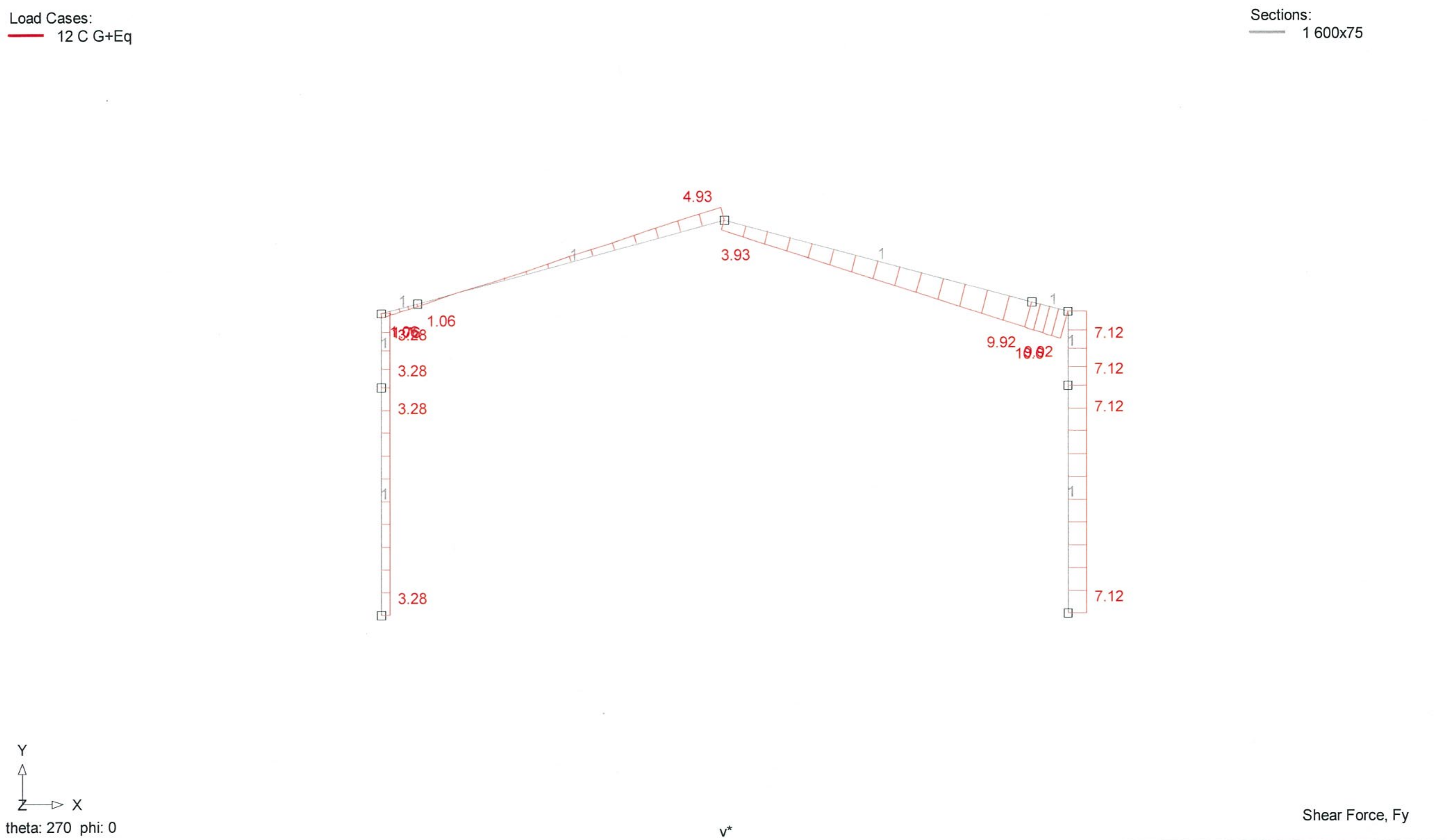
Y
Z X
theta: 270 phi: 0

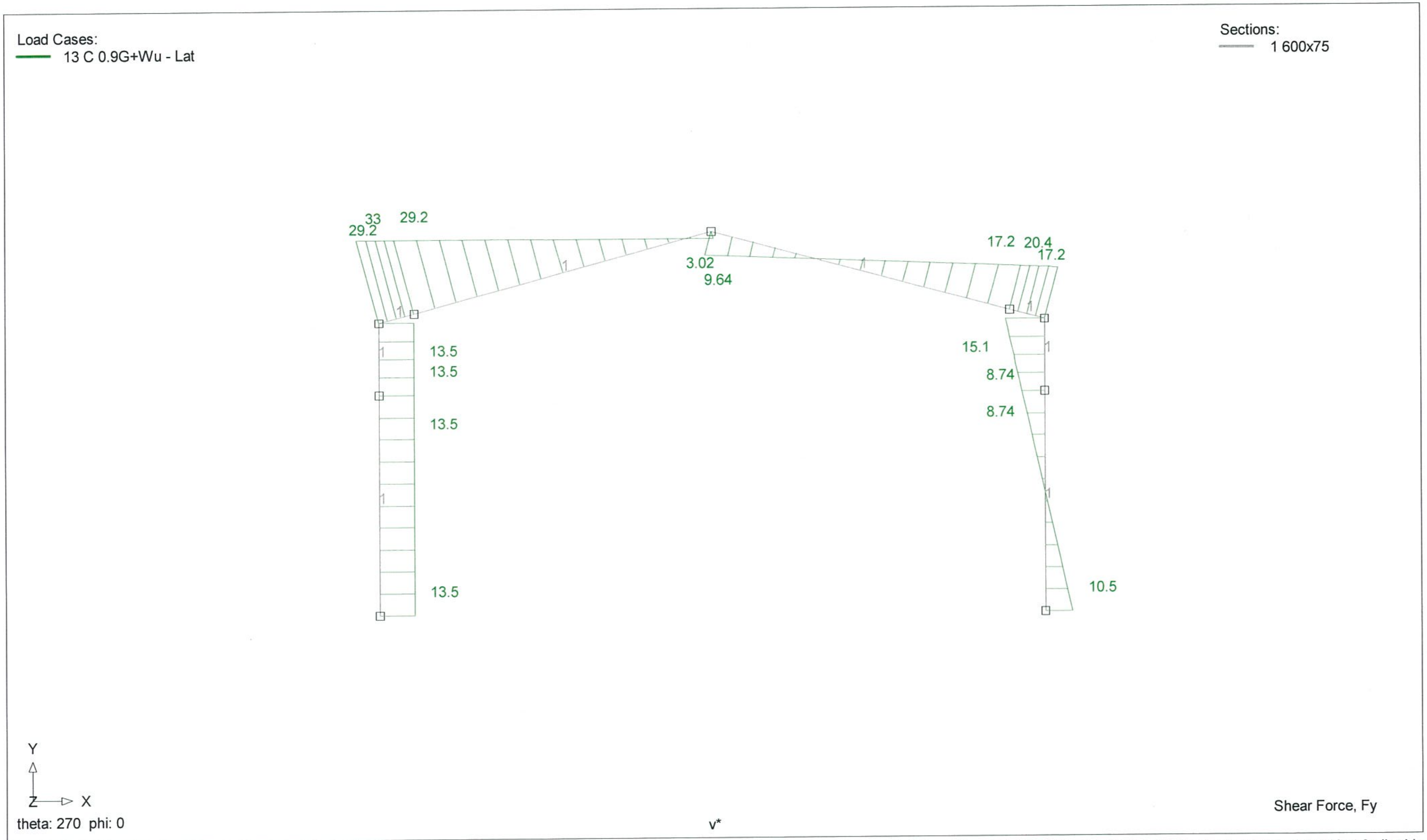
V*

Shear Force, Fy

Microstran [V8.11x]

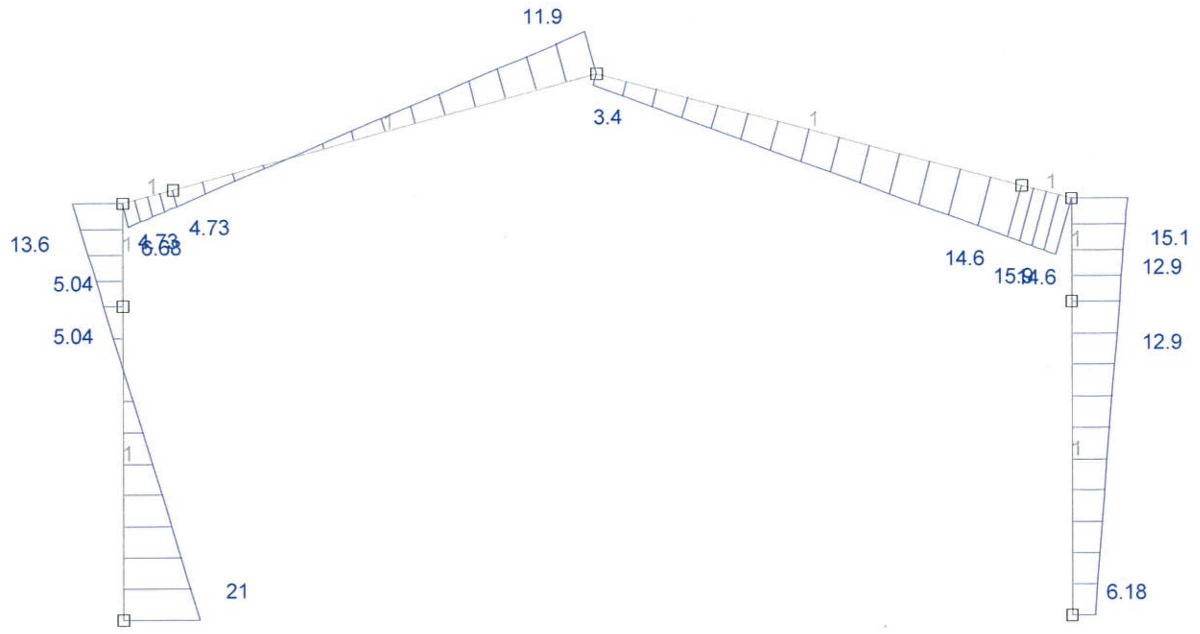
\\dnzchhnas\vandiej$\UserSettings\Documents\Current projects\HF0014 Matakana - Structural Design\HF0014 12x4.8 all grids


Load Cases:
12 C G+Eq
Sections:
1 600x75
4.93
3.93
1.06
3.28
3.28
3.28
9.92
7.12
7.12
7.12
7.12
Y
X
Z
theta: 270 phi: 0
v*
Shear Force, Fy


Load Cases:
13 C 0.9G+Wu - Lat
Sections:
1 600x75
33
29.2
29.2
3.02
9.64
17.2
20.4
17.2
13.5
13.5
13.5
13.5
15.1
8.74
8.74
10.5
Y
Z
X
theta: 270 phi: 0
v*
Shear Force, Fy

Load Cases:
14 C 1.2G+Wu - Lat

Sections:
1 600x75



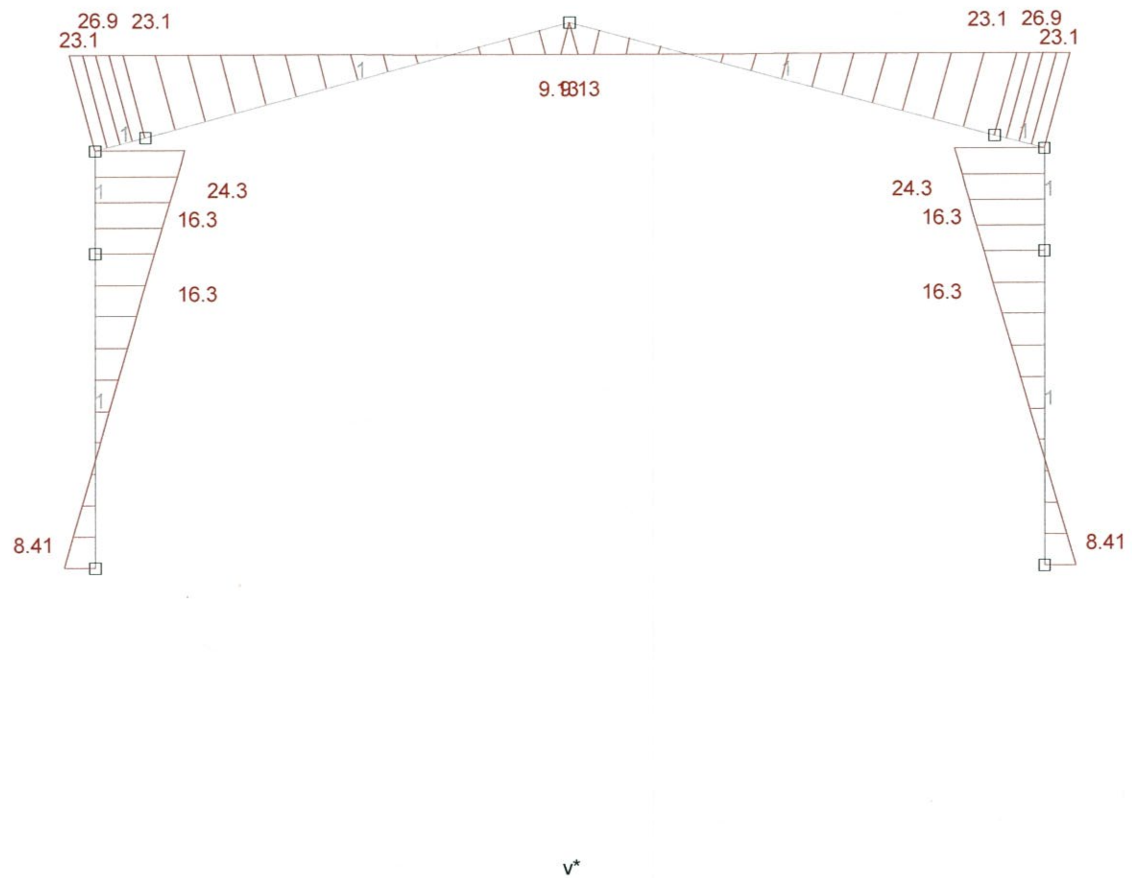
theta: 270 phi: 0

Shear Force, Fy

v*

Load Cases:
15 C 0.9G+Wu - Long

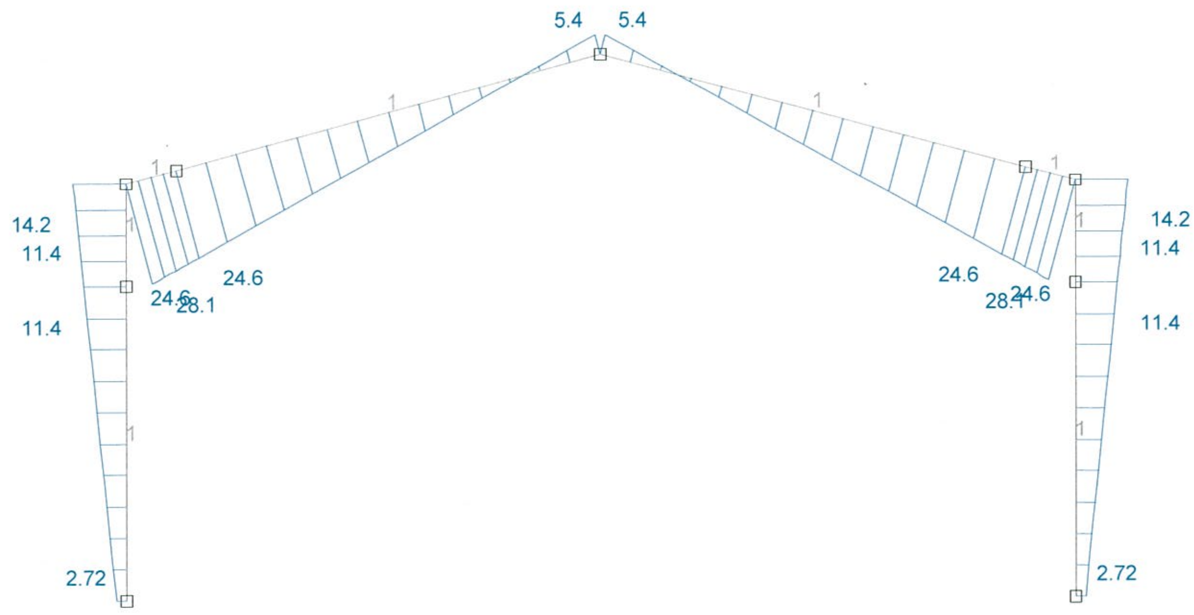
Sections:
1 600x75



Shear Force, Fy

Load Cases:
16 C 1.2G+Wu - Long

Sections:
1 600x75



theta: 270 phi: 0

v*

Shear Force, Fy

INCLUDING PARAPET LOADS

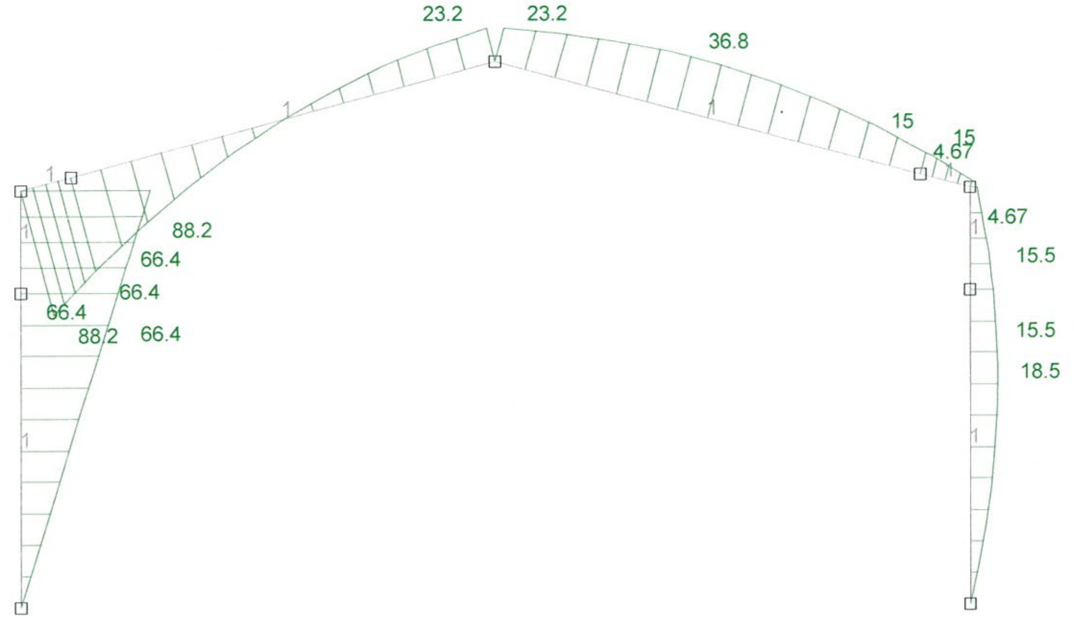
CHH Infortech NZ
Job: HF0014 12x4.8 parapet check
12m x 4.8m hyFRAME

18 Dec 2017
10:20 a.m.

Grid B - critical

Load Cases:
13 C 0.9G+Wu - Lat

Sections:
1 600x75



Y
Z X
theta: 270 phi: 0

m*

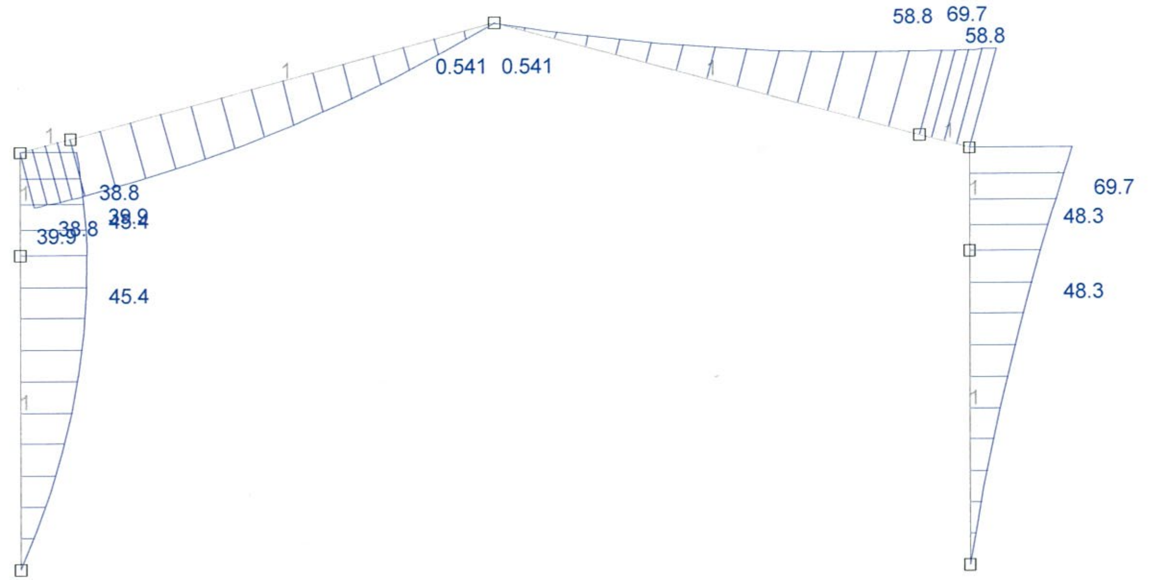
Bending Moment, Mz

Microstran [V8.11x]

\\dnzchhnas\vandiej$\UserSettings\Documents\Current projects\HF0014 Matakana - Structural Design\HF0014 12x4.8 parapet check

Load Cases:
14 C 1.2G+Wu - Lat

Sections:
1 600x75



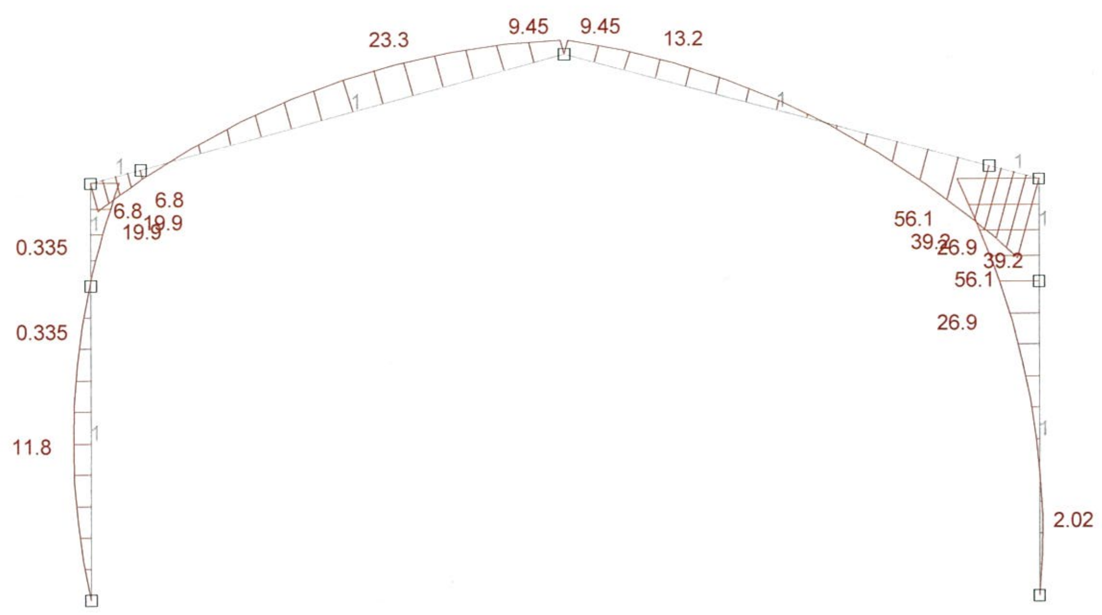
theta: 270 phi: 0

m*

Bending Moment, Mz

Load Cases:
15 C 0.9G+Wu - Long

Sections:
1 600x75



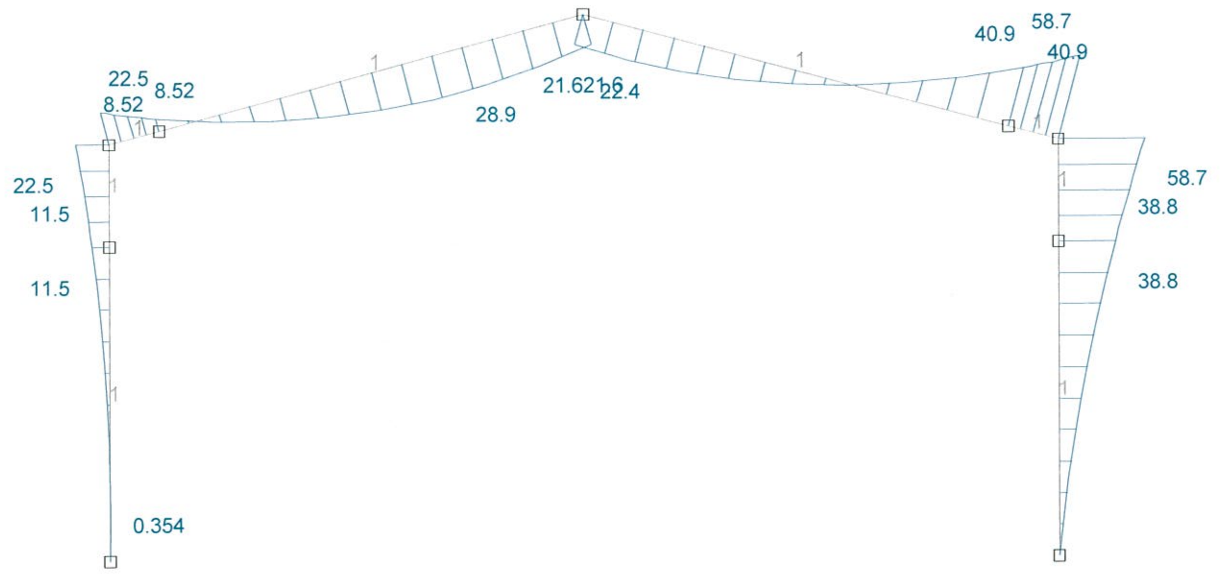
Y
Z X
theta: 270 phi: 0

m*

Bending Moment, Mz

Load Cases:
16 C 1.2G+Wu - Long

Sections:
1 600x75



theta: 270 phi: 0

m*

Bending Moment, Mz

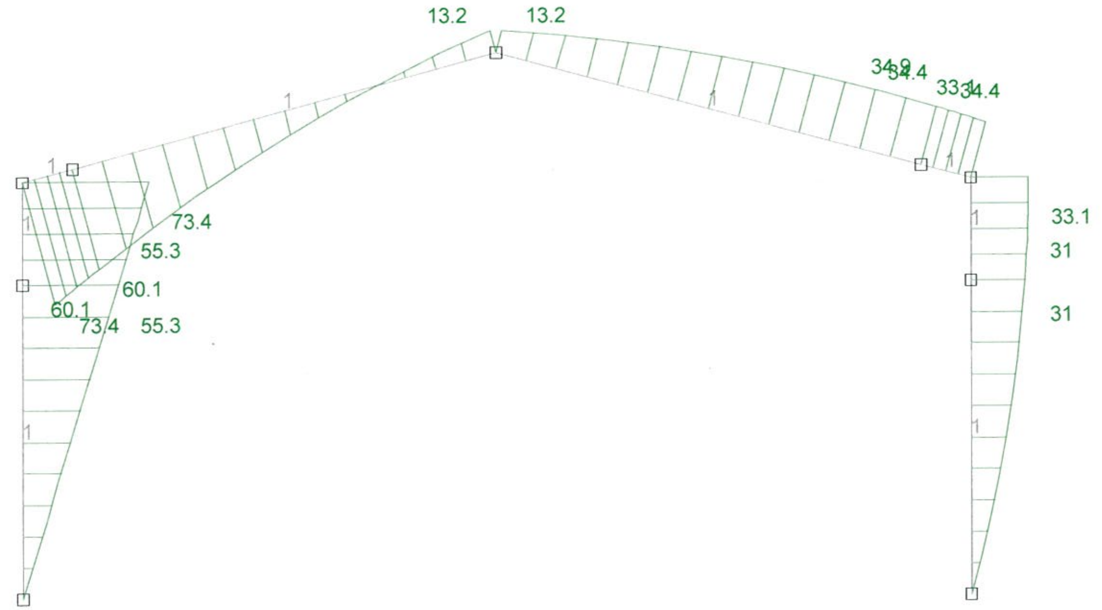
Grid A

CHH Infortech NZ
Job: HF0014 12x4.8 parapet check2
12m x 4.8m hyFRAME

18 Dec 2017
10:24 a.m.

Load Cases:
13 C 0.9G+Wu - Lat

Sections:
1 600x75



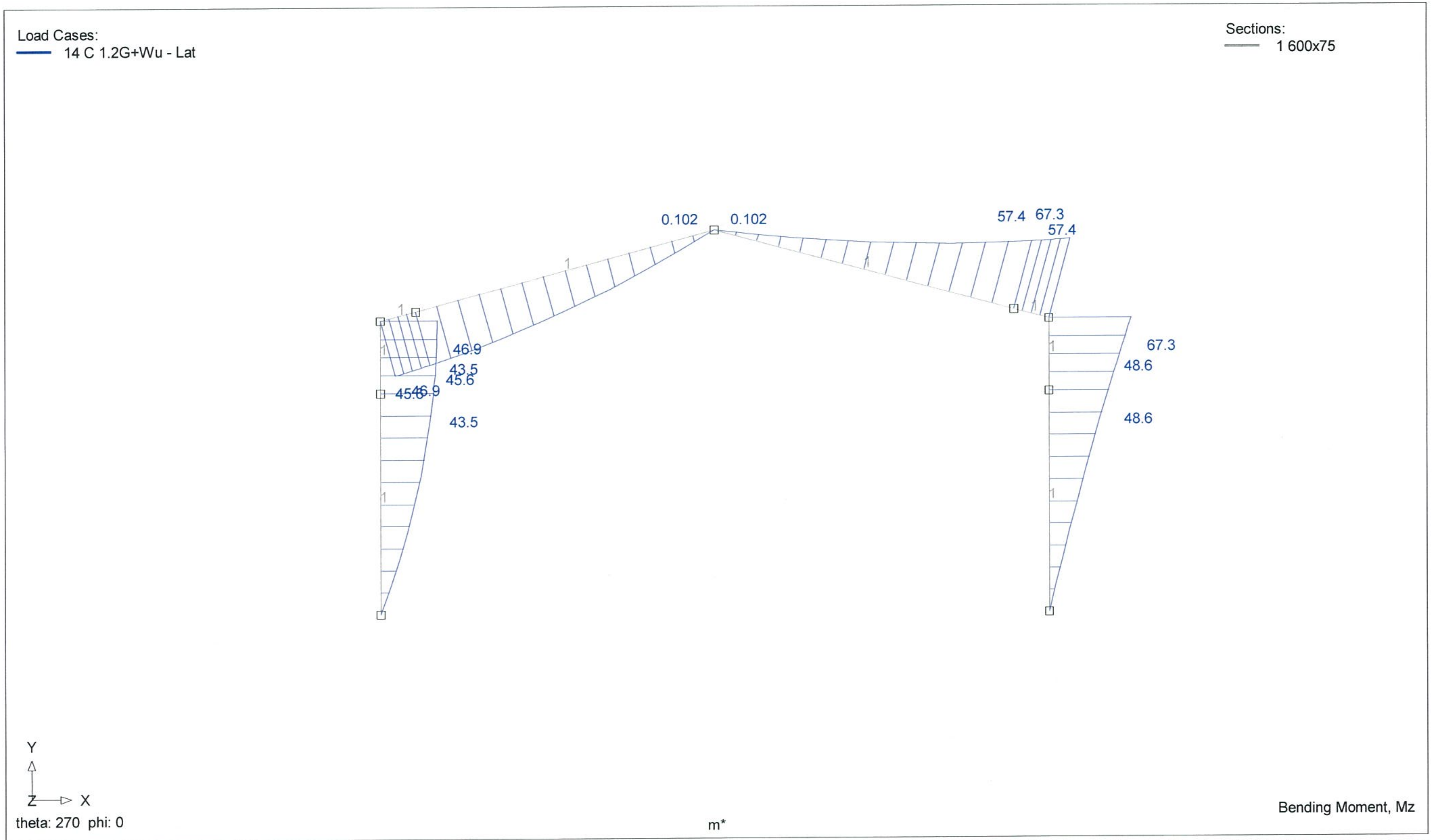
Y
Z X
theta: 270 phi: 0

m*

Bending Moment, Mz

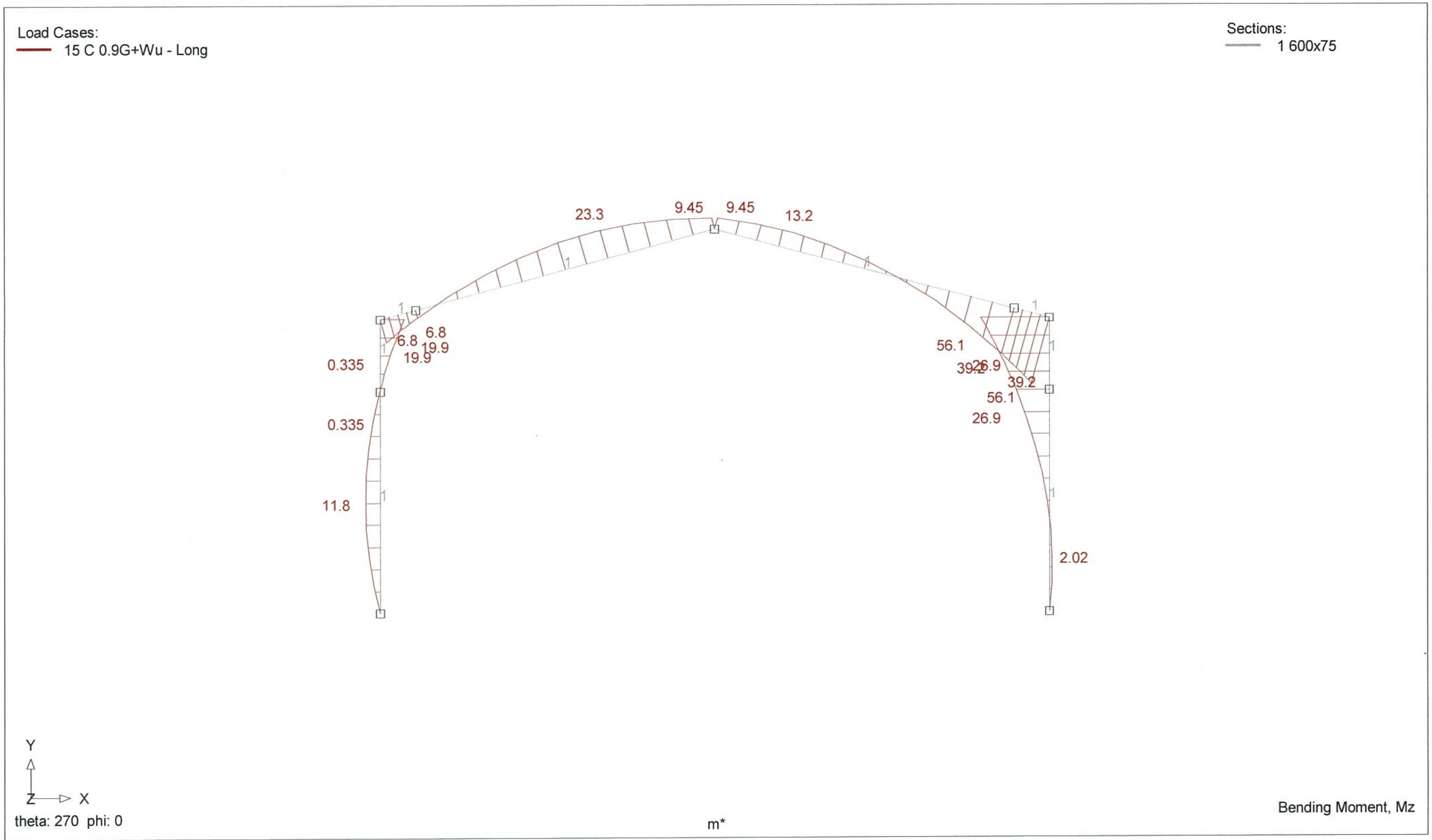
Microstran [V8.11x]

\\dnzchhnas\vandiej$\UserSettings\Documents\Current projects\HF0014 Matakana - Structural Design\HF0014 12x4.8 parapet check2


Load Cases:
14 C 1.2G+Wu - Lat
Sections:
1 600x75
0.102
0.102
57.4
67.3
57.4
67.3
48.6
48.6
46.9
43.5
45.6
45.6
46.9
43.5
Y
Z
X
theta: 270 phi: 0
m*
Bending Moment, Mz

CHH Infortech NZ
Job: HF0014 12x4.8 parapet check2
12m x 4.8m hyFRAME

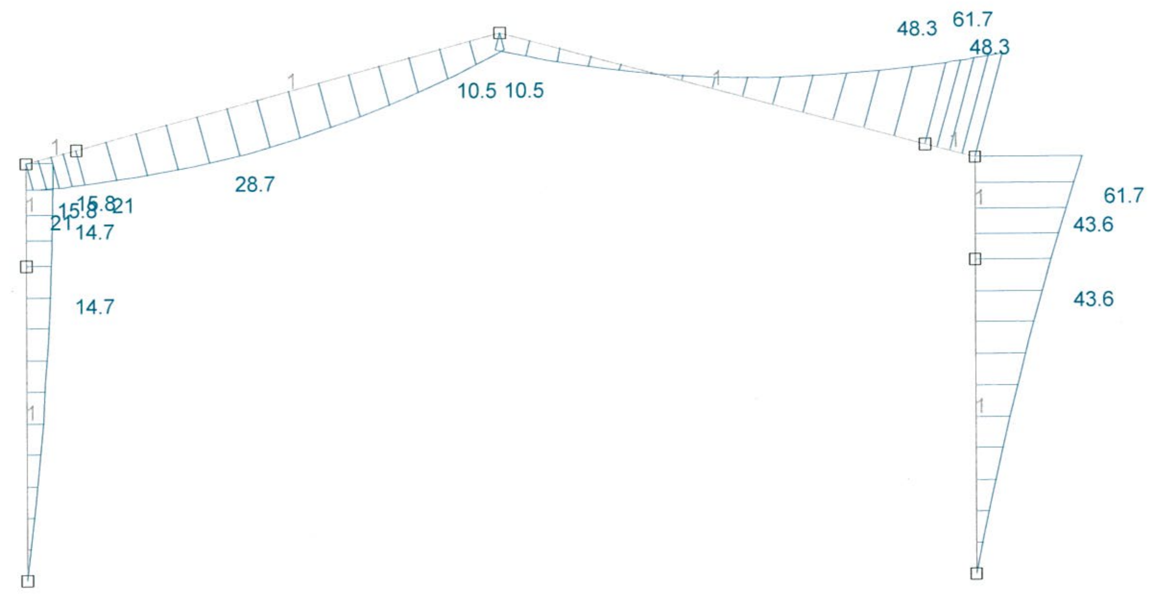
18 Dec 2017
10:24 a.m.



Microstran [V8.11x]
\\dnzchhnas\vandiej$\UserSettings\Documents\Current projects\HF0014 Matakana - Structural Design\HF0014 12x4.8 parapet check2

Load Cases:
16 C 1.2G+Wu - Long

Sections:
1 600x75



Y
Z X
theta: 270 phi: 0

m*

Bending Moment, Mz

Appendix 2 – Portal Frame & Eave Beam Design Capacities – Engineering Analysis Reports from computeIT software

ENGINEERING ANALYSIS REPORT

computeIT toolkIT has been prepared in accordance with soundly based and widely recognised engineering principles to assist engineers in providing engineering analysis (the determination of design capacities and the generation of design action effects for purlins and girts) of CHH Woodproducts Engineered Wood Products (EWP) range. The structural analysis methodology used in computeIT toolkIT references the following design standards:

1. AS/NZS 1170.0 - 2002 Structural design actions, Part 0: General principles.
2. AS/NZS 1170.1 - 2002 Structural design actions, Part 1: Permanent, imposed and other actions.
3. AS/NZS 1170.2 - 2011 Structural design actions, Part 2: Wind actions.
4. AS/NZS 1170.3 - 2003 Structural design actions, Part 3: Snow and Ice Actions.
5. AS 1720.1:2010 Timber Structures, Part 1: Design methods

CHH Woodproducts EWP range has been manufactured to AS/NZS 4357, with design properties characterised in accordance with Section 4 of AS/NZS 4063.2:2010.

This Engineering Analysis Report, and any associated warranty, is void where there has been substitution of alternate products not detailed within the specification.

Version date: 2nd June, 2017

For further information contact:

For further information or advice contact:
Carter Holt Harvey Woodproducts
173 Captain Spings Road,
Onehunga, Auckland, 1061
Free call 0800 808 131
Email: computeITNZ@chhwoodproducts.co.nz

Job environmental details:

Country:	New Zealand	Structure importance level:	2
Wind region:	A6		
Snow region:	Sub-alpine		

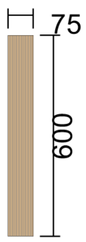
Design Details

<u>Member 1</u>

1) **Member code and description** MD1 - Rafter

1.1) **Member Type** Solid section

2) **Member Properties**



2.1) **Section Properties**

D = 600 mm
b = 75 mm
EI_{xx} = 14.445×10^{12} Nmm^2
EI_{yy} = 0.226×10^{12} Nmm^2

2.2) **Material Properties**

Product = hyFRAME
E_x = 10700 MPa
G = 535.0 MPa
$f_{b,x}^{1}$ = 40.0 MPa

A =	45000 mm^2	$f_{b,y}$ [1] =	38.0 MPa
J =	77.730×10^6 mm^4	f_t [2] =	19.2 MPa
Z =	$4{,}500.00 \times 10^3$ mm^3	f_c =	42.3 MPa
		$f_{s,x}$ =	4.6 MPa
		$f_{s,y}$ =	2.5 MPa

[1] For beams exceeding 95mm - Multiply the published characteristic value for bending by $(95/d)^{0.154}$ where d is the depth of the beam

[2] For tension members with the largest cross-section dimension exceeding 150mm - multiply the published characteristic value for tension by $(150/d)^{0.167}$, where d is the largest cross-sectional dimension of the tension member.

3) Member design effects - moment

$M_d = \O k_1 k_4 k_6 k_9 k_{12} f'_b Z$ AS 1720.1 Eqn 3.2(2)

Load case	Bending moment				L_{ay} mm	k_{12} mm	M_d kNm
	Duration of load factor [a] k_1	M^*_1 kNm	M^*_2 kNm	$\beta = M^*_1 / M^*_2$			
$1.35G_1$	0.57	8.2	0.0	0.00	2000	0.83	57.48
$1.2G_1 + 1.5Q$	0.94	18.8	0.0	0.00	2000	0.86	98.91
$1.2G_1 + S_U + \Psi_c Q$	0.94	24.7	0.0	0.00	2000	0.87	99.53
$G_1 + E + \Psi_c Q$ [b]	1.00	35.8	35.8	1.00	2377	0.59	72.12
$0.9G_1 + W_U$	1.00	-66.4	-66.4	1.00	1350	0.82	100.39
$1.2G_1 + W_U$ [b]	1.00	58.8	58.8	1.00	2377	0.59	72.49

[a] For timber/LVL.

[b] $L_{ay\,effective}$ based on discrete lateral restraints to tension edge

4) Axial and Shear Force

$N_{d,c} = \O k_1 k_4 k_6 k_{12} f'_c A_c$ AS 1720.1 Eqn 3.3(2)

Load case	Duration of load factor [1] k_1	N^*_c kNm	L mm	Major Axis			Minor Axis		
				g_{13}	L_{ax} mm	$N_{d,cx}$ kNm	g_{13}	L_{ay} mm	$N_{d,cy}$ kNm
$1.35G_1$	0.57	5	12500	1.00	12500	258.2	1.00	1350	345.8
$1.2G_1 + 1.5Q$	0.94	11.4	12500	1.00	12500	485.9	1.00	1350	650.9
$1.2G_1 + S_U + \Psi_c Q$	0.94	15	12500	1.00	12500	496.2	1.00	1350	664.7
$G_1 + E + \Psi_c Q$	1.00	7.5	12500	1.00	12500	502.8	1.00	1350	673.5
$0.9G_1 + W_U$	1.00								
$1.2G_1 + W_U$	1.00	26.2	12500	1.00	12500	541.0	1.00	1350	724.7

[1] For timber/LVL.

$V_d = \O k_1 k_4 k_6 f'_s A_s$ AS 1720.1 Eqn 3.2(14)

$N_{d,t} = \O k_1 k_4 k_6 f'_t A_t$ AS 1720.1 Eqn 3.4(2)

Load case	Duration of load factor [1] k_1	Shear Force		Tension	
		V^* kN	V_d kN	N^*_t kN	$N_{d,t}$ kN
$1.35G_1$	0.57	8.7	70.8		
$1.2G_1 + 1.5Q$	0.94	19.3	116.7		
$1.2G_1 + S_u + \Psi_c Q$	0.94	25.4	116.7		
$G_1 + E + \Psi_c Q$	1.00	10	124.2		
$0.9G_1 + W_u$	1.00	33	124.2	38.0	616.9
$1.2G_1 + W_u$	1.00	28.1	124.2		

[1] For timber/LVL.

5) Combined actions check

Combined bending and compression

$$\left(\frac{M^*_x}{M_{d,x}}\right)^2 + \frac{N^*_c}{N_{d,cy}} \leqslant 1.0$$

AS 1720.1 Eqn 3.5(1)

$$\frac{M^*_x}{M_{d,x}} + \frac{N^*_c}{N_{d,cx}} \leqslant 1.0$$

AS 1720.1 Eqn 3.5(2)

Combined bending and tension

$$\frac{k_{12}M^*}{M_d} + \frac{N^*_t}{N_{d,t}} \leqslant 1.0$$

AS 1720.1 Eqn 3.5(3)

$$\frac{M^*_x}{M_{d,x}} - \frac{Z}{A} \cdot \frac{N^*_t}{M_{d,x}} \leqslant 1.0$$

AS 1720.1 Eqn 3.5(4)

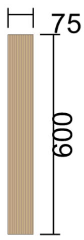
Load case	Duration of load factor k_1	Eqn 3.5(1)	Eqn 3.5(2)	Eqn 3.5(3)	Eqn 3.5(4)
$1.35G_1$	0.57	0.03	0.16		
$1.2G_1 + 1.5Q$	0.94	0.05	0.21		
$1.2G_1 + S_U + \Psi_c Q$	0.94	0.08	0.28		
$G_1 + E + \Psi_c Q$	1.00	0.26	0.51		
$0.9G_1 + W_U$	1.00			0.60	0.62
$1.2G_1 + W_U$	1.00	0.69	0.86		

Member 2

1) Member code and description MD2 - Column

1.1) Member Type Solid section

2) Member Properties



2.1) Section Properties

D = 600 mm
b = 75 mm
EI_{xx} = 14.445×10^{12} Nmm^2
EI_{yy} = 0.226×10^{12} Nmm^2
A = 45000 mm^2
J = 77.730×10^{6} mm^4
Z = $4{,}500.00 \times 10^{3}$ mm^3

2.2) Material Properties

Product = hyFRAME
E_x = 10700 MPa
G = 535.0 MPa
$f_{b,x}$[1] = 40.0 MPa
$f_{b,y}$[1] = 38.0 MPa
f_t[2] = 19.2 MPa
f_c = 42.3 MPa
$f_{s,x}$ = 4.6 MPa
$f_{s,y}$ = 2.5 MPa

[1] For beams exceeding 95mm - Multiply the published characteristic value for bending by $(95/d)^{0.154}$ where d is the depth of the beam

[2] For tension members with the largest cross-section dimension exceeding 150mm - multiply the published characteristic value for tension by $(150/d)^{0.167}$, where d is the largest cross-sectional dimension of the tension member.

3) Member design effects - moment

$$M_d = \text{Ø} k_1 k_4 k_6 k_9 k_{12} f'_b Z$$ AS 1720.1 Eqn 3.2(2)

Load case	Bending moment				L_{ay} mm	k_{12} mm	M_d kNm
	Duration of load factor [a] k_1	M^*_1 kNm	M^*_2 kNm	$\beta = \frac{M^*_1}{M^*_2}$			
$1.35G_1$	0.57	9.8	0.0	0.00	3850	0.57	39.34
$1.2G_1 + 1.5Q$	0.94	22.7	0.0	0.00	3850	0.62	70.61
$1.2G_1 + S_U + \Psi_c Q$	0.94	29.8	0.0	0.00	3850	0.62	71.46
$G_1 + E + \Psi_c Q$	1.00	27.0	0.0	0.00	3850	0.62	76.24
$0.9G_1 + W_U$	1.00	-66.4	-66.4	1.00	1350	0.82	100.39
$1.2G_1 + W_U$	1.00	48.3	0.0	0.00	3850	0.63	76.98

[a] For timber/LVL.

4) Axial and Shear Force

$$N_{d,c} = \O k_1 k_4 k_6 k_{12} f'_c A_c$$ AS 1720.1 Eqn 3.3(2)

Load case	Duration of load factor [1] k_1	N^*_c kNm	L mm	Major Axis			Minor Axis		
				g_{13}	L_{ax} mm	$N_{d,cx}$ kNm	g_{13}	L_{ay} mm	$N_{d,cy}$ kNm
$1.35G_1$	0.57	11.2	5030	1.00	5030	924.4	1.00	1350	345.8
$1.2G_1 + 1.5Q$	0.94	23.2	5030	1.00	5030	1577.2	1.00	1350	644.5
$1.2G_1 + S_U + \Psi_c Q$	0.94	30	5030	1.00	5030	1586.9	1.00	1350	659.7
$G_1 + E + \Psi_c Q$	1.00	13.6	5030	1.00	5030	1652.4	1.00	1350	648.0
$0.9G_1 + W_U$	1.00								
$1.2G_1 + W_U$	1.00	37.8	5030	1.00	5030	1694.6	1.00	1350	712.0

[1] For timber/LVL.

$$V_d = \O k_1 k_4 k_6 f'_s A_s$$ AS 1720.1 Eqn 3.2(14)

$$N_{d,t} = \O k_1 k_4 k_6 f'_t A_t$$ AS 1720.1 Eqn 3.4(2)

Load case	Duration of load factor [1] k_1	Shear Force		Tension	
		V^* kN	V_d kN	N^*_t kN	$N_{d,t}$ kN
$1.35G_1$	0.57	2.6	70.8		
$1.2G_1 + 1.5Q$	0.94	6	116.7		
$1.2G_1 + S_u + \Psi_c Q$	0.94	7.9	116.7		
$G_1 + E + \Psi_c Q$	1.00	7.1	124.2		
$0.9G_1 + W_u$	1.00	27.5	124.2	41.0	616.9
$1.2G_1 + W_u$	1.00	25.0	124.2		

[1] For timber/LVL.

5) Combined actions check

Combined bending and compression

$$\left(\frac{M_x^*}{M_{d,x}}\right)^2 + \frac{N_c^*}{N_{d,cy}} \leqslant 1.0$$

AS 1720.1 Eqn 3.5(1)

$$\frac{M_x^*}{M_{d,x}} + \frac{N_c^*}{N_{d,cx}} \leqslant 1.0$$

AS 1720.1 Eqn 3.5(2)

Combined bending and tension

$$\frac{k_{12}M^*}{M_d} + \frac{N_t^*}{N_{d,t}} \leqslant 1.0$$

AS 1720.1 Eqn 3.5(3)

$$\frac{M_x^*}{M_{d,x}} - \frac{Z}{A} \cdot \frac{N_t^*}{M_{d,x}} \leqslant 1.0$$

AS 1720.1 Eqn 3.5(4)

Load case	Duration of load factor k_1	Eqn 3.5(1)	Eqn 3.5(2)	Eqn 3.5(3)	Eqn 3.5(4)
$1.35G_1$	0.57	0.09	0.26		
$1.2G_1 + 1.5Q$	0.94	0.14	0.34		
$1.2G_1 + S_u + \Psi_c Q$	0.94	0.22	0.44		
$G_1 + E + \Psi_c Q$	1.00	0.15	0.36		
$0.9G_1 + W_u$	1.00			0.61	0.62
$1.2G_1 + W_u$	1.00	0.45	0.65		

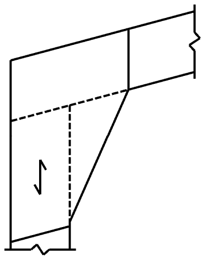
Member 3

1) Member code and description MC1 - Knee

1.1) Member Type Knee gusset - Rafter over column

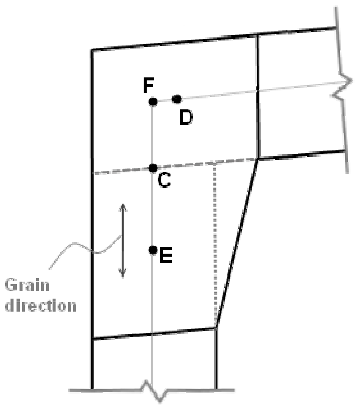
2) Gusset design

2.1) Gusset and section geometry



Column	
- Depth	600 mm
- Breadth	75 mm
Rafter	
- Depth	600 mm
- Breadth	75 mm
Roof pitch	15°
Gusset Width	1200 mm
Gusset thickness	25.0 mm
D_{gusset}	1099.6 mm
Product	F8 Plywood
Grade	Plywood

2.2) User entered gusset design actions



Load case	Duration of load [1] k_1	Node	M* kNm	N_c kN	N^*_t kN	V* kN
1.35G	0.57	F	13.0	9.8		2.6
1.2G + 1.5Q	0.94	F	30.1	22.0		6.0
1.2G + S_u + Ψ_cQ	0.94	F	39.6	28.8		7.9
G + E + Ψ_cQ	1.00	F	35.8	12.6		7.1
0.9G + W_u	1.00	F	-88.2		41.0	24.3
1.2G + W_u	1.00	F	69.7	33.3		25.0

1, for timber/LVL

2.3) Design criteria

$$\frac{N_c^*}{N_{d,c}} + \left(\frac{M_i^*}{M_{d,i}}\right)^2 + \frac{V_i^*}{V_{d,i}} \leqslant 1.0$$ AS 1720.1:2010 Eq. 5(15)

$$\frac{N_t^*}{N_{d,t}} + \left(\frac{M_i^*}{M_{d,i}}\right)^2 + \frac{V_i^*}{V_{d,i}} \leqslant 1.0$$ AS 1720.1:2010 Eq. 5(16)

$M_{d,i} = \O k_1 k_{12} k_{19} g_{19} f'_b Z_i$ AS 1720.1:2010 Eqn 5(8)
$M_{d,i} = 109.5$ kNm

$N_{d,c} = \O k_1 k_{12} k_{19} g_{19} f'_c A_c$ AS 1720.1:2010 Eqn 5(14)
$N_{d,c} = 324.0$ kN

$N_{d,t} = \O k_1 k_{19} g_{19} f'_t A_t$ AS 1720.1:2010 Eqn 5(12)
$N_{d,t} = 243.0$ kN

$V_{d,i} = \O k_1 k_{12} k_{19} g_{19} f'_s A_s$ AS 1720.1:2010 Eqn 5(9)
$V_{d,i} = 148.4$ kN

Note:($\O$ = 0.9, k_1 = 1.0, k_{12} = 1.0)

Load case	Duration of load [1] k_1	Design Capacities $M_{d,i}$ kNm	$N_{d,c}$ kN	$N_{d,t}$ kN	$V_{d,i}$ kN	Capacity ratios EQN 5(15)	EQN 5(16)
1.35G	0.57	62.4	184.7		84.6	0.13	
1.2G + 1.5Q	0.94	102.9	304.6		139.5	0.20	
1.2G + S_u + Ψ_c Q	0.94	102.9	304.6		139.5	0.30	
G + E + Ψ_c Q	1.00	109.5	324.0		148.4	0.19	
0.9G + W_u	1.00	109.5		243.0	148.4		0.98
1.2G + W_u	1.00	109.5	324.0		148.4	0.68	

[1] For timber/LVL.

3) Nail ring design

3.1) User entered critical design actions

Load case	Duration of load [1] k_1	Node	M* kNm	N^*_c kN	N^*_t kN	V* kN
1.35G	0.57	F	13.0	9.8		2.6
1.2G + 1.5Q	0.77	F	30.1	22.0		6.0
1.2G + S_u + Ψ_c Q	0.77	F	39.6	28.8		7.9
G + E + Ψ_c Q	1.14	F	35.8	12.6		7.1
0.9G + W_u	1.14	F	-88.2		41.0	24.3
1.2G + W_u	1.14	F	69.7	33.3		25.0

1, for connections

3.2) Design criteria

3.2.1) Moment capacity

$$M_{d,j} = \O k_1 k_{13} k_{14} k_{16} k_{17} r_{max} Q_k \sum_{i=1}^{n} \left(\frac{r_i}{r_{max}}\right)^{\frac{3}{2}}$$ AS 1720.1 Eqn 4.2(4)

3.2.2) Axial / Shear capacity

$N_{d,i} = Øk_1 k_{13} k_{14} k_{16} k_{17} n Q_k$ AS 1720.1 Eqn 4.2(2)

3.3) Fastener characteristics

Fastener Type	Nails
Diameter	2.87 mm
Minimum length	53.7 mm
Q_k	694.0 N
Along the grain spacing	50 mm
Across the grain spacing	25 mm
Joint Group	JD4
Fastener factor, k_{13}	1
No. shear planes, k_{14}	1
Gusset material factor, k_{16}	1.1

3.4) Proposed Nail ring

Nail ring no.	Node coordinates Node number				No. nails	Nail Ring Capacity $M_{d,j}$ [1,2] kNm	Nail Ring $N_{d,j}$ [1,2] kN
	A mm	B mm	C mm	D mm			
1	0, 0	0, 569	1111, 867	1111, 298	90	56.7	0.716
2	24, 19	24, 563	1087, 848	1087, 304	176	109.9	0.733
3	48, 39	48, 556	1063, 828	1063, 311	258	156.0	0.733

1, - Ø = 0.8
2, - k_1 = 1.0

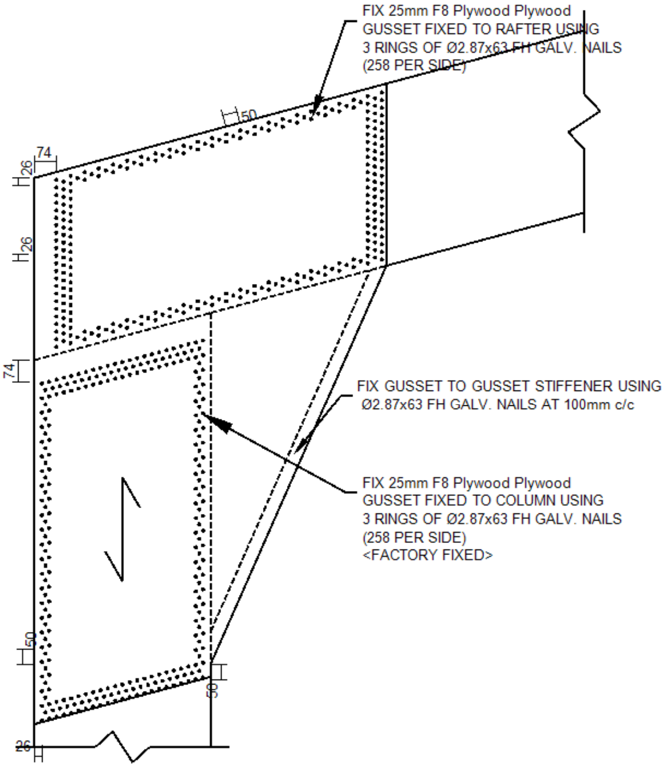
3.5) Combined actions check

3.5.1) For Node F

Load case	Duration of load [3] k_1	Nail ring capacities $M_{d,i}$ [1,2] kNm	Nail capacity $N_{d,a/s}$ [1] kN	Capacity ratios $\frac{M^*}{M_{d,i}}$	Capacity ratios $\frac{\sqrt{N^{*2} + V^{*2}}}{N_{d,a/s}}$
1.35G	0.57	88.9	184.0	0.15	0.06
1.2G + 1.5Q	0.77	120.1	218.2	0.25	0.10
1.2G + S_U + Ψ_cQ	0.77	120.1	195.2	0.33	0.15
G + E + Ψ_cQ	1.14	177.8	344.3	0.20	0.04
0.9G + W_U	1.14	177.8	217.3	0.50	0.22
1.2G + W_U	1.14	177.8	262.1	0.39	0.16

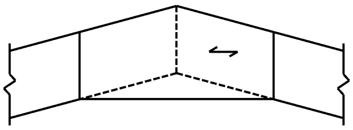
1, Ø = 0.8
2, Moment capacity based on 3 nail rings
3, For connections

4) Proposed Connection



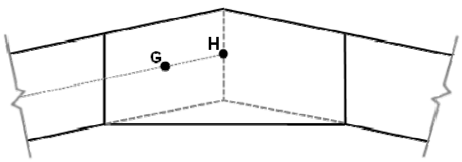
<u>Member 4</u>

1)	**Member code and description**	MC2 - Ridge
1.1)	**Member Type**	Ridge gusset - Mitre joint
2)	**Gusset design**	
2.1)	**Gusset and section geometry**	



Rafter	
- Depth	600 mm
- Breadth	75 mm
Roof pitch	15°
Gusset Width	900 mm
Gusset thickness	25.0 mm
D_{gusset}	776.1 mm
Product	F8 Plywood
Grade	Plywood

2.2) User entered gusset design actions



Load case	Duration of load [1] k_1	Node	M* kNm	N_c kN	N^*_t kN	V* kN
1.35G	0.57	G	-9.8	2.5		0.7
1.2G + 1.5Q	0.94	G	-22.5	5.8		1.6
1.2G + S_U + Ψ_cQ	0.94	G	-29.6	7.6		2.1
G + E + Ψ_cQ	1.00	G	-50.0	5.8		5.0
0.9G + W_U	1.00	G	22.0		34.0	9.7
1.2G + W_U	1.00	G	-26.7	20.1		11.9

1, for timber/LVL

2.3) Design criteria

$$\frac{N_c^*}{N_{d,c}} + \left(\frac{M_i^*}{M_{d,i}}\right)^2 + \frac{V_i^*}{V_{d,i}} \leqslant 1.0$$

AS 1720.1:2010 Eq. 5(15)

$$\frac{N_t^*}{N_{d,t}} + \left(\frac{M_i^*}{M_{d,i}}\right)^2 + \frac{V_i^*}{V_{d,i}} \leqslant 1.0$$

AS 1720.1:2010 Eq. 5(16)

$$M_{d,i} = Øk_1 k_{12} k_{19} g_{19} f'_b Z_i$$
$$M_{d,i} = 57.8 \text{ kNm}$$

AS 1720.1:2010 Eqn 5(8)

$$N_{d,c} = Øk_1 k_{12} k_{19} g_{19} f'_c A_c$$
$$N_{d,c} = 324.0 \text{ kN}$$

AS 1720.1:2010 Eqn 5(14)

$$N_{d,t} = Øk_1 k_{19} g_{19} f'_t A_t$$
$$N_{d,t} = 243.0 \text{ kN}$$

AS 1720.1:2010 Eqn 5(12)

$$V_{d,i} = Øk_1 k_{12} k_{19} g_{19} f'_s A_s$$
$$V_{d,i} = 104.8 \text{ kN}$$

AS 1720.1:2010 Eqn 5(9)

Note:($Ø = 0.9$, $k_1 = 1.0$, $k_{12} = 1.0$)

		Design Capacities				Capacity ratios	
Load case	Duration of load [1] k_1	$M_{d,i}$ kNm	$N_{d,c}$ kN	$N_{d,t}$ kN	$V_{d,i}$ kN	EQN 5(15)	EQN 5(16)
1.35G	0.57	33.0	184.7		59.7	0.11	
1.2G + 1.5Q	0.94	54.3	304.6		98.5	0.21	
1.2G + S_u + Ψ_cQ	0.94	54.3	304.6		98.5	0.34	
G + E + Ψ_cQ	1.00	57.8	324.0		104.8	0.81	
0.9G + W_u	1.00	57.8		243.0	104.8		0.38
1.2G + W_u	1.00	57.8	324.0		104.8	0.39	

[1] For timber/LVL.

3) Nail ring design

3.1) User entered critical design actions

Load case	Duration of load [1] k_1	Node	M* kNm	N^*_c kN	N^*_t kN	V* kN
1.35G	0.57	G	-9.8	2.5		0.7
1.2G + 1.5Q	0.77	G	-22.5	5.8		1.6
1.2G + S_u + Ψ_cQ	0.77	G	-29.6	7.6		2.1
G + E + Ψ_cQ	1.14	G	-50.0	5.8		5.0
0.9G + W_u	1.14	G	22.0		34.0	9.7
1.2G + W_u	1.14	G	-26.7	20.1		11.9

1, for connections

3.2) Design criteria

3.2.1) Moment capacity

$$M_{d,j} = Øk_1 k_{13} k_{14} k_{16} k_{17} r_{max} Q_k \sum_{i=1}^{n} \left(\frac{r_i}{r_{max}}\right)^{\frac{3}{2}}$$

AS 1720.1 Eqn 4.2(4)

3.2.2) Axial / Shear capacity

$$N_{d,i} = Øk_1 k_{13} k_{14} k_{16} k_{17} n Q_k$$

AS 1720.1 Eqn 4.2(2)

3.3) Fastener characteristics

Fastener Type	Nails
Diameter	2.87 mm
Minimum length	53.7 mm
Q_k	694.0 N
Along the grain spacing	50 mm
Across the grain spacing	25 mm
Joint Group	JD4
Fastener factor, k_{13}	1
No. shear planes, k_{14}	1
Gusset material factor, k_{16}	1.1

3.4) Proposed Nail ring

Nail ring no.	Node coordinates Node number				No. nails	Nail Ring Capacity $M_{d,j}$ [1,2] kNm	Nail Ring $N_{d,j}$ [1,2] kN
	A mm	B mm	C mm	D mm			
1	0, 0	0, 569	821, 789	821, 220	78	40.2	0.716
2	24, 19	24, 563	797, 770	797, 226	152	76.7	0.729
3	48, 39	48, 556	773, 751	773, 233	222	108.2	0.733

1, - Ø = 0.8

2, - k_1 = 1.0

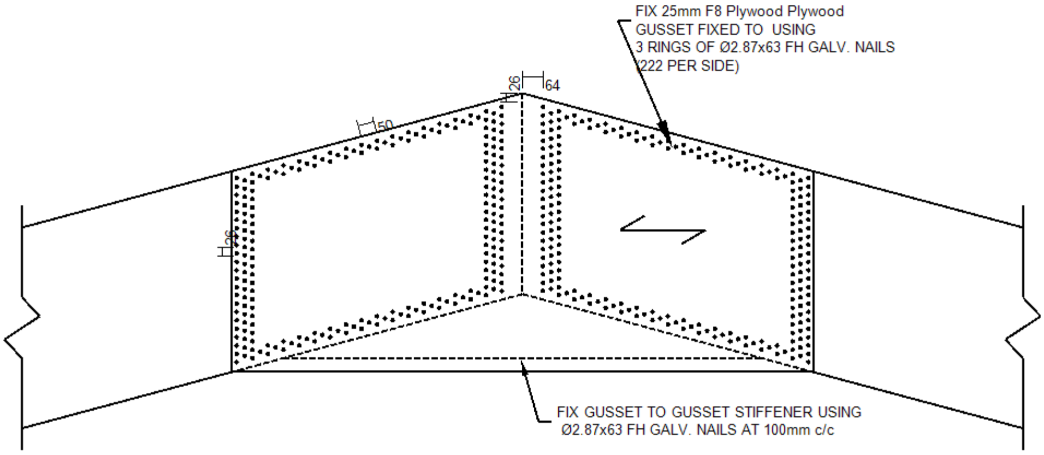
3.5) Combined actions check

3.5.1) For Node G

Load case	Duration of load [3] k_1	Nail ring capacities $M_{d,i}$ [1,2] kNm	Nail capacity $N_{d,a/s}$ [1] kN	Capacity ratios $\frac{M^*}{M_{d,i}}$	Capacity ratios $\frac{\sqrt{N^{*2} + V^{*2}}}{N_{d,a/s}}$
1.35G	0.57	61.7	156.0	0.16	0.02
1.2G + 1.5Q	0.77	83.3	182.9	0.27	0.03
1.2G + S_u + Ψ_c Q	0.77	83.3	161.5	0.36	0.05
G + E + Ψ_c Q	1.14	123.4	220.6	0.41	0.03
0.9G + W_u	1.14	123.4	304.8	0.18	0.12
1.2G + W_u	1.14	123.4	290.7	0.22	0.08

1, Ø = 0.8
2, Moment capacity based on 3 nail rings
3, For connections

4) Proposed Connection


FIX 25mm F8 Plywood Plywood
GUSSET FIXED TO USING
3 RINGS OF Ø2.87x63 FH GALV. NAILS
(222 PER SIDE)
26
64
50
26
FIX GUSSET TO GUSSET STIFFENER USING
Ø2.87x63 FH GALV. NAILS AT 100mm c/c

Engineering Analysis Report

computeIT for beams has been prepared in accordance with soundly based and widely recognised engineering principles to assist engineers in the design and specification of CHH Woodproducts Engineered Wood Products (EWP) range. The structural analysis methodology used in computeIT for beams references the following design standards:

1. AS/NZS 1170.0 - 2002 Structural design actions, Part 0: General principles.
2. AS/NZS 1170.1 - 2002 Structural design actions, Part 1: Permanent, imposed and other actions.
3. AS/NZS 1170.2 - 2006 Structural design actions, Part 2: Wind actions.
4. AS/NZS 1170.3 - 2003 Structural design actions, Part 3: Snow and Ice Actions.
5. AS 1720.1:2010 Timber Structures, Part 1: Design methods.

Design properties used for the CHH EWP range have been manufactured in accordance with AS/NZS 4357 and characterised in accordance with Section 4 of AS/NZS 4063.2:2010.

This Engineering Analysis Report, and any associated warranty, is void where there has been substitution of alternate products not detailed within the specification.

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Engineer details:

Designed by:	**Jeremy Van Diepen**				
Company:	**CHH Woodproducts NZ**				
Phone:	**09 633 1579**	Mobile:		Email:	**jeremy.vandiepen@chhwoodproducts.co.nz**

Project details:

Project:	**Eave beam design**	Revision:	
Client:	**CHH**		
Site address:	**362 Matakana Valeey Road, Matakana**		
Country:	**New Zealand**		

Job environment details:

Building importance level:	**2**	Wind region:	**A (6-7)**	
Snow region:	**Sub-Alpine**	Time snow is on roof	**5 days**	

MEMBER DESIGN DETAILS

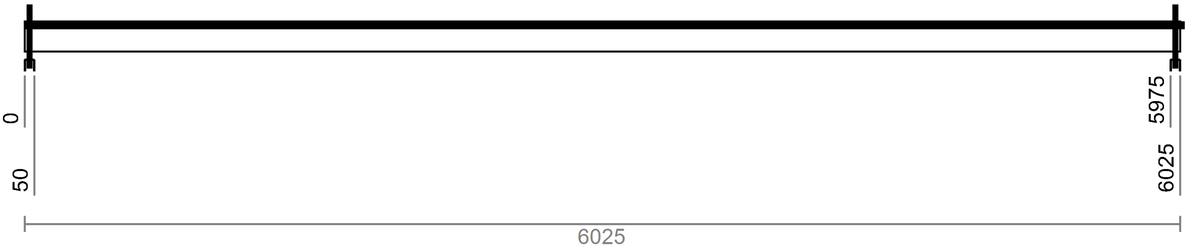
Member 1

1) **Member description** Eave beam

2) **Member specification** Use 240 x 90 hy90
Structural Laminated Veneer Lumber to AS/NZS 4357

3) **Restraints**

Type of restraint
Torsional
Lateral, continuous
Lateral, discrete

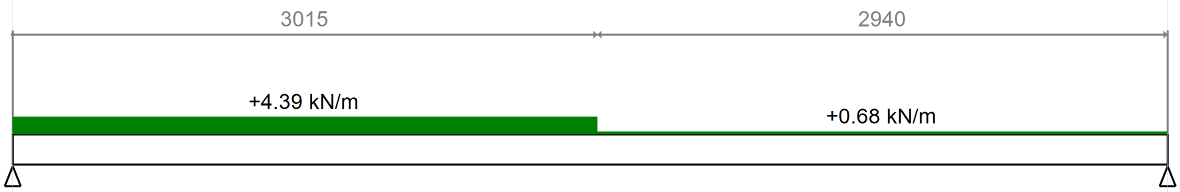


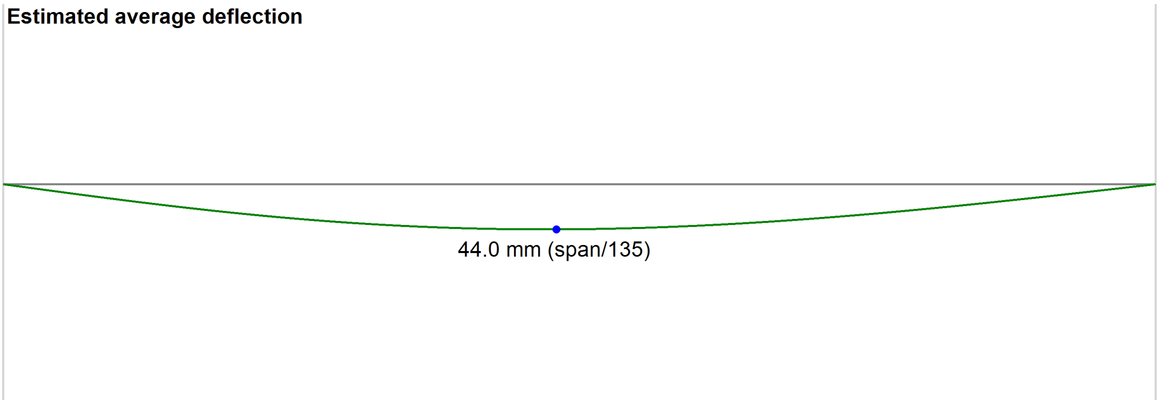
Restraint type	**Distance from LH**	
	Begin	**End**
Lateral, continuous - top	0	6050
Torsional	25	-
Torsional	6000	-

4) **Design load cases**

4.1) **Serviceability load cases**

4.1.1) **Load case : W $_{S1}$**

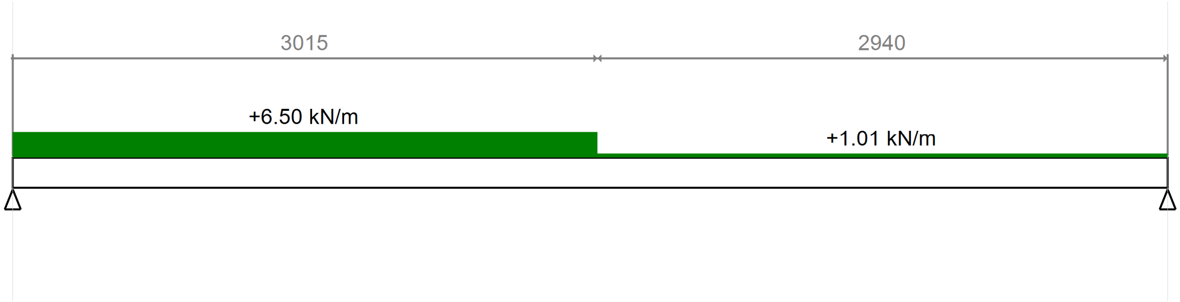




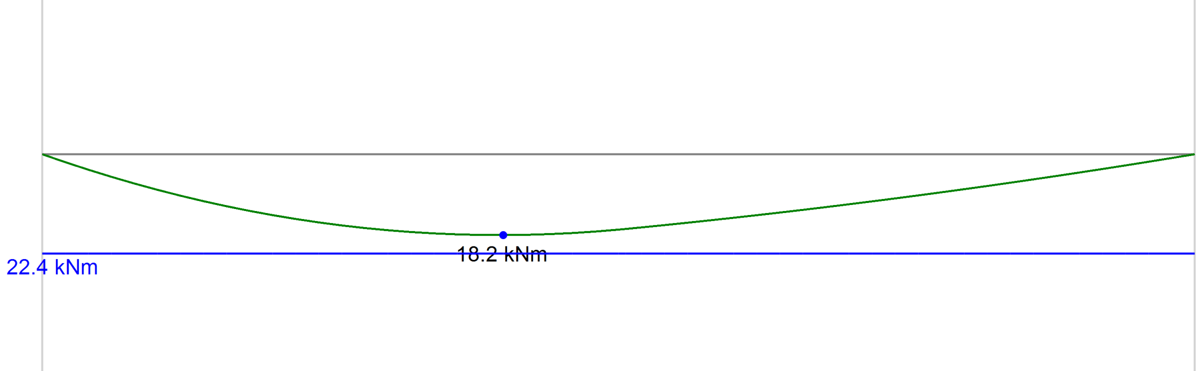
4.2) **Strength load cases**

Application Category 2

4.2.1) **Load case : W $_{U1}$**



Bending moment M*

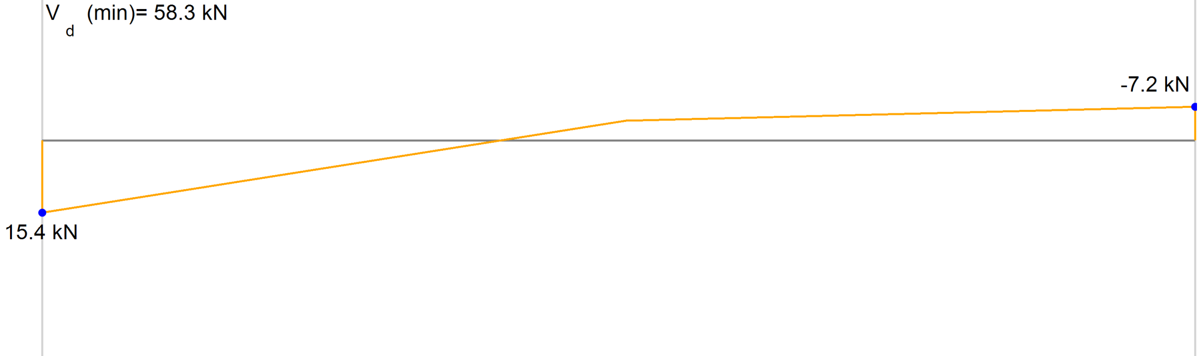


	$L^{\#}$ (m)	k_{12}	M_d (kNm)	M^* (kNm)
1	0.0 - 6.0	1.00	22.4	18.2

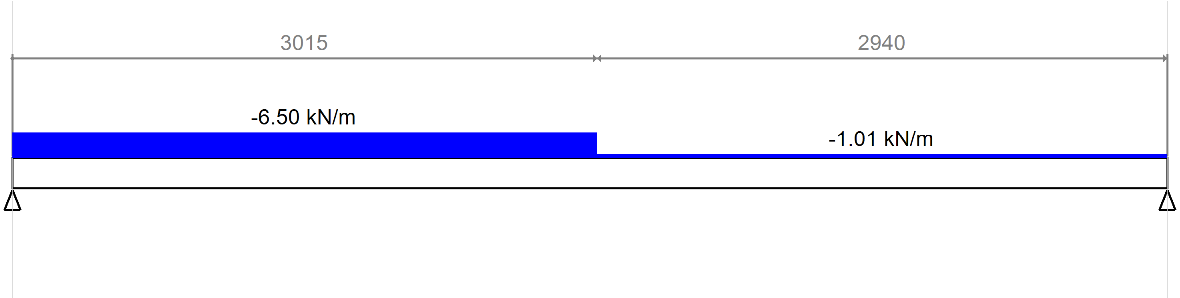
[#] Length of member considered for restraint

Shear V*

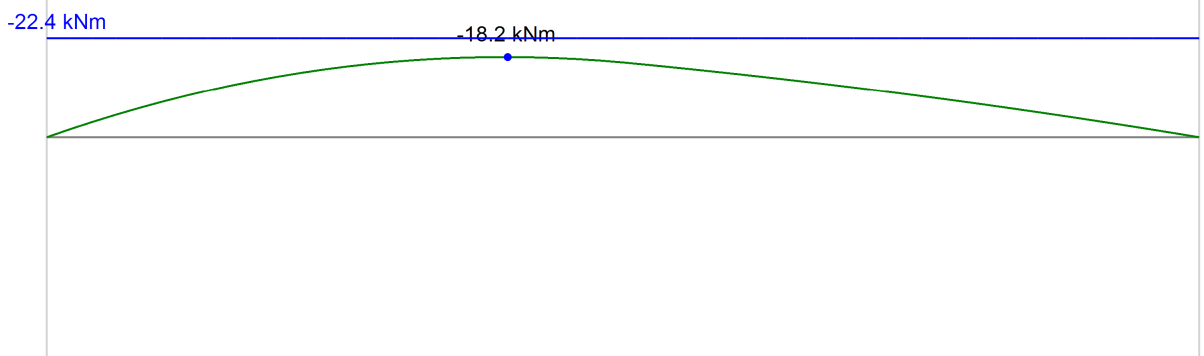
V_d (min)= 58.3 kN



4.2.2) Load case : W_{U2}



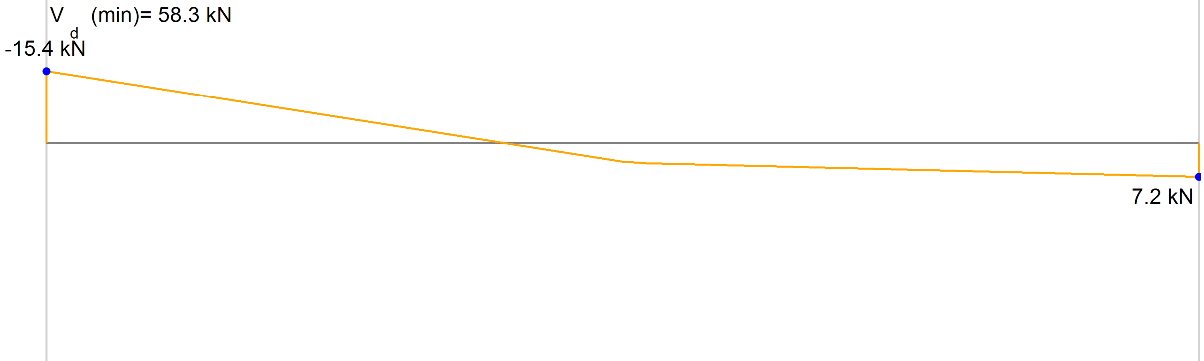
Bending moment M*



	L# (m)	k_{12}	M_d (kNm)	M^* (kNm)
1	0.0 - 6.0	1.00	22.4	-18.2

Length of member considered for restraint

Shear V*



5) **Reaction information**

Load case	Reactions (kN)	
	A	B
W_{U1}	-15.4	-7.2
W_{U2}	15.4	7.2

6) **Connection design**

Support A

Connection type Propietary joist hanger, including bearing support
Use 6/14G Screws

Load case	Reaction kN	N_{dj} kN
W_{U1}	-15.4	20.6
W_{U2}	15.4	20.6

Notes:
1. End and edge distances to comply with AS1720.1
2. Connection design does not consider the suitability of the supporting member/cleat

7) **Entered design actions**

Action	Type	Load (+ Down) and load position			
		Begin		End	
Action: Wind actions - W					
A	UDL	6.5 kN/m	at 0 mm	6.5 kN/m	at 3050 mm
B	UDL	1.0 kN/m	at 3050 mm	1.0 kN/m	at 6025 mm
C	UDL	-6.5 kN/m	at 0 mm	-6.5 kN/m	at 3050 mm

D	UDL	-1.0 kN/m	at 3050 mm	-1.0 kN/m	at 6025 mm

Notes for interpretation of serviceability data

1. “average deflection” is an engineering concept based upon a notional estimated load, notional member rigidity and, in some cases, an approximate model of material response to environmental conditions. These parameters are, ‘standardised’ in AS 1170, AS 1684.1 and AS 1720. Deflections calculated using this methodology cannot therefore be usefully compared with deflections calculated using other methods, eg GLTAA design methodology.

2. Deflection is the flexural response to load – ‘out-of-level’ measurements of installations are not necessarily deflections and can incorporate ‘initial out-of-straightness’, whether intended or not. Furthermore, loads can be higher/lower than the notional estimate and in any comparison with measured levels, material variability needs to also be considered. AS 1720 gives the following basis for estimation of upper bound deflections for various materials.

F-grades – visually graded to AS 2082 / AS 2858	Average + 100%
MGP grades - mechanically graded to AS 1748	Average + 43%
GL grades for glulam to AS 1328	Average + 33%
LVL to AS 4357 (includes hySPAN and hyJOIST (LVL flanges))	Average +18%

As can be seen, comparison of the ‘average deflection’ for different materials, even if calculated on the same basis, does not give the whole picture!

Notes for interpretation of reaction data

1. Negative (-) reactions relate to the 'gravity' or 'downwards' force on the support
2. Positive reactions relate to the 'upwards' forces on the support

Appendix 3 - Mullion deflection, bending and shear equations

The following design action equations have been provided from commonly available equations for a series of evenly spaced point loads as idealised for a mullion in service.

Due to the nature of loading of mullions 'n' in the equations is the number of girts supported by the mullion. It is assumed that the loads applied by girts at each location are equal.

Where n is odd:

$$\partial = \frac{P.L^3}{192.EI}\left[n - \frac{1}{n}\right].\left[3 - \frac{1}{2}\left(1 - \frac{1}{n^2}\right)\right]$$

$$M_{\max} = \frac{\left(n^2 - 1\right).P.L}{8.n}$$

$$R^* = \frac{n.P}{2}$$

Where n is even:

$$\partial = \frac{P.L^3}{192.EI}.n.\left[3 - \frac{1}{2}\left(1 + \frac{4}{n^2}\right)\right]$$

$$M_{\max} = n.\frac{P.L}{8}$$

$$R^* = \frac{n.P}{2}$$

Note: The reaction equation differs slightly from the conventional reaction equation for a series of point loads supported by a simply supported beam. This is to take into account the fact that a girt is located at the base of the mullion.

Appendix 4 – Building Mass/Earthquake loading

hyFRAME System mass										Lateral Load			Longitudinal Load		
									Common Frame		End wall Frame		Reidbrace		
Frames	No/frame	Depth	x	Breadth	Length	Volume	Density	Mass		kg		kg		kg	
		mm		mm	m	m3	kg/m3	kg							
Columns	2	600	x	75	4.8	0.43	600	259							
Rafters	2	600	x	75	6.5	0.59	600	351							
Gussets	6	1200	x	25	2	0.36	550	198							
						1.38		808	1	808	1	808	6	4849	
Bay Framing															
Purlins	12	240	x	45	6	0.78	600	467							
Purlin Blocking	6	240	x	45	1.3	0.08	600	51							
Lateral restraint	2	90	x	45	6.5	0.05	600	32							
Girts	10	240	x	45	6	0.65	600	389							
Girt Blocking	8	90	x	45	1.3	0.04	600	25							
						1.60		963	1	963	0.5	481	5	4814	
End wall framing															
Mullions	2	300	x	90	6.6	0.36	600	214							
Trimmers	2	240	x	45	5.3	0.11	600	69							
Raking Girts	2	240	x	45	6.5	0.14	600	84							
Girts	6	240	x	45	6	0.39	600	233							
Girts	10	240	x	45	3	0.32	600	194							
Girt Blocking	16	90	x	45	1.3	0.08	600	51							
						1.41		845			1	845	2	1690	
						Area	Mass	Mass							
Roofing Mass (per Bay)						m2	kg/m2	kg							
Roof Area	1	12.4	x	6		74.5	15	1118	1	1118	0.5	559	5	5590	
Cladding Mass (per Bay)															
Wall Area	2	5.3	x	6		63.6	15	954	1	954	0.5	477	5	4770	
End Wall Cladding Mass (per Bay)															
Wall Area	1	6.1	x	12		73.2	15	1099			1	1099	2	2197	
										3843	kg	4269	kg	23911	kg
									total seismic weight, wt =	37.7	kN	41.9	kN	234.6	kN
									Z=	0.13					
									Ch(T) =	2.36					
									μ =	1.25		1.25		1.0	
									$C_d(T_1)$ ULS	0.25		0.25		0.31	
									$C_d(T_1)$ SLS / Cd(T1) ULS	0.22		0.22		0.18	(for factoring ULS loads)
									Base Horizontal Shear, V* Eq	9.4	kN	10.4	kN	72.0	kN
									P* Eq, roof level	9.4	kN	10.4	kN	36.0	kN (per wall)

Appendix 5 – Base Bracket Design

Proposed Base Connection Design

Reactions

Load Case	k_1	PF1 R_x kN	R_y kN	R_z kN	$(R_x^2+R_y^2)^{0.5}$ kN	Angle
1.35g	0.57	2.59	11.21	0.00	11.51	77.0
1.2G+1.5Q	0.77	5.99	23.24	0.00	24.00	75.5
G+Eu (Lateral)	1.00	-7.12	13.59	5.40	15.34	62.3
0.9G+Wu (Lateral)	1.00	-13.52	-36.53	0.00	38.95	69.7
1.2G+Wu (Lateral)	1.00	-20.98	22.19	0.00	30.54	46.6
G+Eu (Longitudinal) - uplift	1.00	-2.14	-14.06	18.00	14.22	81.4
G+Eu (Longitudinal) -down	1.00	-2.14	23.96	18.00	24.05	84.9
0.9G+Wu (Long)	1.00	8.41	-42.02	11.40	42.85	78.7
1.2G+Wu (Long)	1.00	2.72	34.59	11.40	34.70	85.5

(-ve = uplift)

k1 adjusted values

Load Case	k_1	PF2 R_x kN	R_y kN	R_z kN	$(R_x^2+R_y^2)^{0.5}$ kN	Angle
1.35g	0.57	4.54	19.67	0.00	20.18	77.0
1.2G+1.5Q	0.77	7.78	30.18	0.00	31.17	75.5
G+Eu (Lateral)	1.00	-7.12	13.59	5.40	15.34	62.3
0.9G+Wu (Lateral)	1.00	-13.52	-36.53	0.00	38.95	69.7
1.2G+Wu (Lateral)	1.00	-20.98	22.19	0.00	30.54	46.6
G+Eu (Longitudinal) - uplift	1.00	-2.14	-14.06	18.00	14.22	81.4
G+Eu (Longitudinal) -down	1.00	-2.14	23.96	18.00	24.05	84.9
0.9G+Wu (Long)	1.00	8.41	-42.02	11.40	42.85	78.7
1.2G+Wu (Long)	1.00	2.72	34.59	11.40	34.70	85.5

Note: Gravity and downwards loads are taken out in bearing

Column dimensions at base

D = 600 mm
B = 75 mm

Critcal design load -PF2 42.85 kN (refer above)
Critcal design load 42.85 kN

Determine number of screws for connection of column to bracket

Joint group JD4 for type 17 screws, lateral load

$$Q_n = n.k.Q_k$$

$$\varnothing Q_n = \varnothing.n.k.Q_k$$

where:

Q_k = 3.13 kN 14gx50 Type 17 screws
k_1 = 1.00 -load factors taken into account above
Ø = 0.80
k = 1.20 -screws driven through close fitting holes in steel side plates

ØQn = 3.00 n.kN

Therefore n = 14.26 14gx50 Type 17 screws

Say 24 14gx50 Type 17 screws (per bracket)
i.e. 12 per side

Calculate bracket thickness and hold down bolt capacity

N^*_t = -42.02 kN
N^*_c = 34.59 kN
V^*_x = 20.98 kN
V^*_z = 18.00 kN
V* = 27.64 kN

Consider Hold Down Bolt Capacity

4/ CS16190 Chemset anchor with Epcon C6 epoxy - refer ramset calculation sheet

N^*_t = -10.5 kN per anchor

V^* = 6.9 kN per anchor

Base Plate Design

N^* = -42.02 kN

V^* = 27.64 kN

Plate Design -tension buckling

$$ØN_{tp} = \frac{Ø f_y.(t_p^2 + t_w^2) b_i}{e}$$

Ø	=	0.90	
f_y	=	250.00	MPa
t_{plate}	=	3.00	mm
t_{washer}	=	8.00	mm
b_i	=	200.00	mm
e	=	50.00	mm

$ØN_{tp}$ = 65.70 kN > N* o.k

Shear Force Design

$$ØV_1 = 2(Ø.0.5.f_y.t_i.b_i)$$

$ØV_1$ = 135.00 kN

$$ØV_2 = \frac{Ø.f_y.t_i.b_i^2}{4.e}$$

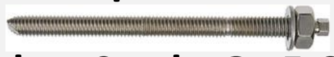
$ØV_2$ = 270.00 kN

ØV = 135.00 kN > V* o.k

Chemical Anchoring - ChemSet Anchor Stud Design Calculator

Ramset™

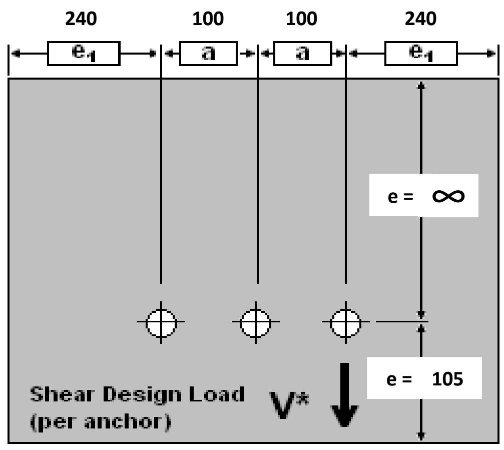
Non-Cracked Concrete - Epcon™ C6



Anchor Type: Anchor Stud - Gr 5.8 | Anchor Size d_b = M 16

Input Description (Strength Limit State Design)	Input Data (per anchor)		
1. Number of anchors (n)	n =	2	-
2. Anchor Spacing (a)	a =	100	mm
3. Concrete Edge Distance (e)	e =	105	mm
4. Concrete Cylinder Strength (f'c)	f'c =	25	MPa
5. Seismic Crack (S_{cr}) or Non-Cracked (N)	S_{cr} or N	N	-
6. Effective Depth (h>6xdh)	h =	130	mm
7. Anchor Stud size (d_b) - M8 → M24	d_b =	16	mm
8. Concrete Edge Distance Corner (e1)	e_1 =	240	mm
9. Internal to a row (I) or end of row (E)	I or E	E	row
10. Dry Hole (D) or Wet hole (W)	D or W	W	-
11. Min Concrete Sub'te Thickness (b_m)	b_m =	166	mm
12. Anchor Stud Grade (5.8, 8.8, 316 SS)	Grade =	5.8	Gr
13. Fixture Thickness (t)	t =	12	mm
14. Effective Length (L_e)	L_e =	142	mm
15. Design Tensile Load-per anchor (N*)	N* =	15	kN
16. Design Shear Load-per anchor (V*)	V* =	8	kN
17. Direction of Shear design load (α)	α =	0	°
18. Service Temperature (˚C)	-40˚C to	+45˚C	-

Plan View - Generic
Dimensions in (mm)



Project Details

Project Name:-

Project Site Address:-

Company Name:-

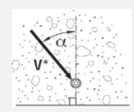
Design Identification:-

Date:-

ChemSet™ Anchor Stud "Specification"				Epcon™ C6 Series "Specification"
Part No.	CS16190	or	CS16190GH	C6-18

Hole Diameters (mm)	Capacity Reduction Factors
Drill d_h = 18	Conc. Tension/Shear ϕ = 0.6
Fixture d_f = 20	Steel Tension/Shear ϕ = 0.8

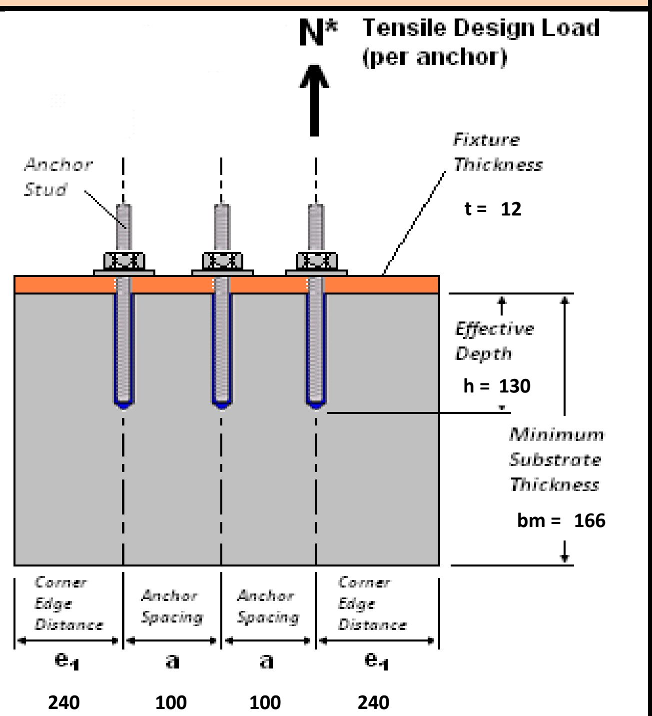
Direction of Shear Design Load - α



α = 0˚- towards edge
α = 180˚ - away from edge

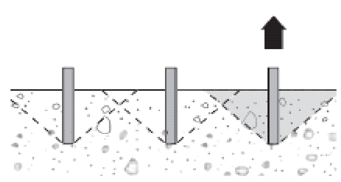
Output Description (Strength Limit State Design)	Output Data (per anchor)		
DESIGN O.K.	MIN. CRITERIA for a,e & h - O.K.		
Des. Red. Ult. CONC. Tensile Capacity	ϕN_{urc} =	31.3	kN
Red. Char. Ult. STEEL Tensile Capacity	ϕN_{us} =	64.7	kN
Des. Red. Ult. CONC. Shear Capacity	ϕV_{urc} =	11.5	kN
Red. Char. Ult. STEEL Shear Capacity	ϕV_{us} =	39.7	kN
Drill hole diameter	d_h =	18	mm
TENSION O.K.			
Design Red. Ult. Tensile Capacity	ϕN_{ur} =	31.3	kN
Tension Design Check	$N^*/\phi N_{ur}$ =	0.48	< 1
SHEAR O.K.			
Design Red. Ult. Shear Capacity	ϕV_{ur} =	11.5	kN
Shear Design Check	$V^*/\phi V_{ur}$ =	0.69	< 1
COMBINED TENSION SHEAR O.K.			
Combined Check - $N^*/\phi N_{ur} + V^*/\phi V_{ur}$ =		1.17	< 1.2

Elevation View - Generic
Dimensions in (mm)

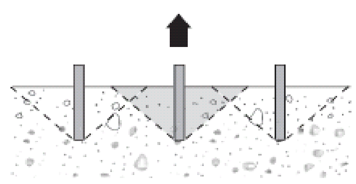


Anchor Loaded

"E" *Anchor end of a row*



"I" *Anchor internal to a row*



Anchor Size	d_b	Metric	8	10	12	16	20	24	30	36
Drill hole diameter	d_h	(mm)	10	12	14	18	24	26	N/A	N/A
Stressed Area	A_b	(mm²)	33	53	79	154	232	337	N/A	N/A
Anchor Stud Yield Strength	f_y	(MPa)	430	430	430	420	420	420	N/A	N/A
Red.Char. Ult. Steel Tensile Capacity	ϕN_{us}	(kN)	14.3	22.7	33.8	64.7	97.6	141.3	N/A	N/A
Red.Char. Ult. Steel Shear capacity	ϕV_{us}	(kN)	8.9	14.1	21.0	39.7	59.9	86.8	N/A	N/A
Edge distance for no conc.cone reduction	e_c	**(mm)**	**35**	**40**	**50**	**65**	**80**	**100**	**N/A**	**N/A**
Anchor spacing for no conc.cone reduction	a_c	**(mm)**	**50**	**60**	**75**	**100**	**120**	**145**	**N/A**	**N/A**
Absolute Minimum edge dist. & anc'r spac.	e_m & a_m	**(mm)**	**25**	**30**	**35**	**50**	**60**	**75**	**N/A**	**N/A**

Effective Depth - h		Design Reduced Ultimate tensile capacity ϕN_{ur} (kN per anchor) Based on edge distance (e_c) and anchor spacing (a_c) for no conc.cone reduction							
60	(mm)	7.4							
65	(mm)	8.4							
70	(mm)	9.4	10.3						
80	(mm)	11.5	12.5						
90	(mm)	13.7	14.9	16.1					
100	(mm)	14.3	17.5	18.8					

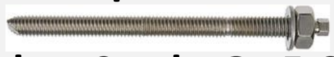
110	(mm)	14.3	20.2	21.7	24.4				
125	(mm)	14.3	22.7	26.3	29.5				
140	(mm)	14.3	22.7	31.2	35.0				
150	(mm)	14.3	22.7	33.8	38.8	42.5			
160	(mm)	14.3	22.7	33.8	42.8	46.8	50.3		
170	(mm)	14.3	22.7	33.8	46.8	51.2	55.1		
180	(mm)	14.3	22.7	33.8	51.0	55.8	60.0		
190	(mm)	14.3	22.7	33.8	55.4	60.5	65.1	N/A	
220	(mm)	14.3	22.7	33.8	64.7	75.4	81.1	N/A	
240	(mm)	14.3	22.7	33.8	64.7	85.9	92.4	N/A	N/A
270	(mm)	14.3	22.7	33.8	64.7	97.6	110.3	N/A	N/A
330	(mm)	14.3	22.7	33.8	64.7	97.6	141.3	N/A	N/A
350	(mm)	14.3	22.7	33.8	64.7	97.6	141.3	N/A	N/A

The design engineer should ensure the structural element is capable of supporting these loads. Refer to Ramset™ Specifiers Anchoring Resource Book ANZ for more information or explanation of Tech. Data.

Chemical Anchoring - ChemSet Anchor Stud Design Calculator

Ramset™

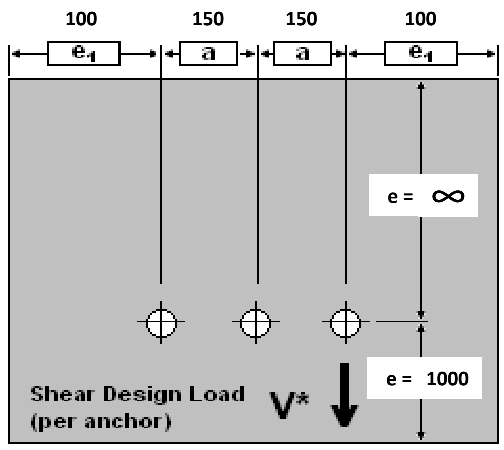
Non-Cracked Concrete - Epcon™ C6



Anchor Type: Anchor Stud - Gr 5.8 Anchor Size d_b = M 12

Input Description (Strength Limit State Design)	Input Data (per anchor)		
1. Number of anchors (n)	n =	2	-
2. Anchor Spacing (a)	a =	150	mm
3. Concrete Edge Distance (e)	e =	1000	mm
4. Concrete Cylinder Strength (f'c)	f'c =	25	MPa
5. Seismic Crack (S_{cr}) or Non-Cracked (N)	S_{cr} or N	N	-
6. Effective Depth (h>6xdh)	h =	110	mm
7. Anchor Stud size (d_b) - M8 → M24	d_b =	12	mm
8. Concrete Edge Distance Corner (e1)	e_1 =	100	mm
9. Internal to a row (I) or end of row (E)	I or E	e	row
10. Dry Hole (D) or Wet hole (W)	D or W	w	-
11. Min Concrete Sub'te Thickness (b_m)	b_m =	138	mm
12. Anchor Stud Grade (5.8, 8.8, 316 SS)	Grade =	5.8	Gr
13. Fixture Thickness (t)	t =	8	mm
14. Effective Length (L_e)	L_e =	118	mm
15. Design Tensile Load-per anchor (N*)	N* =	5	kN
16. Design Shear Load-per anchor (V*)	V* =	7	kN
17. Direction of Shear design load (α)	α =	90	°
18. Service Temperature (˚C)	-40˚C to	+45˚C	-

Plan View - Generic
Dimensions in (mm)



Project Details

Project Name:-

Project Site Address:-

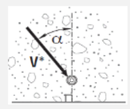
Company Name:-

Design Identification:-

Date:-

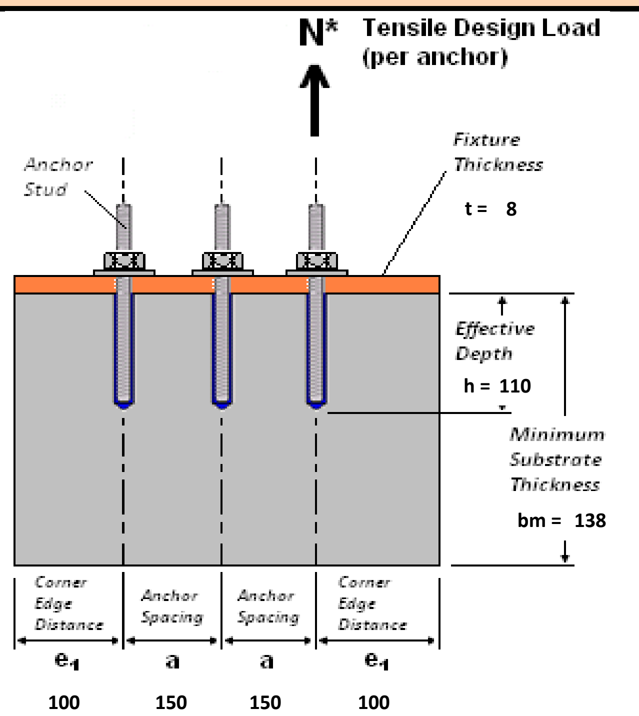
ChemSet™ Anchor Stud "Specification"	Epcon™ C6 Series "Specification"
Part No. CS12160 or CS12160GH	C6-18

Hole Diameters (mm)	Capacity Reduction Factors	Direction of Shear Design Load - α
Drill d_h = 14	Conc. Tension/Shear ϕ = 0.6	α = 0˚- towards edge
Fixture d_f = 15	Steel Tension/Shear ϕ = 0.8	α = 180˚ - away from edge



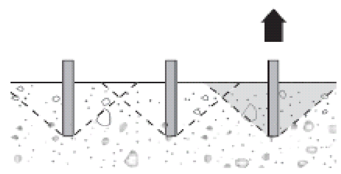
Output Description (Strength Limit State Design)	Output Data (per anchor)		
DESIGN O.K.	MIN. CRITERIA for a,e & h - O.K.		
Des. Red. Ult. CONC. Tensile Capacity	ϕN_{urc} =	21.7	kN
Red. Char. Ult. STEEL Tensile Capacity	ϕN_{us} =	33.8	kN
Des. Red. Ult. CONC. Shear Capacity	ϕV_{urc} =	7.3	kN
Red. Char. Ult. STEEL Shear Capacity	ϕV_{us} =	21.0	kN
Drill hole diameter	d_h =	14	mm
TENSION O.K.			
Design Red. Ult. Tensile Capacity	ϕN_{ur} =	21.7	kN
Tension Design Check	$N^*/\phi N_{ur}$ =	0.23	< 1
SHEAR O.K.			
Design Red. Ult. Shear Capacity	ϕV_{ur} =	7.3	kN
Shear Design Check	$V^*/\phi V_{ur}$ =	0.96	< 1
COMBINED TENSION SHEAR O.K.			
Combined Check - $N^*/\phi N_{ur} + V^*/\phi V_{ur}$ =		1.19	< 1.2

Elevation View - Generic
Dimensions in (mm)

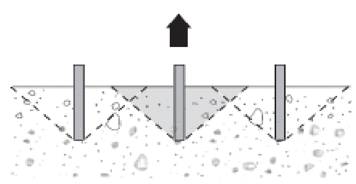


Anchor Loaded

"E" *Anchor end of a row*



"I" *Anchor internal to a row*



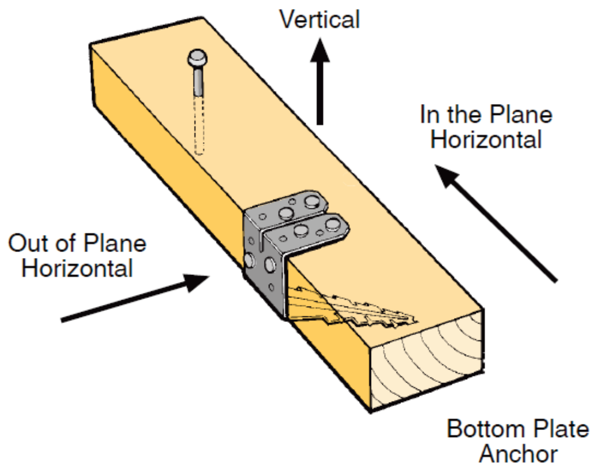
Anchor Size	d_b	Metric	8	10	12	16	20	24	30	36
Drill hole diameter	d_h	(mm)	10	12	14	18	24	26	N/A	N/A
Stressed Area	A_b	(mm²)	33	53	79	154	232	337	N/A	N/A
Anchor Stud Yield Strength	f_y	(MPa)	430	430	430	420	420	420	N/A	N/A
Red.Char. Ult. Steel Tensile Capacity	ϕN_{us}	(kN)	14.3	22.7	33.8	64.7	97.6	141.3	N/A	N/A
Red.Char. Ult. Steel Shear capacity	ϕV_{us}	(kN)	8.9	14.1	21.0	39.7	59.9	86.8	N/A	N/A
Edge distance for no conc.cone reduction	e_c	**(mm)**	**35**	**40**	**50**	**65**	**80**	**100**	**N/A**	**N/A**
Anchor spacing for no conc.cone reduction	a_c	**(mm)**	**50**	**60**	**75**	**100**	**120**	**145**	**N/A**	**N/A**
Absolute Minimum edge dist. & anc'r spac.	e_m & a_m	**(mm)**	**25**	**30**	**35**	**50**	**60**	**75**	**N/A**	**N/A**

Effective Depth - h		Design Reduced Ultimate tensile capacity ϕN_{ur} (kN per anchor) Based on edge distance (e_c) and anchor spacing (a_c) for no conc.cone reduction							
60	(mm)	7.4							
65	(mm)	8.4							
70	(mm)	9.4	10.3						
80	(mm)	11.5	12.5						
90	(mm)	13.7	14.9	16.1					
100	(mm)	14.3	17.5	18.8					

110	(mm)	**14.3**	**20.2**	**21.7**	**24.4**				
125	(mm)	**14.3**	**22.7**	**26.3**	**29.5**				
140	(mm)	**14.3**	**22.7**	**31.2**	**35.0**				
150	(mm)	**14.3**	**22.7**	**33.8**	**38.8**	**42.5**			
160	(mm)	**14.3**	**22.7**	**33.8**	**42.8**	**46.8**	**50.3**		
170	(mm)	**14.3**	**22.7**	**33.8**	**46.8**	**51.2**	**55.1**		
180	(mm)	**14.3**	**22.7**	**33.8**	**51.0**	**55.8**	**60.0**		
190	(mm)	**14.3**	**22.7**	**33.8**	**55.4**	**60.5**	**65.1**	**N/A**	
220	(mm)	**14.3**	**22.7**	**33.8**	**64.7**	**75.4**	**81.1**	**N/A**	
240	(mm)	**14.3**	**22.7**	**33.8**	**64.7**	**85.9**	**92.4**	**N/A**	**N/A**
270	(mm)	**14.3**	**22.7**	**33.8**	**64.7**	**97.6**	**110.3**	**N/A**	**N/A**
330	(mm)	**14.3**	**22.7**	**33.8**	**64.7**	**97.6**	**141.3**	**N/A**	**N/A**
350	(mm)	**14.3**	**22.7**	**33.8**	**64.7**	**97.6**	**141.3**	**N/A**	**N/A**

The design engineer should ensure the structural element is capable of supporting these loads. Refer to Ramset™ Specifiers Anchoring Resource Book ANZ for more information or explanation of Tech. Data.

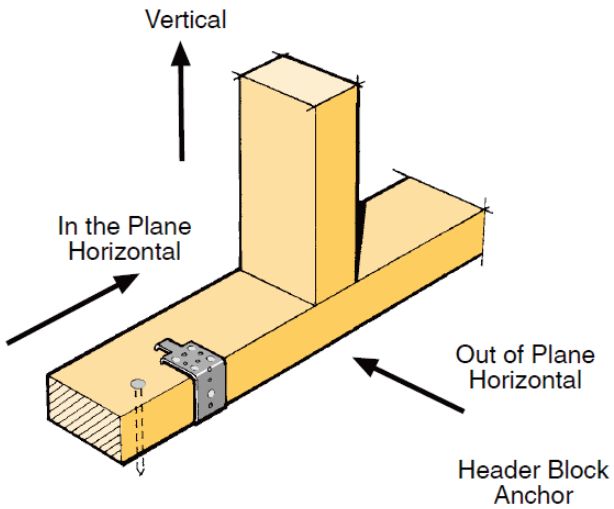
Appendix 6 – Proprietary connector manufacturer's literature



BOTTOM PLATE ANCHOR

0.95mm G300 Z450 GALVANISED STEEL

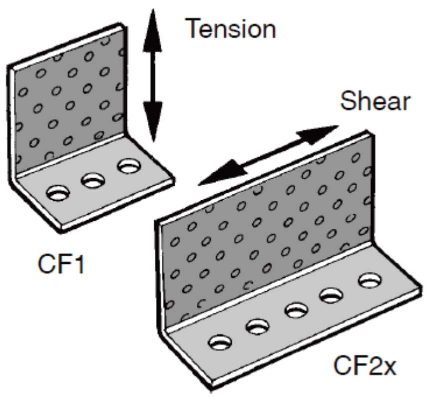
Characteristic Load	
Vertical	8.0 kN
In the Plane Horizontal	7.0 kN
Out of Plane Horizontal	7.0 kN
Nails as shown: 6 x LUMBERLOK 30mm x 3.15 dia. Product Nails **AND** 75mm x 4 dia. concrete nail adjacent to B. P. Anchor	



HEADER BLOCK ANCHOR

0.95mm G300 Z450 GALVANISED STEEL

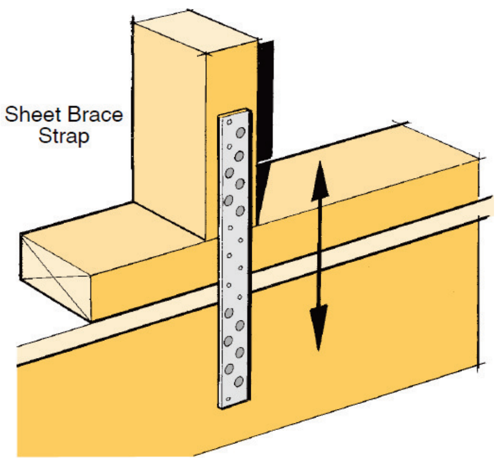
Characteristic Load	
Vertical	9.0 kN
In the Plane Horizontal	7.0 kN
Out of Plane Horizontal	7.0 kN
Nails as shown: 5 x LUMBERLOK 30mm x 3.15 dia. Product Nails **AND** 75mm x 4 dia. concrete nail adjacent to H. B. Anchor	



CONCRETE FIXING CLEAT

1.55mm G300 Z275 GALVANISED STEEL

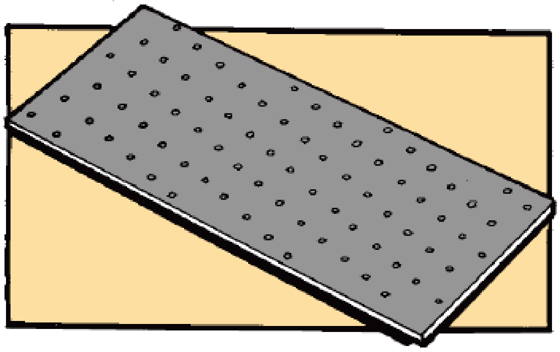
Load	CF1 (single cleat)		CF2x (single cleat)	
	1 x M12 Bolt with Washer		2 x M12 Bolts with Washers	
	18 Nails (30mm x 3.15)	6 Screws (Type 17 35mm x 14g)	30 Nails (30mm x 3.15)	10 Screws (Type 17 35mm x 14g)
Tension	13.5 kN	13.5 kN	27.0 kN	27.0 kN
Shear	9.5 kN	11.0 kN	16.0 kN	18.5 kN



SHEET BRACE STRAP 200, 300, 400, 600mm

0.91mm G300 Z275 GALVANISED STEEL or
0.9mm STAINLESS STEEL 304-2B

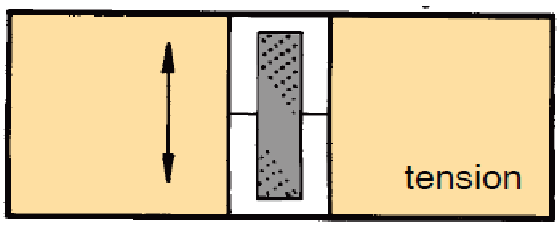
Characteristic Load	
Tension	6.0 kN
Nails: 6 x LUMBERLOK 30mm x 3.15 dia. Product Nails per End as shown	

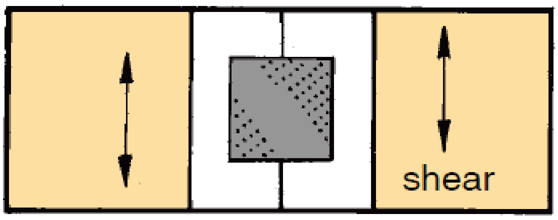


NAILON PLATE

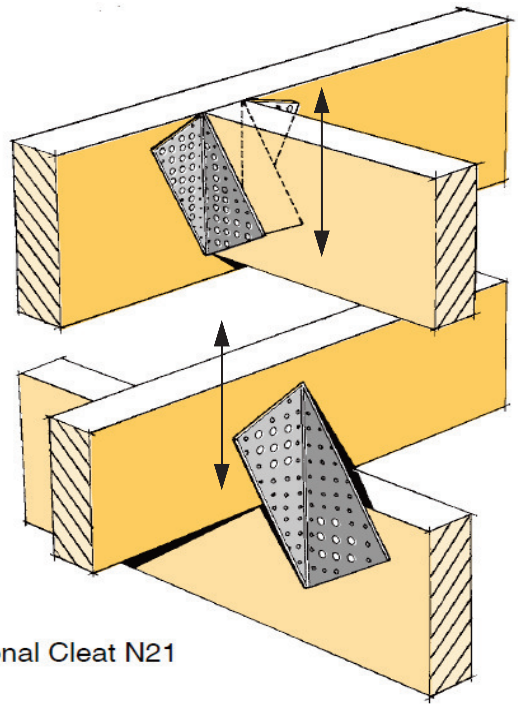
1mm x 110mm wide - 0.91mm G300 Z275 GALVANISED STEEL
1mm x 110mm wide - 0.9mm STAINLESS STEEL 304-2B
2mm x 113mm wide - 1.55mm G300 Z275 GALVANISED STEEL
3mm x 130mm wide - 3.0mm NZCC-SD STEEL
3mm x 240mm wide - 3.0mm NZCC-SD STEEL

Characteristic Load (Fastener)		
Nail:	30mm x 3.15 dia.	1.0 kN/Nail
Screw:	Type 17 35mm x 14g	3.0 kN/Screw
	Type 17 35mm x 12g S/S	2.5 kN/Screw





Characteristic Load (Plate)		
1mm Plate Galv.	Tension Shear	210 N/mm per plate 163 N/mm per plate
1mm Plate S/S	Tension Shear	198 N/mm per plate 119 N/mm per plate
2mm Plate	Tension Shear	367 N/mm per plate 279 N/mm per plate
3mm Plate	Tension Shear	480 N/mm per plate 288 N/mm per plate
110mm wide – 40 nails OR 15 screws per 200mm plate length 113mm wide – 40 nails OR 15 screws per 200mm plate length 130mm wide – 50 nails OR 15 screws per 200mm plate length 240mm wide – 95 nails OR 25 screws per 200mm plate length		



Diagonal Cleat N21

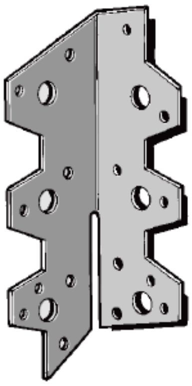
DIAGONAL CLEAT N21

0.91mm G300 Z275 GALVANISED STEEL or
0.9mm STAINLESS STEEL 304-2B

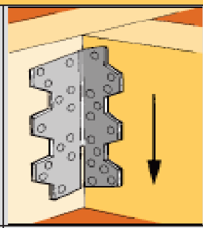
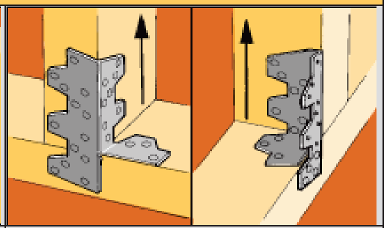
Characteristic Load		
Nail:	30mm x 3.15 dia.	1.0 kN/Nail
Screw:	Type17 35mm x 14g	3.0 kN/Screw
	Type17 35mm x 12g S/S	2.5 kN/Screw
Plate:	Shear / Tension	20 kN/Plate
20 x Nails per Flange OR 5 x Screws per Flange		

MULTIGRIP

0.91mm G300 Z275 GALVANISED STEEL or
0.9mm STAINLESS STEEL 304-2B

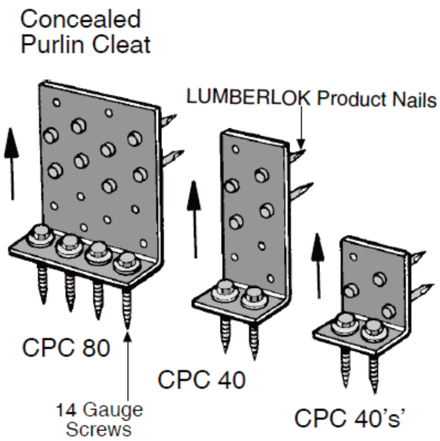


Multi Grip

Characteristic Load		
		
	Shear	Tension
Nails (All holes with 30mm x 3.15 nails)	11.9 kN / pair	4.0 kN
Screws (3 / Type17 35mm x 14g per flange)	10.9 kN / pair	-

CONCEALED PURLIN CLEAT

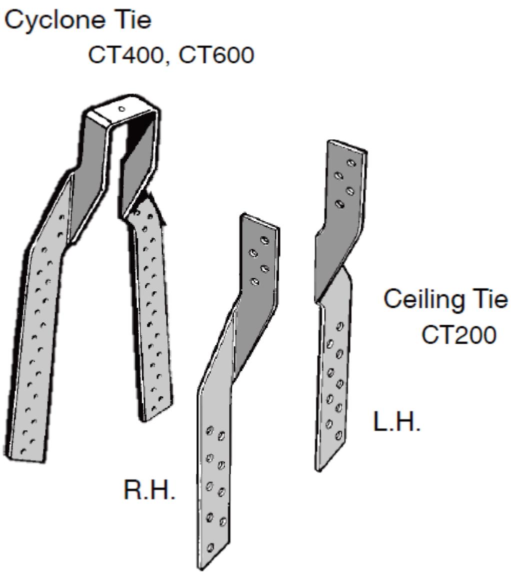
1.55mm G300 Z275 GALVANISED STEEL or
0.9mm STAINLESS STEEL 304-2B



Characteristic Load	CPC40s	CPC40	CPC80
Uplift / Tension	5.0 kN/pair	8.0 kN/pair	16.0 kN/pair
Fix as shown with - LUMBERLOK Product Nails 30mm x 3.15 dia. Type 17 35mm x 14g Screws Note: Stainless Steel use Type 17 35mm x 12g S/S Hex Head Screws			

CEILING TIE CT200 and CYCLONE TIE CT400, CT600

0.91mm G300 Z275 GALVANISED STEEL or
0.9mm STAINLESS STEEL 304-2B



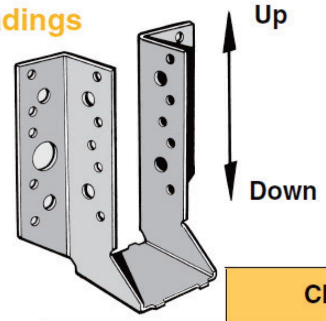
Characteristic Load	CT200 (pair)	CT400, CT600
Uplift / Tension (4 Nails each end)	10.5 kN	
Uplift / Tension (6 Nails each end)	-	12.0 kN
All nails to be LUMBERLOK Product Nails 30mm x 3.15 dia.		

JOIST HANGERS

Material: 0.91mm G300 Z275 Galvanised Steel
or 0.9mm Stainless Steel 304-2B

USE STAINLESS STEEL OPTION IN EXTERIOR SITUATIONS

Loadings



Joist Hangers are also available in:
52x90, 52x120 and 52x190 to suit rough sawn timber
37x90, 37x120 and 37x190 to suit 35mm gauged timber

All sizes (except 37mm wide) are also available in Stainless Steel 304-2B

	Characteristic Load - Nails			Characteristic Load - Screws		
Joist Hanger Type	**No. of Nails per Flange***	**Down**	**Uplift**	**No. of Screws per Flange***	**Down**	**Uplift**
JH 47 x 90	3	9.0 kN	6.0 kN	1	7.0 kN	4.7 kN
JH 47 x 120	5	15.0 kN	10.0 kN	2	14.0 kN	12.0 kN
JH 47 x 190	9	27.0 kN	18.0 kN	3	21.0 kN	18.0 kN
JH 95 x 165	8	24.0 kN	16.0 kN	3	21.0 kN	18.0 kN
JH 70 x 180	8	24.0 kN	16.0 kN	3	21.0 kN	18.0 kN
Nail with LUMBERLOK Product Nails 30mm x 3.15 dia.				Fix with Type 17-12g x 35mm Hex Head Screws		

* 4 Flanges per hanger

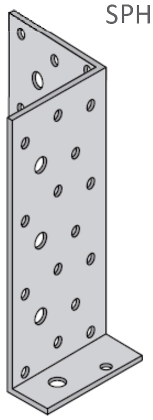
Note: Loads for 47mm Joist Hangers also apply to 52mm & 37mm.

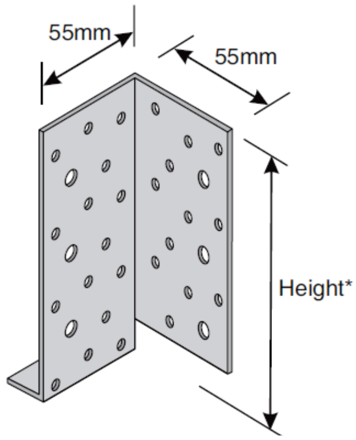
SPLIT HANGERS

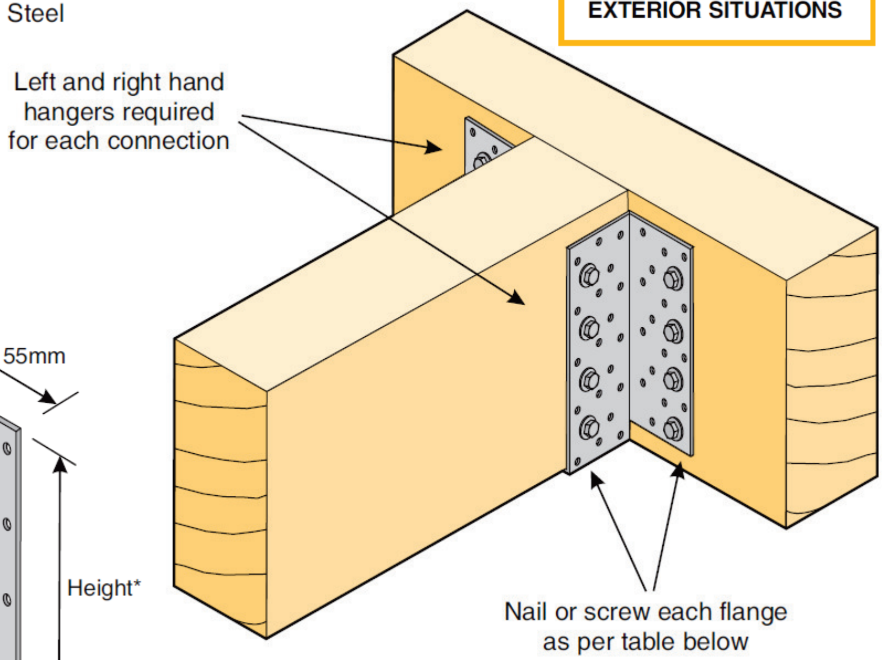
Material: 1.55 G300 Z275 Galvanised Steel

NOT TO BE USED IN EXTERIOR SITUATIONS

- Always used in pairs.
- Split Hangers are available in heights of: SPH140 = 140mm
  SPH180 = 180mm
  SPH220 = 220mm





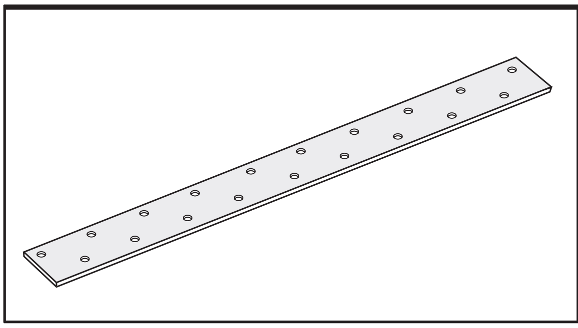


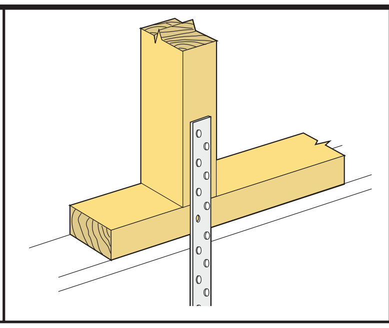
Loadings

	Characteristic Loads (per pair) - Nails			Characteristic Loads (per pair) - Screws		
Hanger Type	**No. of Nails per Flange**	**Down**	**Uplift**	**No. of Screws per Flange**	**Down**	**Uplift**
SPH140	8	24.0 kN	16.0 kN	3	21.0 kN	18.0 kN
SPH180	10	30.0 kN	20.0 kN	4	28.0 kN	24.0 kN
SPH220	12	36.0 kN	24.0 kN	5	35.0 kN	30.0 kN
Nail with LUMBERLOK Product Nails 30mm x 3.15 dia.				Fix with Type 17-14g x 35mm Hex Head Screws		

Sheet Brace Straps

6kN or 12kN Capacity fixing for sheet-braced wall panels.





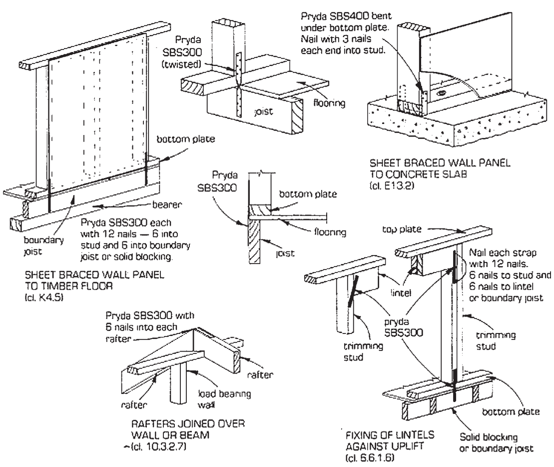
Features

Mild steel strap fixing complying with the requirements of NZS3604:2011 for a 6kN Capacity Strap. Use 2 straps, each with nailing as below, where 12kN Capacity fixing is required. Where straps longer than 600mm are required 10m & 30m coils are now available. The coils can easily be cut to the required length.

Applications

Nails with 6 nails at each end of strap for 6KN Capacity fixing. Use Pryda product nails (30 x 3.15mm Hot Dip Galvanised Bracket Nails).

(figures in brackets refer to NZS3604:1990 Clause numbers.)



Specifications

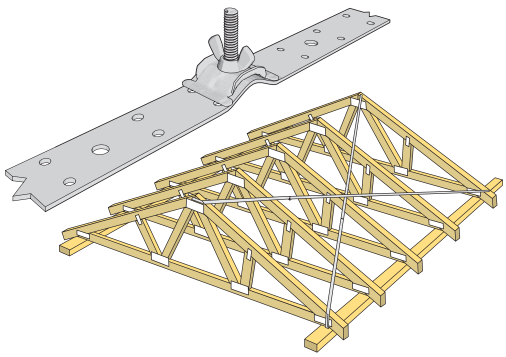
Sizes:	25 x 1.0 x 300mm	
	25 x 1.0 x 400mm	
	25 x 1.0 x 600mm	
	25 x 1.0mm x 10m	
	25 x 1.0mm x 30m	
Material:	1.0mm G300 Z275 galvanised steel coil or stainless steel.	
Product Code:	**Galvanised**	**Stainless**
	SBS300	SBS300/S
	SBS400	SBS400/S
	SBS600	SBS600/S
10m coil	SBS10M	
30m coil	SBS30M	
Packing:	Galvanised Steel:	50 per ctn
	Stainless Steel:	10 per ctn
	Coils:	each.
Nails:	30 x 3.15mm Pryda Product Nails.	

Installation

Refer to Pryda Building Guide.

Strapbrace & Maxi Strap

Convenient multi-purpose bracing for roofs, ceilings and walls.



Truss and Rafter Bracing

Applications

Pryda Strapbrace is suitable for bracing walls and truss/rafter roof construction (spans up to 12m) in residential buildings. Use Pryda Maxi Strap for larger spans and commercial and industrial buildings. Pryda Tensioners provide a fast, reliable and simple method of tensioning long lengths of bracing strap.

Pryda Strapbrace complies with NZS3604:2011 Light Timber Frame Buildings, requirements for metal bracing strip with 8kN capacity, but is not suitable for use as holding down straps on braced wall panels. Use Pryda Sheet Brace Straps for this application.

Pryda Strapbrace and Maxi Strap act in tension only. Braces must be applied in pairs as illustrated. Holes are pre-punched for Pryda Product Nails 30 x 3.15mm and 6mm tensioner bolts.

Specifications

Product Code Galvanised	Size	Packing
SB10	Strapbrace 10m (25mm)	each
SB10T	Strapbrace 10m plus 5 tensioners (25mm)	each
SB30	Strapbrace 30m (25mm)	each
SB30T	Strabrace 30m with 5 tensioners (25mm)	each
SBT	Strapbrace Tensioners - 8 bags of 5	1 ctn
SBI/10	Maxi Strap 10m (50mm)	each
SBI/15	Maxi Strap 15m (50mm)	each
SBI	Maxi Strap 30m (50mm)	each
SBI/T	Maxi Strap Tensioner	each
Stainless Steel		
SB15/S	Strapbrace 15m (25mm)	each
SBT/SS316	Strapbrace Tensioner (25mm)	each
SBI/S	Maxi Strap 30m (50 mm)	each
SBI/TS	Maxi Strap Tensioner	each
Material:	Strapbrace: 25 x 0.8mm G550 Z275 galvanised or stainless steel	
	Maxi Strap: 50 x 0.8mm G550 Z275 galvanised or stainless steel	

Loads (Wind only)

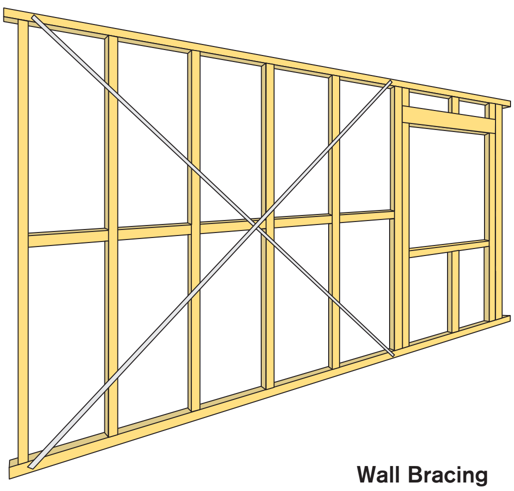
Steel Strength	Characteristic Strength (kN)	Wind Loading (kN)
Strapbrace	8.2	6.6
Maxi Strap	16.4	13.2

Strapbrace & Maxi Strap

Continued.

Wall Bracing

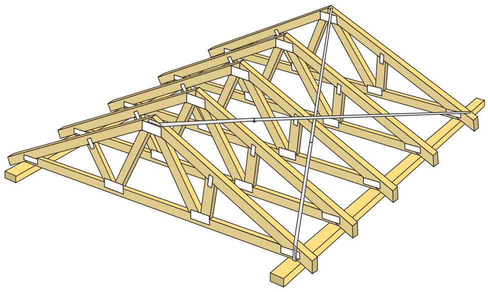
- Make sure wall frame is approximately square. Nail end of brace to the top wall plate within 150mm of a stud, using 3 x Pryda Product Nails 30 x 3.15mm. Unroll brace coil at angle of approximately 45 and cut to length. Tighten by pulling down onto bottom wall plate. Nail within 150mm of stud with 3 nails.
- Fix another brace in the same way diagonally opposite the first length. The two braces must cross to form a strong rigid brace. Fit tensioners (usually one per 3.6m length of brace) and plumb frame. Nail braces to intermediate studs with 2 nails after tensioning braces.



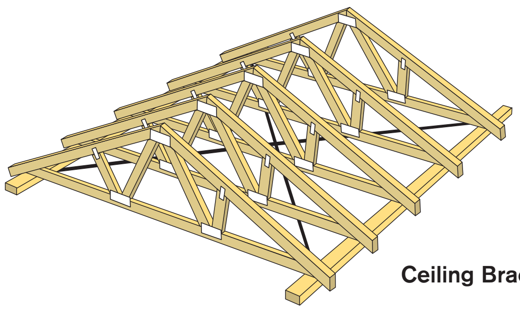
Wall Bracing

Roof and Ceiling Bracing

Use in crossed pairs as for wall braces. For residential construction in accordance with NZS3604:2011, secure braces with 6 nails (12 nails for Maxi Strap) at each end, and 2 nails (after tensioning braces) at truss/ rafter or Purlin crossing.



Truss and Rafter Bracing



Ceiling Bracing