Standard Steel Framed

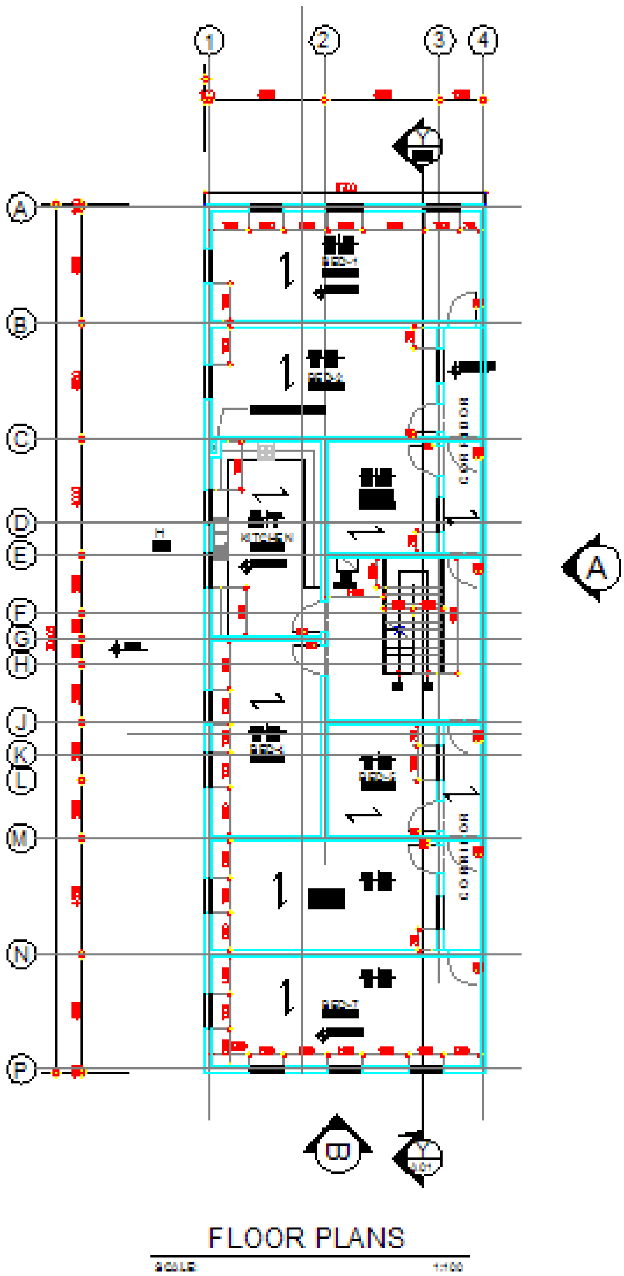
Labour Camp Unit

Design Assessment

Prepared For

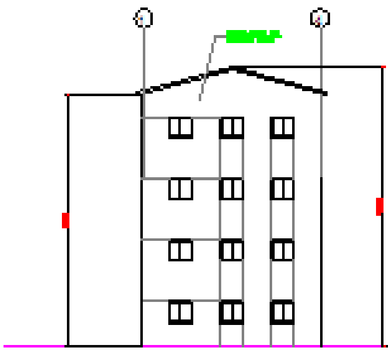
BRE Assessment

Phone 07866731125 Fax 02089562374 email serdar.dundar@gmail.com

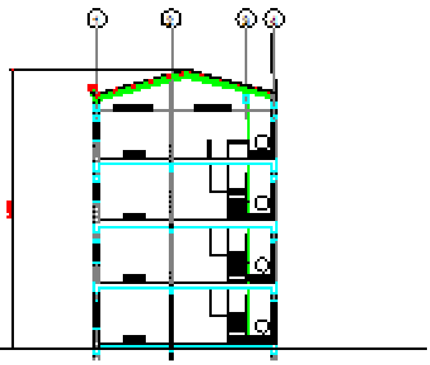


FLOOR PLANS

ELEVATION-A



FRONT ELEVATION



PROPOSED SECTION

ELEVATIONS AND SECTION

This document forms an assessment of construction for 4 storey labour camp unit in light gauge cold rolled steel.

Structure

Internally clad with fire-rated plasterboard. Externally clad with tile roof, and rendered walls.

Light gauge steel framed utilizing Scottsdale Construction Systems 140 and 90 mm C-Sections in S350G steel grade throughout.

Design Parameters

The strength of the Sections are based on **AS/NZS 4600** which is the limit state code for Australia and New Zealand. This Code is similar to the concepts of **BS 5950-5** but incorporates more detailed checking for torsional and distortional buckling than the present United Kingdom and American Codes. The difference in strengths is nominal and a comparison will be drawn between the particular section strengths as an Appendix to this Design Assessment. Using As/NZS 4600 is conservative.

Load Parameters

This Design based on Loading in accordance with **BS 6399** Parts 1,2 and 3 and follows normal Limit State Design. Loads Considered are:

Dead Loads – self weight, finishes etc

Imposed Loads – live load, snow load, wind load

Notional loads – are neglected for this building and assumption is made that the bracing system provided will take the notional loads.

Design Methodology

Although it is possible to analyse and check every component stud or joist in the structure, this type of structure lends itself to a simple strip approach in design.

Because the structure is a highly redundant series of common panel components linked in both directions, the method of assessment will be by tracking load paths through a 'strip' of building from roof to ground. This permits the average or maximum loading to be checked against the strength of a metre wide strip of connected components. This is a simple Pass/Fail approach for any selected part of the structure.

All SCS sections have load span graphs available which may be used to determine the strength in bending or axial loading for any combination of length and lateral braced Condition. From these basic parameters, any combination of strength may be calculated.

In this instance however, several spreadsheets and subroutines are written to calculate the loads along two load paths of the building. For every floor level these two load paths are analysed and then the overall stability of the building is handled with additional subroutines. X braces are used to take the lateral loads but it is also demonstrated how much does the plasterboard and a structural timber board will add to the lateral strength of the building (OSB boards assumed to be added external walls as well as the bitrock board and the added strength is calculated).

Wind Load Calculations:

The wind load calculations are done according to BS6399-Part2. Since the procedure is very time consuming to do by hand a free software is used to make analysis and the report is attached below. The basic wind speed is taken as 20.50m/sec which is basic wind speed for London. The grid reference and the input details are listed below.

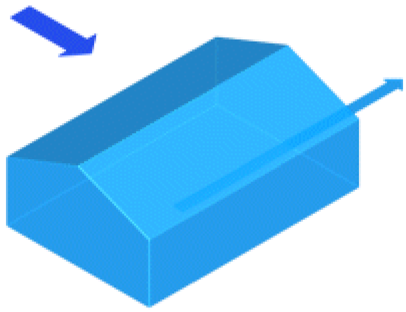
SCS Europe- Scottsdale
500 Chiswick High Road
Center 500 – Suite 35
W4 5RG
London

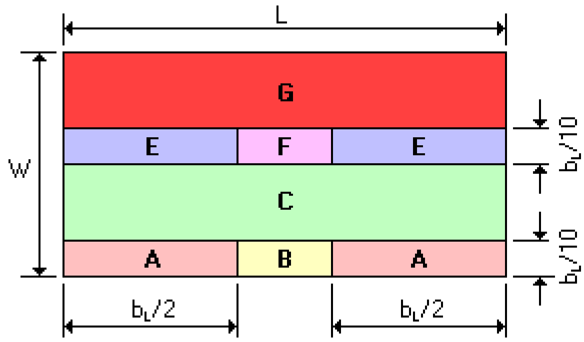
Telephone: 07952880084 Fax: 020 8956 2374 Email: serdar.dundar@gmail.com

Calculation Method:	Hybrid			
Grid Reference:	TQ201778		Probability Factor:	1.000
Site Altitude:	5.000	m	Duration	All year
Basic Wind Speed:	20.500	m/s		
Annual Risk:	0.02000		Seasonal Factor:	1

Roof results, wind normal to face 1

Zone	Zone width & depth	External applied loads	Internal applied loads	Overall applied loads
A	15.000 m, 3.000 m	+0.098, -0.538 kN/m²	+0.084, -0.125 kN/m²	+0.223, -0.622 kN/m²
B	0.000 m, 3.000 m	+0.098, -0.391 kN/m²	+0.084, -0.125 kN/m²	+0.223, -0.475 kN/m²
C		+0.098, -0.196 kN/m²	+0.084, -0.125 kN/m²	+0.223, -0.279 kN/m²
E		-0.636 kN/m²	+0.084, -0.125 kN/m²	-0.719 kN/m²
F		-0.440 kN/m²	+0.084, -0.125 kN/m²	-0.524 kN/m²
G		-0.245 kN/m²	+0.084, -0.125 kN/m²	-0.328 kN/m²

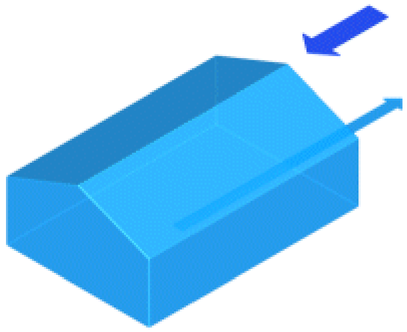


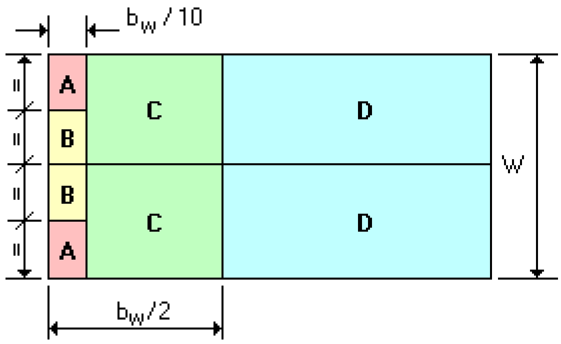


Width measured across wind, depth measured in wind direction

Roof results, wind normal to face 2

Zone	Zone width & depth	External applied loads	Internal applied loads	Overall applied loads
A	2.425 m, 0.970 m	-0.782 kN/m²	+0.084, -0.125 kN/m²	-0.866 kN/m²
B	2.425 m, 0.970 m	-0.734 kN/m²	+0.084, -0.125 kN/m²	-0.817 kN/m²
C	4.850 m, 3.880 m	-0.293 kN/m²	+0.084, -0.125 kN/m²	-0.377 kN/m²
D		-0.196 kN/m²	+0.084, -0.125 kN/m²	-0.279 kN/m²

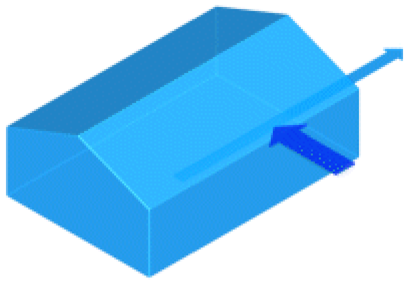


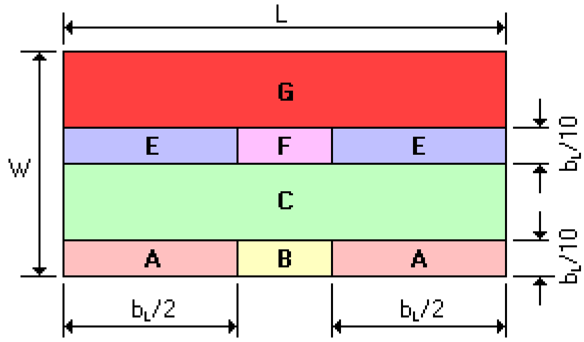


Width measured across wind, depth measured in wind direction

Roof results, wind normal to face 3

Zone	Zone width & depth	External applied loads	Internal applied loads	Overall applied loads
A	15.000 m, 3.000 m	+0.098, -0.538 kN/m²	+0.084, -0.125 kN/m²	+0.223, -0.622 kN/m²
B	0.000 m, 3.000 m	+0.098, -0.391 kN/m²	+0.084, -0.125 kN/m²	+0.223, -0.475 kN/m²
C		+0.098, -0.196 kN/m²	+0.084, -0.125 kN/m²	+0.223, -0.279 kN/m²
E		-0.636 kN/m²	+0.084, -0.125 kN/m²	-0.719 kN/m²
F		-0.440 kN/m²	+0.084, -0.125 kN/m²	-0.524 kN/m²
G		-0.245 kN/m²	+0.084, -0.125 kN/m²	-0.328 kN/m²

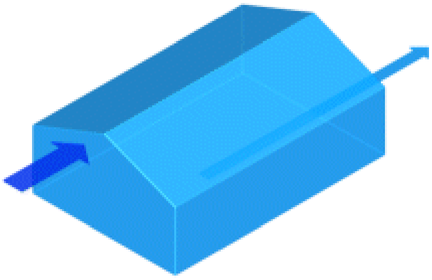


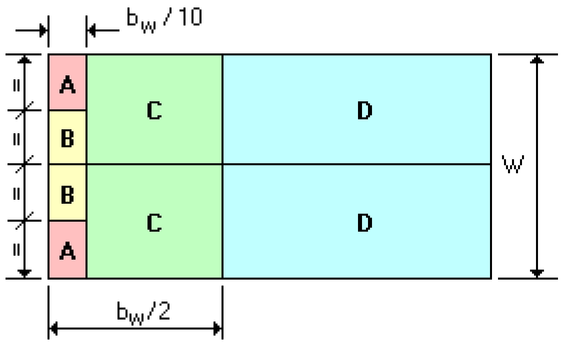


Width measured across wind, depth measured in wind direction

Roof results, wind normal to face 4

Zone	Zone width & depth	External applied loads	Internal applied loads	Overall applied loads
A	2.425 m, 0.970 m	-0.782 kN/m²	+0.084, -0.125 kN/m²	-0.866 kN/m²
B	2.425 m, 0.970 m	-0.734 kN/m²	+0.084, -0.125 kN/m²	-0.817 kN/m²
C	4.850 m, 3.880 m	-0.293 kN/m²	+0.084, -0.125 kN/m²	-0.377 kN/m²
D		-0.196 kN/m²	+0.084, -0.125 kN/m²	-0.279 kN/m²

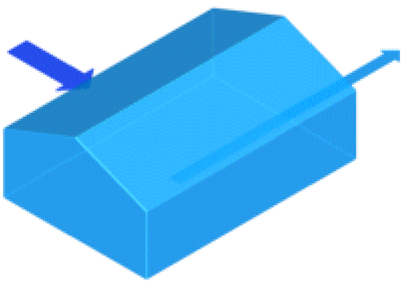


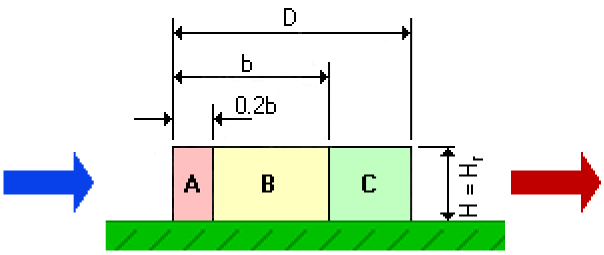


Width measured across wind, depth measured in wind direction

Wall results, wind loads on face 1

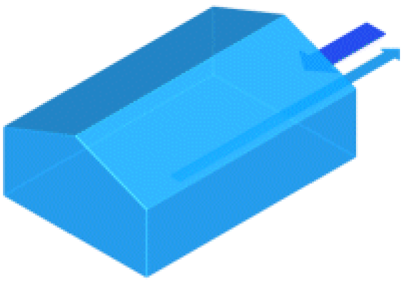
Zone	Zone length	External applied loads	Internal applied loads	Overall applied loads
Windward	30.000 m	+0.406 kN/m²	+0.084, -0.125 kN/m²	+0.531 kN/m²
Leeward	30.000 m	-0.239 kN/m²	+0.084, -0.125 kN/m²	-0.322 kN/m²
A (face 2)	1.940 m	-0.620 kN/m²	+0.084, -0.125 kN/m²	-0.704 kN/m²
B (face 2)	7.760 m	-0.382 kN/m²	+0.084, -0.125 kN/m²	-0.465 kN/m²
C (face 2)	20.300 m	-0.239 kN/m²	+0.084, -0.125 kN/m²	-0.322 kN/m²
A (face 4)	1.940 m	-0.620 kN/m²	+0.084, -0.125 kN/m²	-0.704 kN/m²
B (face 4)	7.760 m	-0.382 kN/m²	+0.084, -0.125 kN/m²	-0.465 kN/m²
C (face 4)	20.300 m	-0.239 kN/m²	+0.084, -0.125 kN/m²	-0.322 kN/m²

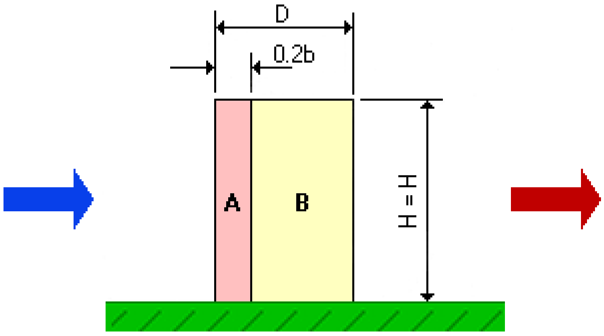




Wall results, wind loads on face 2

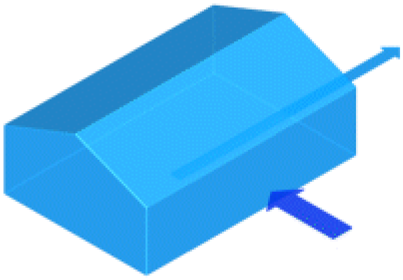
Zone	Zone length	External applied loads	Internal applied loads	Overall applied loads
Windward	9.700 m	+0.371 kN/m²	+0.084, -0.125 kN/m²	+0.497 kN/m²
Leeward	9.700 m	-0.242 kN/m²	+0.084, -0.125 kN/m²	-0.326 kN/m²
A (face 1)	6.000 m	-0.630 kN/m²	+0.084, -0.125 kN/m²	-0.713 kN/m²
B (face 1)	3.700 m	-0.388 kN/m²	+0.084, -0.125 kN/m²	-0.471 kN/m²
A (face 3)	6.000 m	-0.630 kN/m²	+0.084, -0.125 kN/m²	-0.713 kN/m²
B (face 3)	3.700 m	-0.388 kN/m²	+0.084, -0.125 kN/m²	-0.471 kN/m²

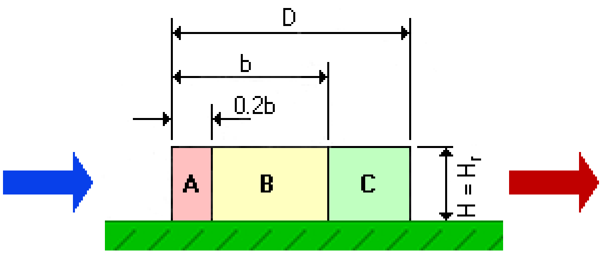




Wall results, wind loads on face 3

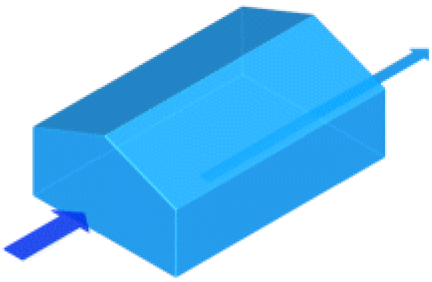
Zone	Zone length	External applied loads	Internal applied loads	Overall applied loads
Windward	30.000 m	+0.406 kN/m²	+0.084, -0.125 kN/m²	+0.531 kN/m²
Leeward	30.000 m	-0.239 kN/m²	+0.084, -0.125 kN/m²	-0.322 kN/m²
A (face 2)	1.940 m	-0.620 kN/m²	+0.084, -0.125 kN/m²	-0.704 kN/m²
B (face 2)	7.760 m	-0.382 kN/m²	+0.084, -0.125 kN/m²	-0.465 kN/m²
C (face 2)	20.300 m	-0.239 kN/m²	+0.084, -0.125 kN/m²	-0.322 kN/m²
A (face 4)	1.940 m	-0.620 kN/m²	+0.084, -0.125 kN/m²	-0.704 kN/m²
B (face 4)	7.760 m	-0.382 kN/m²	+0.084, -0.125 kN/m²	-0.465 kN/m²
C (face 4)	20.300 m	-0.239 kN/m²	+0.084, -0.125 kN/m²	-0.322 kN/m²

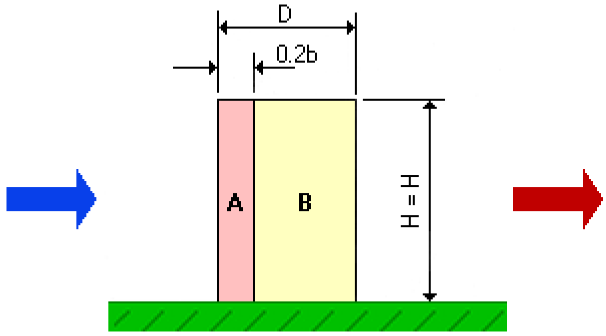




Wall results, wind loads on face 4

Zone	Zone length	External applied loads	Internal applied loads	Overall applied loads
Windward	9.700 m	+0.371 kN/m²	+0.084, -0.125 kN/m²	+0.497 kN/m²
Leeward	9.700 m	-0.242 kN/m²	+0.084, -0.125 kN/m²	-0.326 kN/m²
A (face 1)	6.000 m	-0.630 kN/m²	+0.084, -0.125 kN/m²	-0.713 kN/m²
B (face 1)	3.700 m	-0.388 kN/m²	+0.084, -0.125 kN/m²	-0.471 kN/m²
A (face 3)	6.000 m	-0.630 kN/m²	+0.084, -0.125 kN/m²	-0.713 kN/m²
B (face 3)	3.700 m	-0.388 kN/m²	+0.084, -0.125 kN/m²	-0.471 kN/m²





Imposed Load Roof

Live BS 6399-3 Clause 4.3.1 non trafficable 15 degree pitch
Q_k 0.6 = 0.60 kN/m^2

Snow Load S_b ground Figure 1 = 0.40 kN/m^2

For 0-100 metre elevation $S_o = S_b$ = 0.40 kN/m^2
$S_d = S_b \mu_i$
μ_I = 0.8[(60-α)/30] = 0.8[(60-15)/30] = 1.2
S_d = 0.4 x 0.67 = 0.48 kN/m$_2$ < Live

UNIT LOADS

All loads in kN/m2.

Floor Dead Loads:

$$DEAD\_LOADS\_F := \begin{pmatrix} \text{"22mm t\&g chipboard"} & 0.176 \\ \text{"36mm fibreboard silencio"} & 0.060 \\ \text{"22mm t\&g chipboard"} & 0.176 \\ \text{"Joists @ 400mm"} & 0.075 \\ \text{"insulation"} & 0.010 \\ \text{"additional\_1"} & 0 \\ \text{"additional\_2"} & 0 \\ \text{"addtional\_3"} & 0 \end{pmatrix}$$

Ceiling Loads:

$$DEAD\_LOADS\_C := \begin{pmatrix} \text{"fibreglass insulation"} & 0.10 \\ \text{"resillient bars"} & 0.05 \\ \text{"12.5mm firecheck board"} & 0.11 \\ \text{"12.5mm firecheck board"} & 0.11 \\ \text{"additional\_1"} & 0 \\ \text{"additional\_2"} & 0 \\ \text{"additional\_3"} & 0 \\ \text{"addtional\_4"} & 0 \end{pmatrix}$$

Roof Dead Loads (On Plan):

$$DEAD\_LOADS\_R := \begin{pmatrix} \text{"Roof Tiles"} & 0.060 \\ \text{"Roof Lining"} & 0.010 \\ \text{"Insulation"} & 0.010 \\ \text{"Steel Framing"} & 0.15 \\ \text{"additional\_1"} & 0 \\ \text{"additional\_2"} & 0 \\ \text{"additional\_3"} & 0 \\ \text{"addtional\_4"} & 0 \end{pmatrix}$$

Internal Wall Dead Loads:

$$DEAD\_LOADS\_IW := \begin{pmatrix} \text{"12.5mm firecheck board"} & 0.11 \\ \text{"12.5mm firecheck board"} & 0.11 \\ \text{"Steel Framing"} & 0.1 \\ \text{"Insulation"} & 0.010 \\ \text{"additional\_1"} & 0 \\ \text{"additional\_2"} & 0 \\ \text{"additional\_3"} & 0 \\ \text{"addtional\_4"} & 0 \end{pmatrix}$$

External Wall Dead Loads:

$$DEAD\_LOADS\_EW := \begin{pmatrix} \text{"12.5mm firecheck board"} & 0.11 \\ \text{"12.5mm firecheck board"} & 0.11 \\ \text{"Steel Framing"} & 0.15 \\ \text{"Insulation"} & 0.010 \\ \text{"22mm bitrock"} & 0.12 \\ \text{"cladding system"} & 0.40 \\ \text{"additional\_1"} & 0 \\ \text{"addtional\_2"} & 0 \end{pmatrix}$$

Live Loads:

$$LIVE\_LOADS := \begin{pmatrix} \text{"Floor Live Loads"} & 1.5 \\ \text{"Partition Walls"} & 0.5 \\ \text{"Services"} & 0.15 \\ \text{"Imposed Roof Load"} & 0.60 \\ \text{"Snow"} & 0.48 \\ \text{"additional\_1"} & 0 \\ \text{"additional\_2"} & 0 \end{pmatrix}$$

$$WIND\_LOADS := \begin{pmatrix} \text{"Stud"} & 0.713 \\ \text{"Rafter"} & 0.23 \\ \text{"Rafter Suction"} & 0.87 \\ \text{"Overall Walls"} & 0.85 \\ \text{"additional\_1"} & 0 \\ \text{"additional\_2"} & 0 \end{pmatrix}$$

$E := 205000\,N \cdot mm^{-2}$

$p_y := 350 N \cdot mm^{-2}$

Z350 steel is used throughout

$DEAD\_LOADS\_F = 1.367\,kN\cdot m^{-2}$

$DEAD\_LOADS\_R = 0.23\,kN\cdot m^{-2}$

$DEAD\_LOADS\_EW = 0.9\,kN\cdot m^{-2}$

$LIVE\_LOAD\_F = 1.65\,kN\cdot m^{-2}$

$LIVE\_LOAD\_R = 0.60\,kN\cdot m^{-2}$

$DEAD\_LOADS\_C = 0.37\,kN\cdot m^{-2}$

$DEAD\_LOADS\_IW = 0.33\,kN\cdot m^{-2}$

$LIVE\_LOAD\_C = 0.15\,kN\cdot m^{-2}$

Joist Load Combinations:

$LCJ\_1 := 1.4DEAD\_LOADS\_F + 1.6\cdot LIVE\_LOAD\_F$

$LCJ\_2 := DEAD\_LOADS\_F + LIVE\_LOAD\_F$

$LCJ\_3 := LIVE\_LOAD\_F$

$LCJ\_4 := 1kN$

Design of Joist Type 1:

$Span\_j1 := 3.8m$ $\quad$ $Spacing\_j1 := 0.4m$ $\quad$ $Depth\_j1 := 300mm$

Design Moment: $M\_j1 := \frac{(LCJ\_1\cdot Spacing\_j1)\cdot Span\_j1^2}{8}$ $\qquad M\_j1 = 3.288\,kN\cdot m$

Design Shear: $V\_j1 := \frac{(LCJ\_1\cdot Spacing\_j1)\cdot Span\_j1}{2}$ $\qquad V\_j1 = 3.461\,kN$

Static Deflection Criteria:

Dead & Imposed check:

Maximum Allowable Deflection $\delta 1 := \frac{Span\_j1}{350}$

Minimum I required $I\_j1_{di} := \frac{5\cdot(LCJ\_2\cdot Spacing\_j1)\,Span\_j1^4}{384\cdot E\cdot \delta 1}$ $\qquad I\_j1_{di} = 1.472\times 10^6\,mm^4$

Imposed Check:

Maximum Allowable Deflection $\delta 2 := \frac{Span\_j1}{450}$

Minimum I required $I\_j1_i := \frac{5\cdot(LCJ\_3\cdot Spacing\_j1)\,Span\_j1^4}{384\cdot E\cdot \delta 2}$ $\qquad I\_j1_i = 1.035\times 10^6\,mm^4$

Dynamic Deflection Criteria:

Natural Frequency Check (8Hz Limit):

Maximum Allowable Deflection $\delta 3 := 5mm$

Minimum I required $I\_j1_{dy} := \frac{5\cdot\left(DEAD\_LOADS\_F + 0.3kN\cdot m^{-2}\right)\cdot Spacing\_j1\,Span\_j1^4}{384\cdot E\cdot \delta 3}$ $\qquad I\_j1_{dy} = 1.766\times 10^6\,mm^4$

1kN point load (apparent flexibility when walking):

Number of joists resisting the point load: $N_{eff} := \frac{1000mm}{Spacing\_j1}$

Serdar Dundar
20/08/2006
07866731125

Maximum deflection for perceptibility limit: $\delta 4 := 1.65\text{mm}$

Minimum I required: $I\_j1_p := \frac{LCJ\_4 \cdot Span\_j1^3}{48 \cdot E \cdot \delta 4} \cdot \frac{1}{N_{eff}}$ $\qquad I\_j1_p = 1.352 \times 10^6\,\text{mm}^4$

Overall Requirement for Deflection Criteria:

$I\_j1_{req} := \max\left(I\_j1_{di}, I\_j1_i, I\_j1_{dy}, I\_j1_p\right)$ **natural frequency governs** $\qquad \boxed{I\_j1_{req} = 1.8 \times 10^6\,\text{mm}^4}$

Strength Criteria :

Maximum Chord Forces: $Ax\_j1_{chord} := \frac{M\_j1}{Depth\_j1}$ $\qquad \boxed{Ax\_j1_{chord} = 11\,\text{kN}}$

Maximum Diagonal Forces: $Ax\_j1_{diag} := \frac{V\_j1}{\sin\left(\frac{65}{180}\pi\right)}$ $\qquad \boxed{Ax\_j1_{diag} = 3.819\,\text{kN}}$

(assuming the angle between last diagonal and the bottom chord is 65deg.)

Selected section for Type 1 Joists according to the section property tables provided by Scottsdale Steel: ***SCS_500_90_350_0.75mm***

Check the properties of the proposed truss against the deflection and strength criterias:

Plate thickness fixed on one side of the joists if exist: $tp\_j1 := 0\text{mm}$ Joist Thickness: $t\_j1 := 0.75\text{mm}$

Effective area of a single section under full compression: $A\_j1_{eff} := 93\text{mm}^2$

Gross area of a single section: $A\_j1_{gross} := 151\text{mm}^2$

Neutral Axis Location measured from the bottom of the joist:

$$NA\_j1 := \frac{A\_j1_{eff} \cdot (Depth\_j1) + A\_j1_{gross} \cdot 10\text{mm} + tp\_j1 \cdot Depth\_j1^2 \cdot 0.5}{A\_j1_{eff} + A\_j1_{gross} + tp\_j1 \cdot Depth\_j1}$$

$NA\_j1 = 120.5\,\text{mm}$

$Depth\_j1 - NA\_j1 = 179.5\,\text{mm}$

Equivalent moment of inertia of the joist:

$$I\_j1 := A\_j1_{eff} \cdot (Depth\_j1 - NA\_j1)^2 + A\_j1_{gross} \cdot NA\_j1^2 + tp\_j1 \cdot Depth\_j1 \cdot \left(\frac{Depth\_j1}{2} - NA\_j1\right)^2 + \frac{tp\_j1 \cdot Depth\_j1^3}{12}$$

Equivalent Section Modulus of the Joist: $\qquad \boxed{I\_j1 = 5189139.8\,\text{mm}^4}$

$W\_j1_{top} := \frac{I\_j1}{NA\_j1}$ $\qquad W\_j1_{bot} := \frac{I\_j1}{Depth\_j1 - NA\_j1}$

$\boxed{W\_j1_{top} = 4.31 \times 10^4\,\text{mm}^3}$

$\boxed{W\_j1_{bot} = 2.89 \times 10^4\,\text{mm}^3}$

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Check Against the requirements:

$$W\_j1_{req} := \frac{M\_j1}{p_y \cdot 0.9}$$ $$W\_j1_{req} = 1.044 \times 10^4\, mm^3$$

$$I\_j1_{req} = 1.766 \times 10^6\, mm^4$$ $$I\_j1_{req} = 1.766 \times 10^6\, mm^4$$

If both ratios below are smaller then 1 the proposed joist is OK.

$$RW\_j1 := \frac{W\_j1_{req}}{\min(W\_j1_{top}, W\_j1_{bot})} \qquad RI\_j1 := \frac{I\_j1_{req}}{I\_j1}$$

$$RW\_j1 = 0.361$$

$$RI\_j1 = 0.34$$

Assuming that the top and bottom chord are fully restrained!

"Proposed Joist OK"

Amount of <u>total</u> screws or rivets required at each end of the last diagonals:

Max. shear one rivet can take according to BS5950-5:1998 Annex A.1.4.
Refer to the code for all the other required checks, only shear capacity in tilting and bearing is considered!!! The screws and rivets are assumed to be 4.8mm diameter

$$P\_j_{screw} := 3.2 \cdot \sqrt{t\_j1^3 \cdot 4.8mm} \cdot p_y$$ $$P\_j_{screw} = 1.594\, kN$$

$$N\_j1_{screw} := \frac{Ax\_j1_{diag}}{P\_j_{screw}}$$ $$\boxed{N\_j1_{screw} = 2.4}$$

- Round up the required screws to the nearest even number and distribute the screws to either flange at either end equally.
- Check rest of the diagonals and determine the additional screws needed for the others. The number of additional screws will decrease as the diagonal is closer to the middle of the joist
- There will be a need for gusset plates if the additonal number of screws per flange is more then 2. Refer to structural details provided by the str. eng.

<u>Rafter Load Combinations:</u>

$$LCR\_1 := 1.4DEAD\_LOADS\_R + 1.6 \cdot LIVE\_LOAD\_R$$

$$LCR\_2 := 1.4 \cdot DEAD\_LOADS\_R + 1.4 \cdot WIND\_R$$

$$LCR\_3 := 1.2 \cdot (DEAD\_LOADS\_R + LIVE\_LOAD\_R + WIND\_R)$$

$$LCR\_4 := 1.0 \cdot DEAD\_LOADS\_R - 1.4 \cdot WIND\_RS$$

$$LCR\_5 := DEAD\_LOADS\_R + LIVE\_LOAD\_R$$

$$LCR\_6 := LIVE\_LOAD\_R$$

$$LCR\_7 := 1kN$$

Design of Rafter Type 1:

$Span\_r1 := 4.85m$ $\quad Spacing\_r1 := 0.4m$ $\quad slope := 15deg$

Governing Load Combination: $LC\_R := \max(LCR\_1, LCR\_2, LCR\_3)$ $\qquad LC\_R = 1.282\, kN \cdot m^{-2}$

Design Moment:

$$M\_r1 := \frac{\left(LC\_R \cdot Spacing\_r1 \cdot \cos(slope)^{-1}\right) \cdot Span\_r1^2}{8}$$ $$M\_r1 = 1.561\, kN \cdot m$$

Design Shear:

$$V\_r1 := \frac{(LC\_R \cdot Spacing\_r1) \cdot Span\_r1 \cdot \cos(slope)^{-1}}{2}$$ $$V\_r1 = 1.287\, kN$$

Serdar Dundar
20/08/2006
07866731125

Section Capacities of the proposed section: ***SCS_500_140_350_1.2mm @ 400 centers***

Design Capacities			
ΦMu Bending Capacity	kN-m	3.699300	**Distortional Buckling Governs**
ΦVvy Shear Capacity	kN	11.251200	
ΦRby Bearing Capacity	kN	2.847970	**Unstiffened End Bearing**

The section is assumed to be fully restraint by the structural board over the top of the section.
The section is assumed to be fully restrained by the noggings under the wind suction loading.

Effective moment of inertia of a single section : $I\_r1_{eff} := 831211\text{mm}^4$

Effective elastic section modulus of a single section: $W\_r1_{eff} := 11125\text{mm}^3$

Provide nogs at 1200mm centers

$$W\_r1_{req} := \frac{M\_r1}{p_y \cdot 0.9}$$ $\boxed{W\_r1_{req} = 4955\,\text{mm}^3}$

Static Deflection Criteria:

Dead & Imposed check:

Maximum Allowable Deflection $\delta1 := \frac{Span\_r1}{240}$

Minimum I required $I\_r1_{di} := \frac{5 \cdot \left(LCR\_5 \cdot Spacing\_r1 \cdot \cos(slope)^{-1}\right) Span\_r1^4}{384 \cdot E \cdot \delta1}$ $I\_r1_{di} = 5.977 \times 10^5\,\text{mm}^4$

Imposed Check:

Maximum Allowable Deflection $\delta2 := \frac{Span\_r1}{300}$

Minimum I required $I\_r1_i := \frac{5 \cdot \left(LCR\_6 \cdot Spacing\_r1 \cdot \cos(slope)^{-1}\right) Span\_r1^4}{384 \cdot E \cdot \delta2}$ $I\_r1_i = 5.401 \times 10^5\,\text{mm}^4$

1kN point load (apparent flexibility when walking):

Number of joists resisting the point load: $N_{eff} := \frac{1000\text{mm}}{Spacing\_r1}$

Maximum deflection for perceptibility limit: $\delta4 := 35\text{mm}$

Minimum I required: $I\_r1_p := \frac{LCR\_7 \cdot \cos(slope)^{-1} \cdot Span\_r1^3}{48 \cdot E \cdot \delta4} \cdot \frac{1}{N_{eff}}$ $I\_r1_p = 1.372 \times 10^5\,\text{mm}^4$

Overall Requirement for Deflection Criteria:

$I\_r1_{req} := \max\left(I\_r1_{di}, I\_r1_i, I\_r1_p\right)$ **dead load+imposed governs** $\boxed{I\_r1_{req} = 6 \times 10^5\,\text{mm}^4}$

If both ratios below are smaller then 1 the proposed joist is OK.

$RW\_j1 := \frac{W\_r1_{req}}{W\_r1_{eff}}$ $\quad$ $RI\_j1 := \frac{I\_r1_{req}}{I\_r1_{eff}}$ $\quad$ $RW\_j1 = 0.445$

$RI\_j1 = 0.719$

"Proposed Rafter OK"

Check Connection Requirements:

Uplift Force per rafter at each end:

$$F\_r1_{uplift} := \frac{(LCR\_4 \cdot Spacing\_r1) \cdot Span\_r1 \cdot \cos(slope)^{-1}}{2}$$

End Connections of every rafter need to be designed to take these loads:

$\boxed{F\_r1_{uplift} = -0.992\,kN}$ UPLIFT FORCE

$\boxed{V\_r1 = 1.287\,kN}$ SHEAR FORCE

<u>Ceiling Load Combinations:</u>

$LCC\_1 := 1.4 DEAD\_LOADS\_C + 1.6 \cdot LIVE\_LOAD\_C$
$LCC\_2 := DEAD\_LOADS\_C + LIVE\_LOAD\_C$
$LCC\_3 := LIVE\_LOAD\_C$
$LCC\_4 := 1kN$

<u>Design of ceiling Type 1:</u>

$Span\_c1 := 3.5m$ $\quad$ $Spacing\_c1 := 0.4m$

Governing Load Combination: $LC\_C := \max(LCC\_1, LCC\_2, LCC\_3)$ $\quad$ $LC\_C = 0.758\,kN \cdot m^{-2}$

Design Moment: $M\_c1 := \frac{(LC\_C \cdot Spacing\_c1) \cdot Span\_c1^2}{8}$ $\quad$ $M\_c1 = 0.464\,kN \cdot m$

Design Shear: $V\_c1 := \frac{(LC\_C \cdot Spacing\_c1) \cdot Span\_c1}{2}$ $\quad$ $V\_c1 = 0.531\,kN$

Section Capacities of the proposed section: ***SCS_500_90_350_0.75mm @ 400 centers***

Design Capacities			
ΦMu Bending Capacity	kN-m	1.006000	**Distortional Buckling Governs**
ΦVvy Shear Capacity	kN	4.343000	
ΦRby Bearing Capacity	kN	0.798253	**Unstiffened End Bearing**

Effective moment of inertia of a single section : $I\_c1_{eff} := 183315mm^4$

Effective elastic section modulus of a single section: $W\_c1_{eff} := 3653mm^3$

$W\_c1_{req} := \frac{M\_c1}{p_y \cdot 0.9}$ $\quad$ $\boxed{W\_c1_{req} = 1474\,mm^3}$

<u>Provide nogs at 1200mm centers</u> The ceiling members are assumed to be fully restrained by the nogs.

Static Deflection Criteria:

Dead & Imposed check:

Maximum Allowable Deflection $\delta 1 := \frac{Span\_c1}{300}$

Minimum I required $I\_c1_{di} := \frac{5 \cdot (LCC\_2 \cdot Spacing\_c1) \, Span\_c1^4}{384 \cdot E \cdot \delta 1}$ $\quad I\_c1_{di} = 1.699 \times 10^5 \, mm^4$

Imposed Check:

Maximum Allowable Deflection $\delta 2 := \frac{Span\_c1}{450}$

Minimum I required $I\_c1_i := \frac{5 \cdot (LCC\_3 \cdot Spacing\_c1) \, Span\_c1^4}{384 \cdot E \cdot \delta 2}$ $\quad I\_c1_i = 7.353 \times 10^4 \, mm^4$

1kN point load (apparent flexibility when walking):

Number of joists resisting the point load: $N_{eff} := \frac{1000mm}{Spacing\_c1}$

Maximum deflection for perceptibility limit: $\delta 4 := 35mm$

Minimum I required: $I\_c1_p := \frac{LCC\_4 \cdot Span\_c1^3}{48 \cdot E \cdot \delta 4} \cdot \frac{1}{N_{eff}}$ $\quad I\_c1_p = 4.98 \times 10^4 \, mm^4$

Overall Requirement for Deflection Criteria:

$I\_c1_{req} := \max\left(I\_c1_{di}, I\_c1_i, I\_c1_p\right)$ **dead load+imposed governs** $\boxed{I\_c1_{req} = 1.7 \times 10^5 \, mm^4}$

If both ratios below are smaller then 1 the proposed joist is OK.

$RW\_c1 := \frac{W\_c1_{req}}{W\_c1_{eff}}$ $\quad RI\_c1 := \frac{I\_c1_{req}}{I\_c1_{eff}}$

$RW\_c1 = 0.403$

$RI\_c1 = 0.927$

"Proposed Ceiling Joist OK"

WALL STUD CALCULATIONS:

LOAD PATH_1 THROUGH TYPE 1 EXTERNAL WALL PANELS

Stud Load Combinations:

Description: This load path is the side wall receiving joist type 1 at every floor level and rafter type1 at the roof level.

$$WIND\_S := WIND\_LOADS_{1,2} \cdot kN \cdot m^{-2}$$

$$LCS\_1 := 1.4DEAD\_LOADS\_S + 1.6 \cdot LIVE\_LOAD\_S$$
$$LCS\_2 := 1.4 \cdot DEAD\_LOADS\_S + 1.4 \cdot WIND\_S$$
$$LCS\_3 := 1.2 \cdot (DEAD\_LOADS\_S + LIVE\_LOAD\_S + WIND\_S)$$
$$LCS\_4 := 1.0DL + 1.4WIND\_S$$
$$LCS\_5 := WIND\_S$$

***Note :** The vertical load exterted on the walls due to the wind load on the roof slope is not taken into account in the below calculations. The wind load is only taken into account from bending point of view. It can be shown that the -espcially for a 4 storey building-, the effect of the wind in vertical direction on the studs is considerably lower than the effect of live load and dead load and hence it is not taken into account, the roof uplift can not govern the design of wall studs by inspection hence the uplift is only considered in the design of rafters. However while the overall stability of the building is considered , additional studs are provided to take the vertical reaction at the ends of the braces -which are not taken into account in the stud design calculations- these also will add to the overall vertical load bearing capacity of the walls.*

Wall Heights and stud spacings at each floor level through this load path:

Walls_E :=

"storey"	"Height"	"Stud Spacing"
4	2.7	600
3	2.7	600
2	2.7	400
1	2.7	400

Instead of B2B sections input the spacing as half of the real spacing. (input 200mm centers instead of 400 B2B sections)

Section properties of the proposed wall stud section at each floor through this load path:

Fill the matrix below according to the section property tables provided by Scottsdale.

Section_LP1 :=

"storey"	"Stud_depth"	"Thickness"	"Axial"	"Moment"	"Ieff"	"Weff"	"Nog_centers"
4	140	0.75	17.0	1.62	466799	5707	1350
3	140	1.0	24.0	2.43	671801	8757	1350
2	140	1.0	24.0	2.43	671801	8757	1350
1	140	1.2	32.0	3.70	831211	11125	1350

Loads transferred from roof:

Dead Loads from rafter:

$$R\_r1_{DL} := \left| \begin{array}{l} R\_r_{DL_1} \leftarrow 0 \\ \text{for } i \in 2..\text{rows(Walls\_E)} \\ \quad R\_r_{DL_i} \leftarrow \dfrac{(\text{DEAD\_LOADS\_R})\cdot \text{Span\_r1}\cdot \cos(\text{slope})^{-1}}{2} \\ \text{submatrix}\left(R\_r_{DL}, 2, \text{rows(Walls\_E)}, 1, 1\right) \end{array} \right.$$

$$R\_r1_{DL} = \begin{pmatrix} 0.577 \\ 0.577 \\ 0.577 \\ 0.577 \end{pmatrix} \frac{1}{m} kN$$

Live Loads from rafter:

$$R\_r1_{LL} := \left| \begin{array}{l} R\_r_{LL_1} \leftarrow 0 \\ \text{for } i \in 2..\text{rows(Walls\_E)} \\ \quad R\_r_{LL_i} \leftarrow \dfrac{(\text{LIVE\_LOAD\_R})\cdot \text{Span\_r1}\cdot \cos(\text{slope})^{-1}}{2} \\ \text{submatrix}\left(R\_r_{LL}, 2, \text{rows(Walls\_E)}, 1, 1\right) \end{array} \right.$$

$$R\_r1_{LL} = \begin{pmatrix} 1.506 \\ 1.506 \\ 1.506 \\ 1.506 \end{pmatrix} \frac{1}{m} kN$$

Loads Transferred from ceiling:

Dead Loads from ceiling:

$$R\_c1_{DL} := \left| \begin{array}{l} R\_c_{DL_1} \leftarrow 0 \\ \text{for } i \in 2..\text{rows(Walls\_E)} \\ \quad R\_c_{DL_i} \leftarrow \dfrac{\text{DEAD\_LOADS\_C}\cdot \text{Span\_c1}}{2} \\ \text{submatrix}\left(R\_c_{DL}, 2, \text{rows(Walls\_E)}, 1, 1\right) \end{array} \right.$$

$$R\_c1_{DL} = \begin{pmatrix} 0.647 \\ 0.647 \\ 0.647 \\ 0.647 \end{pmatrix} \frac{1}{m} kN$$

Live Loads from ceiling:

$$R\_c1_{LL} := \left| \begin{array}{l} R\_c_{LL_1} \leftarrow 0 \\ \text{for } i \in 2..\text{rows(Walls\_E)} \\ \quad R\_c_{LL_i} \leftarrow \dfrac{(\text{LIVE\_LOAD\_C})\cdot \text{Span\_r1}\cdot \cos(\text{slope})^{-1}}{2} \\ \text{submatrix}\left(R\_c_{LL}, 2, \text{rows(Walls\_E)}, 1, 1\right) \end{array} \right.$$

$$R\_c1_{LL} = \begin{pmatrix} 0.377 \\ 0.377 \\ 0.377 \\ 0.377 \end{pmatrix} \frac{1}{m} kN$$

Serdar Dundar
20/08/2006
07866731125

Dead load of the Walls_E:

$$R\_w_{DL} := \left| \begin{array}{l} R\_w_{DL_1} \leftarrow 0 \\ \text{for } i \in 2..\text{rows}(Walls\_E) \\ \quad R\_w_{DL_i} \leftarrow Walls\_E_{i,2} \cdot DEAD\_LOADS\_EW \cdot m \\ \text{submatrix}(R\_w_{DL}, 2, \text{rows}(Walls\_E), 1, 1) \end{array} \right.$$

$$R\_w_{DL} = \begin{pmatrix} 2.43 \\ 2.43 \\ 2.43 \\ 2.43 \end{pmatrix} \frac{1}{m} kN$$

Dead Loads Transferred from Joists:

$$R\_j1_{DL} := \left| \begin{array}{l} R\_j1_{DL_1} \leftarrow 0 \\ \text{for } i \in 2..\text{rows}(Walls\_E) \\ \quad \left| \begin{array}{l} R\_j1_{DL_i} \leftarrow \dfrac{(DEAD\_LOADS\_F) \cdot Span\_j1}{2} \\ R\_j1_{DL_2} \leftarrow 0 \end{array} \right. \\ \text{submatrix}(R\_j1_{DL}, 2, \text{rows}(Walls\_E), 1, 1) \end{array} \right.$$

$$R\_j1_{DL} = \begin{pmatrix} 0 \\ 2.6 \\ 2.6 \\ 2.6 \end{pmatrix} \frac{1}{m} kN$$

Live Loads Transferred from Joists:

$$R\_j1_{LL} := \left| \begin{array}{l} R\_j1_{LL_1} \leftarrow 0 \\ \text{for } i \in 2..\text{rows}(Walls\_E) \\ \quad \left| \begin{array}{l} R\_j1_{LL_i} \leftarrow \dfrac{(LIVE\_LOAD\_F) \cdot Span\_j1}{2} \\ R\_j1_{LL_2} \leftarrow 0 \end{array} \right. \\ \text{submatrix}(R\_j1_{LL}, 2, \text{rows}(Walls\_E), 1, 1) \end{array} \right.$$

$$R\_j1_{LL} = \begin{pmatrix} 0 \\ 3.13 \\ 3.13 \\ 3.13 \end{pmatrix} \frac{1}{m} kN$$

Total Vertical Loads on Walls_E from top to bottom:

$$Vertical\_LP1 := \left| \begin{array}{l} LP1_{1,1} \leftarrow (\text{rows}(Walls\_E) - 1) \cdot kN \cdot m^{-1} \\ LP1_{1,2} \leftarrow R\_r1_{DL_1} + R\_c1_{DL_1} + R\_w_{DL_1} \\ LP1_{1,3} \leftarrow R\_r1_{LL_1} + R\_c1_{LL_1} \\ \text{for } i \in 2..\text{rows}(Walls\_E) - 1 \\ \quad \left| \begin{array}{l} LP1_{i,2} \leftarrow R\_w_{DL_i} + R\_j1_{DL_i} + LP1_{i-1,2} \\ LP1_{i,3} \leftarrow R\_j1_{LL_i} + LP1_{i-1,3} \\ LP1_{i,1} \leftarrow (\text{rows}(Walls\_E) - i) \cdot kN \cdot m^{-1} \end{array} \right. \\ LP1 \end{array} \right.$$

Serdar Dundar
20/08/2006
07866731125

Unfactored Vertical loads on every floor level per stud along load path 1:

$$\begin{array}{cccc} & & \text{DEAD} & \text{LIVE} \\ \text{Vertical\_LP1} = & \begin{pmatrix} 4 & 3.65 & 1.88 \\ 3 & 8.68 & 5.02 \\ 2 & 13.71 & 8.15 \\ 1 & 18.74 & 11.29 \end{pmatrix} & \frac{1}{m} kN & \end{array}$$

Unfactored moments acting on the wall studs per level due to wind loads:

$$R\_w_{WL} := \left| \begin{array}{l} R\_w_{WL_1} \leftarrow 0 \\ \text{for } i \in 2 .. \text{rows}(Walls\_E) \\ \quad R\_w_{WL_i} \leftarrow \dfrac{\left(Walls\_E_{i,2} \cdot m\right)^2 \cdot WIND\_S}{8} \cdot \left(Walls\_E_{i,3} \cdot mm\right) \\ \text{submatrix}\left(R\_w_{WL}, 2, \text{rows}(Walls\_E), 1, 1\right) \end{array} \right.$$

WIND

$$R\_w_{WL} = \begin{pmatrix} 0.39 \\ 0.39 \\ 0.26 \\ 0.26 \end{pmatrix} kN \cdot m$$

Assume all the studs are taking the same wind load, to be on the conservative side use the max. local pressure on the whole building.

$$V\_LOADS\_LP1 := \left| \begin{array}{l} \text{for } i \in 1 .. \text{rows}(Vertical\_LP1) \\ \quad \left| \begin{array}{l} V_{i,1} \leftarrow Walls\_E_{i+1,3} \cdot mm \cdot \left(1.4 \cdot Vertical\_LP1_{i,2} + 1.6 \cdot Vertical\_LP1_{i,3}\right) \\ V_{i,2} \leftarrow Walls\_E_{i+1,3} \cdot mm \cdot 1.4 \cdot Vertical\_LP1_{i,2} \\ V_{i,3} \leftarrow Walls\_E_{i+1,3} \cdot mm \cdot \left(1.2 \cdot Vertical\_LP1_{i,2} + 1.2 \cdot Vertical\_LP1_{i,3}\right) \\ V_{i,4} \leftarrow Walls\_E_{i+1,3} \cdot mm \cdot 1.0 \cdot Vertical\_LP1_{i,2} \\ V_{i,5} \leftarrow 0 \cdot kN \end{array} \right. \\ V \end{array} \right.$$

Factored Vertical loads on every floor level per stud along load path 1 (each row corresponding to a load combination stated above) :

Every row is representing the floor level.
Every column is corresponding to a load combination.

$$V\_LOADS\_LP1 = \begin{array}{ccccc} \text{LC1} & \text{LC2} & \text{LC3} & \text{LC4} & \text{LC5} \\ \end{array}$$

$$V\_LOADS\_LP1 = \begin{pmatrix} 4.88 & 3.07 & 3.99 & 2.19 & 0.00 \\ 12.11 & 7.29 & 9.86 & 5.21 & 0.00 \\ 12.90 & 7.68 & 10.49 & 5.48 & 0.00 \\ 17.72 & 10.49 & 14.41 & 7.49 & 0.00 \end{pmatrix} kN$$

$$H\_LOADS\_LP1 := \left| \begin{array}{l} \text{for } i \in 1 .. \text{rows}(R\_w_{WL}) \\ \quad \left| \begin{array}{l} H_{i,1} \leftarrow 0 \\ H_{i,2} \leftarrow 1.4 \cdot R\_w_{WL_i} \\ H_{i,3} \leftarrow 1.2 \cdot R\_w_{WL_i} \\ H_{i,4} \leftarrow 1.4 \cdot R\_w_{WL_i} \\ H_{i,5} \leftarrow R\_w_{WL_i} \end{array} \right. \\ H \end{array} \right.$$

$$\begin{array}{cccccc} & \text{LC1} & \text{LC2} & \text{LC3} & \text{LC4} & \text{LC5} \end{array}$$

$$H\_LOADS\_LP1 = \begin{pmatrix} 0.00 & 0.55 & 0.47 & 0.55 & 0.39 \\ 0.00 & 0.55 & 0.47 & 0.55 & 0.39 \\ 0.00 & 0.36 & 0.31 & 0.36 & 0.26 \\ 0.00 & 0.36 & 0.31 & 0.36 & 0.26 \end{pmatrix} kN \cdot m$$

Stud Capacity Checks:

$$Check\_LP1 := \left| \begin{array}{l} \text{for } i \in 1 .. \text{rows}(Walls\_E) - 1 \\ \quad \text{for } j \in 1 .. \text{cols}(H\_LOADS\_LP1) \\ \quad\quad \left| \begin{array}{l} M\_all \leftarrow Section\_LP1_{i+1,5} \cdot kN \cdot m \\ A\_all \leftarrow Section\_LP1_{i+1,4} \cdot kN \\ M\_appl \leftarrow H\_LOADS\_LP1_{i,j} \\ A\_appl \leftarrow V\_LOADS\_LP1_{i,j} \\ Ratio_{i,j} \leftarrow \left(\dfrac{M\_appl}{M\_all} \right) + \left(\dfrac{A\_appl}{A\_all} \right) \end{array} \right. \\ Ratio \end{array} \right.$$

$$\begin{array}{cccccc} & \text{LC1} & \text{LC2} & \text{LC3} & \text{LC4} & \text{LC5} \end{array}$$

$$Check\_LP1 = \begin{pmatrix} 0.29 & 0.52 & 0.52 & 0.47 & 0.24 \\ 0.5 & 0.53 & 0.6 & 0.44 & 0.16 \\ 0.54 & 0.47 & 0.57 & 0.38 & 0.11 \\ 0.55 & 0.43 & 0.53 & 0.33 & 0.07 \end{pmatrix}$$

LOAD PATH_2 THROUGH TYPE 2 INTERNAL WALL PANELS

Stud Load Combinations:

$WIND\_S := WIND\_LOADS_{1,2} \cdot kN \cdot m^{-2}$

$LCS\_1 := 1.4DEAD\_LOADS\_S + 1.6 \cdot LIVE\_LOAD\_S$

Description: This load path is an internal wall receiving joist type 1 on one side and joist type 2 on the other side. And this wall type is also receivign the ridge support inside the roof space along its length.

$Span\_j2 := 3.8m$

Wall Heights and stud spacings at each floor level through this load path:

$$Walls\_I := \begin{pmatrix} \text{"storey"} & \text{"Height"} & \text{"Stud Spacing"} \\ 4 & 2.7 & 600 \\ 3 & 2.7 & 400 \\ 2 & 2.7 & 300 \\ 1 & 2.7 & 200 \end{pmatrix}$$

Only the first load combination is going to be analysed for the internal Walls.

Instead of B2B sections input the spacing as half of the real spacing. (input 200mm centers instead of 400 B2B sections)

Section properties of the proposed wall stud section at each floor through this load path:

Fill the matrix below according to the section property tables provided by Scottsdale.

$$Section\_LP2 := \begin{pmatrix} \text{"storey"} & \text{"Stud\_depth"} & \text{"Thickness"} & \text{"Axial"} & \text{"Moment"} & \text{"Ieff"} & \text{"Weff"} & \text{"Nog\_centers"} \\ 4 & 90 & 0.75 & 12.5 & 1.00 & 183315 & 3654 & 1350 \\ 3 & 90 & 1.0 & 18 & 1.49 & 247995 & 5009 & 1350 \\ 2 & 90 & 1.0 & 18 & 1.49 & 671801 & 8757 & 1350 \\ 1 & 90 & 1.0 & 18 & 1.49 & 831211 & 11125 & 1350 \end{pmatrix}$$

Loads transferred from roof:

Dead Loads from rafter:

$$R\_r1_{DL} := \left| \begin{array}{l} R\_r_{DL_1} \leftarrow 0 \\ \text{for } i \in 2 .. rows(Walls\_I) \\ \quad R\_r_{DL_i} \leftarrow 2 \cdot \dfrac{(DEAD\_LOADS\_R) \cdot Span\_r1 \cdot \cos(slope)^{-1}}{2} \\ submatrix(R\_r_{DL}, 2, rows(Walls\_I), 1, 1) \end{array} \right.$$

$$R\_r1_{DL} = \begin{pmatrix} 1.155 \\ 1.155 \\ 1.155 \\ 1.155 \end{pmatrix} \frac{1}{m} kN$$

Live Loads from rafter:

$$R\_r1_{LL} := \left| \begin{array}{l} R\_r_{LL_1} \leftarrow 0 \\ \text{for } i \in 2..\text{rows}(Walls\_I) \\ \quad R\_r_{LL_i} \leftarrow 2\dfrac{(LIVE\_LOAD\_R)\cdot Span\_r1\cdot \cos(slope)^{-1}}{2} \\ \text{submatrix}(R\_r_{LL}, 2, \text{rows}(Walls\_I), 1, 1) \end{array} \right.$$

$$R\_r1_{LL} = \begin{pmatrix} 3.013 \\ 3.013 \\ 3.013 \\ 3.013 \end{pmatrix} \frac{1}{m} kN$$

Loads Transferred from ceiling:

Dead Loads from ceiling:

$$R\_c1_{DL} := \left| \begin{array}{l} R\_c_{DL_1} \leftarrow 0 \\ \text{for } i \in 2..\text{rows}(Walls\_I) \\ \quad R\_c_{DL_i} \leftarrow 2\cdot\dfrac{DEAD\_LOADS\_C\cdot Span\_c1}{2} \\ \text{submatrix}(R\_c_{DL}, 2, \text{rows}(Walls\_I), 1, 1) \end{array} \right.$$

$$R\_c1_{DL} = \begin{pmatrix} 1.295 \\ 1.295 \\ 1.295 \\ 1.295 \end{pmatrix} \frac{1}{m} kN$$

Live Loads from ceiling:

$$R\_c1_{LL} := \left| \begin{array}{l} R\_c_{LL_1} \leftarrow 0 \\ \text{for } i \in 2..\text{rows}(Walls\_I) \\ \quad R\_c_{LL_i} \leftarrow 2\cdot\dfrac{(LIVE\_LOAD\_C)\cdot Span\_r1\cdot \cos(slope)^{-1}}{2} \\ \text{submatrix}(R\_c_{LL}, 2, \text{rows}(Walls\_I), 1, 1) \end{array} \right.$$

$$R\_c1_{LL} = \begin{pmatrix} 0.753 \\ 0.753 \\ 0.753 \\ 0.753 \end{pmatrix} \frac{1}{m} kN$$

Dead load of the Walls_I:

$$R\_w_{DL} := \left| \begin{array}{l} R\_w_{DL_1} \leftarrow 0 \\ \text{for } i \in 2..\text{rows}(Walls\_I) \\ \quad R\_w_{DL_i} \leftarrow Walls\_I_{i,2}\cdot DEAD\_LOADS\_IW\cdot m \\ \text{submatrix}(R\_w_{DL}, 2, \text{rows}(Walls\_I), 1, 1) \end{array} \right.$$

$$R\_w_{DL} = \begin{pmatrix} 0.89 \\ 0.89 \\ 0.89 \\ 0.89 \end{pmatrix} \frac{1}{m} kN$$

Serdar Dundar
20/08/2006
07866731125

Dead Loads Transferred from Joists:

$$R\_j1_{DL} := \begin{array}{|l} R\_j1_{DL_1} \leftarrow 0 \\ \text{for } i \in 2..\text{rows}(Walls\_I) \\ \quad \begin{array}{|l} R\_j1_{DL_i} \leftarrow \frac{(DEAD\_LOADS\_F)\cdot Span\_j1}{2}\cdot\left(1 + \frac{Span\_j2}{Span\_j1}\right) \\ R\_j1_{DL_2} \leftarrow 0 \end{array} \\ \text{submatrix}\left(R\_j1_{DL}, 2, \text{rows}(Walls\_I), 1, 1\right) \end{array}$$

$$R\_j1_{DL} = \begin{pmatrix} 0 \\ 5.19 \\ 5.19 \\ 5.19 \end{pmatrix} \frac{1}{m} kN$$

Live Loads Transferred from Joists:

$$R\_j1_{LL} := \begin{array}{|l} R\_j1_{LL_1} \leftarrow 0 \\ \text{for } i \in 2..\text{rows}(Walls\_I) \\ \quad \begin{array}{|l} R\_j1_{LL_i} \leftarrow \frac{(LIVE\_LOAD\_F)\cdot Span\_j1}{2}\cdot\left(1 + \frac{Span\_j2}{Span\_j1}\right) \\ R\_j1_{LL_2} \leftarrow 0 \end{array} \\ \text{submatrix}\left(R\_j1_{LL}, 2, \text{rows}(Walls\_I), 1, 1\right) \end{array}$$

$$R\_j1_{LL} = \begin{pmatrix} 0 \\ 6.27 \\ 6.27 \\ 6.27 \end{pmatrix} \frac{1}{m} kN$$

Total Vertical Loads on Walls from top to bottom:

$$Vertical\_LP2 := \begin{array}{|l} LP2_{1,1} \leftarrow (\text{rows}(Walls\_I) - 1)\cdot kN\cdot m^{-1} \\ LP2_{1,2} \leftarrow R\_r1_{DL_1} + R\_c1_{DL_1} + R\_w_{DL_1} \\ LP2_{1,3} \leftarrow R\_r1_{LL_1} + R\_c1_{LL_1} \\ \text{for } i \in 2..\text{rows}(Walls\_I) - 1 \\ \quad \begin{array}{|l} LP2_{i,2} \leftarrow R\_w_{DL_i} + R\_j1_{DL_i} + LP2_{i-1,2} \\ LP2_{i,3} \leftarrow R\_j1_{LL_i} + LP2_{i-1,3} \\ LP2_{i,1} \leftarrow (\text{rows}(Walls\_I) - i)\cdot kN\cdot m^{-1} \end{array} \\ LP2 \end{array}$$

Unfactored Vertical loads on every floor level per stud along load path 1:

$$
\text{Vertical\_LP2} = \begin{array}{cccc} & & \text{DEAD} & \text{LIVE} \\ \end{array}
\begin{pmatrix} 4 & 3.34 & 3.77 \\ 3 & 9.43 & 10.04 \\ 2 & 15.51 & 16.31 \\ 1 & 21.6 & 22.58 \end{pmatrix} \frac{1}{m} kN
$$

$$
\text{V\_LOADS\_LP2} := \left| \begin{array}{l} \text{for } i \in 1 .. \text{rows(Vertical\_LP2)} \\ \quad V_{i,1} \leftarrow \left(\text{Walls\_I}_{i+1,3} \cdot mm\right) \cdot \left(1.4 \cdot \text{Vertical\_LP2}_{i,2} + 1.6 \cdot \text{Vertical\_LP2}_{i,3}\right) \\ V \end{array} \right.
$$

Factored Vertical loads on every floor level per stud along load path 2 (each row corresponding to a load combination stated above :

Every row is representing the floor level.
Every column is corresponding to a load combination.

$$
\boxed{\text{V\_LOADS\_LP2} = \begin{array}{c} \text{LC1} \\ \begin{pmatrix} 6.42 \\ 11.70 \\ 14.34 \\ 13.27 \end{pmatrix} \end{array} kN}
$$

Stud Capacity Checks:

$$
\text{Check\_LP2} := \left| \begin{array}{l} \text{for } i \in 1 .. \text{rows(Walls\_I)} - 1 \\ \quad \left| \begin{array}{l} \text{A\_all} \leftarrow \text{Section\_LP2}_{i+1,4} \cdot kN \\ \text{A\_appl} \leftarrow \text{V\_LOADS\_LP2}_{i} \\ \text{Ratio}_{i} \leftarrow \dfrac{\text{A\_appl}}{\text{A\_all}} \end{array} \right. \\ \text{Ratio} \end{array} \right.
$$

$$
\boxed{\text{Check\_LP2} = \begin{array}{c} \text{LC1} \\ \begin{pmatrix} 0.51 \\ 0.65 \\ 0.80 \\ 0.74 \end{pmatrix} \end{array}}
$$

Serdar Dundar
20/08/2006
07866731125

Overall Stability of the Building / Wind Load Calculations:

Wind Load Distribution through the Floors: The gable end is parallel to the x axis!

Building length along x : $s_x := 9.7\text{m}$ $\alpha := \text{slope}$ Joist Depth : $j_d := 300\text{mm}$

Building Length along y: $s_y := 30\text{m}$

Wind Loads acting across the side wall (along X axis) $\text{Wind}_O := 1.4\cdot\text{WIND\_LOADS}_{4,2}\cdot\text{kN}\cdot\text{m}^{-2}$

$$\text{Wind\_X} := \left| \begin{array}{l} \text{Wind}_1 \leftarrow \left(s_y\cdot\frac{s_x}{2}\cdot\tan(\alpha)\cdot\text{Wind}_O\right) + \frac{\text{Walls\_E}_{2,2}\,\text{m} + j_d}{2}\cdot s_y\cdot\text{Wind}_O \\ \text{for } i \in 2 .. \text{rows}(\text{Walls\_E}) - 1 \\ \quad \text{Wind}_i \leftarrow \text{Wind}_{i-1} + \left[\left(\text{Walls\_E}_{i,2}\,\text{m} + j_d\right)\cdot s_y\cdot\text{Wind}_O\right] \\ \text{Wind} \end{array} \right.$$

$$\text{Wind\_X} = \begin{pmatrix} 100 \\ 207 \\ 314 \\ 421 \end{pmatrix}\text{kN}$$

Wind Loads acting across the gable wall (along Y axis)

$$\text{Wind\_Y} := \left| \begin{array}{l} \text{Wind}_1 \leftarrow \left(s_x\cdot\frac{s_x}{2}\cdot\tan(\alpha)\cdot\text{Wind}_O\right) + \frac{\text{Walls\_E}_{2,2}\,\text{m} + j_d}{2}\cdot s_x\cdot\text{Wind}_O \\ \text{for } i \in 2 .. \text{rows}(\text{Walls\_E}) - 1 \\ \quad \text{Wind}_i \leftarrow \text{Wind}_{i-1} + \left(\text{Walls\_E}_{i,2}\,\text{m} + j_d\right)\cdot s_x\cdot\text{Wind}_O \\ \text{Wind} \end{array} \right.$$

$$\text{Wind\_Y} = \begin{pmatrix} 32 \\ 67 \\ 102 \\ 136 \end{pmatrix}\text{kN}$$

Overall Stability of the Building / Bracing strength provided by the X braces:

Assumptions : The strap width is always 150mm.
The screw size to fix the straps and also the hold down anchors is always 4.8mm min.

Dead Load to take into account during the shear wall capacity calculations:

$$DL\_lateral_{LP1} := \left| \begin{array}{l} \text{for } i \in 1 .. \text{rows}(Vertical\_LP1) \\ \quad DL_i \leftarrow 0.9 \cdot Vertical\_LP1_{i,2} \\ DL \end{array} \right.$$

$$DL\_lateral_{LP2} := \left| \begin{array}{l} \text{for } i \in 1 .. \text{rows}(Vertical\_LP2) \\ \quad DL_i \leftarrow 0.9 \cdot Vertical\_LP2_{i,2} \\ DL \end{array} \right.$$

$$DL\_lateral_{LP1} = \begin{pmatrix} 3.289 \\ 7.814 \\ 12.339 \\ 16.863 \end{pmatrix} \frac{1}{m} kN$$

Use these values to fill in the DL column in the matrix below.
These are the unfactored dead load per floor as calculated above.
These are multiplied by 0.9 as the code states.

$$DL\_lateral_{LP2} = \begin{pmatrix} 3.007 \\ 8.484 \\ 13.961 \\ 19.438 \end{pmatrix} \frac{1}{m} kN$$

Function used to check strap bracing:

$$
F_{psw}(H, Lx, floor, Freq, type, t\_strap) := \left|
\begin{array}{l}
level \leftarrow rows(Section\_LP1) - floor \\
\text{if } type = \text{"E"} \\
\quad \left| \begin{array}{l}
stud\_ax \leftarrow Section\_LP1_{level,4} \cdot kN \\
t\_stud \leftarrow Section\_LP1_{level,3} \cdot mm \\
stud\_DL \leftarrow 0.9 \cdot Vertical\_LP1_{level-1,2} \cdot Walls\_E_{level,3} \cdot mm
\end{array} \right. \\
\text{if } type = \text{"I"} \\
\quad \left| \begin{array}{l}
stud\_ax \leftarrow Section\_LP2_{level,4} \\
t\_stud \leftarrow Section\_LP2_{level,3} \cdot mm \\
stud\_DL \leftarrow 0.9 \cdot Vertical\_LP2_{level-1,2} \cdot Walls\_I_{level,3} \cdot mm
\end{array} \right. \\
F_{diag} \leftarrow \dfrac{Freq}{\cos\left(\dfrac{H}{Lx} deg\right)} \\
A\_strap \leftarrow t\_strap \cdot 150mm \\
\Delta \leftarrow \dfrac{Freq \cdot \sqrt{H^2 + Lx^2}}{A\_strap \cdot E} \\
Strength \leftarrow A\_strap \cdot p_y \cdot 0.9 \\
\text{if } \left(\dfrac{\Delta}{H} < \dfrac{1}{500}\right) \cdot (Strength > Freq) = 1 \\
\quad \left| \begin{array}{l}
Ratio \leftarrow \dfrac{Freq}{Strength} \\
screws\_diag \leftarrow \dfrac{F_{diag}}{3.2 \cdot \sqrt{\min(t\_stud, t\_strap)^3 \cdot 4.8mm} \cdot p_y} \\
end\_stud \leftarrow \dfrac{Freq \cdot H}{Lx} \cdot kN^{-1} \\
anchor\_force \leftarrow end\_stud - stud\_DL \cdot kN^{-1} \\
\begin{pmatrix} Ratio \\ screws\_diag \\ end\_stud \\ anchor\_force \end{pmatrix}
\end{array} \right. \\
\begin{pmatrix} 0 \\ 0 \\ 0 \\ 0 \end{pmatrix} \quad \text{otherwise}
\end{array} \right.
$$

Serdar Dundar
20/08/2006
07866731125

Walls taking the lateral load acting along X direction in the ground floor:

$$W\_L1_x := \begin{pmatrix} \text{"H"} & \text{"L"} & \text{"No.X"} & \text{"Length X"} & \text{"F\_req\_per\_X"} & \text{"Floor"} & \text{"Int./Ext"} & \text{"Strap\_t"} \\ 2.7 & 9.7 & 3 & 1.6 & 18 & 0 & \text{"E"} & 1.2 \\ 2.7 & 9.7 & 3 & 2.0 & 18 & 0 & \text{"I"} & 1.2 \\ 2.7 & 9.7 & 3 & 2.0 & 18 & 0 & \text{"I"} & 1.2 \\ 2.7 & 5.3 & 2 & 1.6 & 18 & 0 & \text{"I"} & 1.2 \\ 2.7 & 4.4 & 2 & 1.6 & 18 & 0 & \text{"I"} & 1.2 \\ 2.7 & 5.3 & 2 & 1.6 & 18 & 0 & \text{"I"} & 1.2 \\ 2.7 & 9.7 & 3 & 2 & 18 & 0 & \text{"I"} & 1.2 \\ 2.7 & 9.7 & 3 & 2 & 18 & 0 & \text{"I"} & 1.2 \\ 2.7 & 9.7 & 3 & 1.6 & 18 & 0 & \text{"E"} & 1.2 \end{pmatrix}$$

$$R_{x_1} = \begin{pmatrix} 0.3 \\ 0.3 \\ 0.3 \\ 0.3 \\ 0.3 \\ 0.3 \\ 0.3 \\ 0.3 \\ 0.3 \end{pmatrix} \quad Screw\_strap_{x_1} = \begin{pmatrix} 5.6 \\ 7.3 \\ 7.3 \\ 7.3 \\ 7.3 \\ 7.3 \\ 7.3 \\ 7.3 \\ 5.6 \end{pmatrix} \quad End\_stud_{x_1} = \begin{pmatrix} 30.38 \\ 24.3 \\ 24.3 \\ 30.38 \\ 30.38 \\ 30.38 \\ 24.3 \\ 24.3 \\ 30.38 \end{pmatrix} \quad Anchor\_force_{x_1} = \begin{pmatrix} 23.63 \\ 20.41 \\ 20.41 \\ 26.49 \\ 26.49 \\ 26.49 \\ 20.41 \\ 20.41 \\ 23.63 \end{pmatrix}$$

R_{x_1} matrix gives the ratio of the force applied to the strap to the strength of strap. If the value is zero, it means either the force applied is greater than the strength or the deflection of the X braced portion is greater than 1/500.

$Screw\_strap_{x_1}$ matrix gives the required screws at the end of the straps. When calculating the screw strength the diameter is assumed to be 4.8mm. The thinner gauge is considered (of stud or strap) for pull-over strength which is assumed to be governing failure mode for the screws.

$End\_stud_{x_1}$ matrix gives the force at the ends of the strap caused by the lateral force only. It needs to be checked against the axial load that can be carried by each stud. Usually providing B2B studs at the ends of the straps gives the required strength.

$Anchor\_force_{x_1}$ matrix gives the required force to hold down the ends of strap. So the end studs need to be fixed to an anchor to resist these forces.

$$\boxed{\sum_{i=2}^{rows(W\_L1_x)} \left(W\_L1_{x_{i,3}} \cdot W\_L1_{x_{i,5}}\right) = 432}$$

Total Lateral Force carried by the X braces only.
The plasterboard and effect of the structural boards are not taken into account.

Serdar Dundar
20/08/2006
07866731125

The load need to be taken by the lateral bracing system is 421kN according to the above wind load calculations. Since the building is symmetrical there is no need to make calculations for torsional rigidity of the building. Assuming that the floor will act as a diaphram and distribute these loads to the walls according to their stiffenesses, adding the capacities of individual walls will be the overall strength of the building and can be compared against the total shear on the building.

Walls taking the lateral load acting along Y direction in the ground floor:

$$W\_L1_y := \begin{pmatrix} \text{"H"} & \text{"L"} & \text{"No.X"} & \text{"Length X"} & \text{"F\_req\_per\_X"} & \text{"Floor"} & \text{"Int./Ext"} & \text{"Strap\_t"} \\ 2.7 & 30 & 5 & 2.0 & 15 & 0 & \text{"E"} & 1.2 \\ 2.7 & 30 & 5 & 2.0 & 15 & 0 & \text{"E"} & 1.2 \end{pmatrix}$$

$$R_{y_1} = \begin{pmatrix} 0.3 \\ 0.3 \end{pmatrix} \quad Screw\_strap_{y_1} = \begin{pmatrix} 4.7 \\ 4.7 \end{pmatrix} \quad End\_stud_{y_1} = \begin{pmatrix} 20.25 \\ 20.25 \end{pmatrix} \quad Anchor\_force_{y_1} = \begin{pmatrix} 13.505 \\ 13.505 \end{pmatrix}$$

R_{y_1} matrix gives the ratio of the force applied to the strap to the strength of strap. If the value is zero, it means either the force applied is greater than the strength or the deflection of the X braced portion is greater than 1/500.

$Screw\_strap_{y_1}$ matrix gives the required screws at the end of the straps. When calculating the screw strength the diameter is assumed to be 4.8mm. The thinner gauge is considered (of stud or strap) for pull-over strength which is assumed to be governing failure mode for the screws.

$End\_stud_{y_1}$ matrix gives the force at the ends of the strap caused by the lateral force only. It needs to be checked against the axial load that can be carried by each stud. Usually providing B2B studs at the ends of the straps gives the required strength.

$Anchor\_force_{y_1}$ matrix gives the required force to hold down the ends of strap. So the end studs need to be fixed to an anchor to resist these forces.

$$\boxed{\sum_{i=2}^{rows(W\_L1_y)} \left(W\_L1_{y_{i,3}} \cdot W\_L1_{y_{i,5}}\right) = 150}$$

Total Lateral Force carried by the X braces only.
The plasterboard and effect of the structural boards are not taken into account.

The load need to be taken by the lateral bracing system is 136kN according to the above wind load calculations. Since the building is symmetrical there is no need to make calculations for torsional rigidity of the building. Assuming that the floor will act as a diaphram and distribute these loads to the walls according to their stiffenesses, adding the capacities of individual walls will be the overall strength of the building and can be compared against the total shear on the building.

NOTE: None of the internal walls are taken into account in this direction of the building. Hence the internal walls along y direction does not need to have any X braces inside. And the floor diaphram assumed to be able to transfer the wind loads accross to the external side walls only.

Walls taking the lateral load acting along X direction in the first floor:

$$W\_L2_x := \begin{pmatrix} "H" & "L" & "No.X" & "Length X" & "F\_req\_per\_X" & "Floor" & "Int./Ext" & "Strap\_t" \\ 2.7 & 9.7 & 3 & 1.6 & 14 & 1 & "E" & 1.2 \\ 2.7 & 9.7 & 3 & 2 & 14 & 1 & "I" & 1.2 \\ 2.7 & 9.7 & 3 & 2 & 14 & 1 & "I" & 1.2 \\ 2.7 & 5.3 & 2 & 1.6 & 14 & 1 & "I" & 1.2 \\ 2.7 & 4.4 & 2 & 1.6 & 14 & 1 & "I" & 1.2 \\ 2.7 & 5.3 & 2 & 1.6 & 14 & 1 & "I" & 1.2 \\ 2.7 & 9.7 & 3 & 2 & 14 & 1 & "I" & 1.2 \\ 2.7 & 9.7 & 3 & 2 & 14 & 1 & "I" & 1.2 \\ 2.7 & 9.7 & 3 & 1.6 & 14 & 1 & "E" & 1.2 \end{pmatrix}$$

$$R_{x_2} = \begin{pmatrix} 0.2 \\ 0.2 \\ 0.2 \\ 0.2 \\ 0.2 \\ 0.2 \\ 0.2 \\ 0.2 \\ 0.2 \end{pmatrix} \quad Screw\_strap_{x_2} = \begin{pmatrix} 5.7 \\ 5.7 \\ 5.7 \\ 5.7 \\ 5.7 \\ 5.7 \\ 5.7 \\ 5.7 \\ 5.7 \end{pmatrix} \quad End\_stud_{x_2} = \begin{pmatrix} 23.63 \\ 18.9 \\ 18.9 \\ 23.63 \\ 23.63 \\ 23.63 \\ 18.9 \\ 18.9 \\ 23.63 \end{pmatrix} \quad Anchor\_force_{x_2} = \begin{pmatrix} 18.69 \\ 14.71 \\ 14.71 \\ 19.44 \\ 19.44 \\ 19.44 \\ 14.71 \\ 14.71 \\ 18.69 \end{pmatrix}$$

R_{x_2} matrix gives the ratio of the force applied to the strap to the strength of strap. If the value is zero, it means either the force applied is greater than the strength or the deflection of the X braced portion is greater than 1/500.

$Screw\_strap_{x_2}$ matrix gives the required screws at the end of the straps. When calculating the screw strength the diameter is assumed to be 4.8mm. The thinner gauge is considered (of stud or strap) for pull-over strength which is assumed to be governing failure mode for the screws.

$End\_stud_{x_2}$ matrix gives the force at the ends of the strap caused by the lateral force only. It needs to be checked against the axial load that can be carried by each stud. Usually providing B2B studs at the ends of the straps gives the required strength.

$Anchor\_force_{x_2}$ matrix gives the required force to hold down the ends of strap. So the end studs need to be fixed to an anchor to resist these forces.

$$\boxed{\sum_{i=2}^{rows(W\_L2_x)} \left(W\_L2_{x_{i,3}} \cdot W\_L2_{x_{i,5}}\right) = 336}$$

Total Lateral Force carried by the X braces only.
The plasterboard and effect of the structural boards are not taken into account.

Serdar Dundar
20/08/2006
07866731125

The load need to be taken by the lateral bracing system is 314kN according to the above wind load calculations. Since the building is symmetrical there is no need to make calculations for torsional rigidity of the building. Assuming that the floor will act as a diaphram and distribute these loads to the walls according to their stiffenesses, adding the capacities of individual walls will be the overall strength of the building and can be compared against the total shear on the building.

Walls taking the lateral load acting along Y direction in the first floor:

$$W\_L2_y := \begin{pmatrix} \text{"H"} & \text{"L"} & \text{"No.X"} & \text{"Length X"} & \text{"F\_req\_per\_X"} & \text{"Floor"} & \text{"Int./Ext"} & \text{"Strap\_t"} \\ 2.7 & 30 & 5 & 2.0 & 11 & 1 & \text{"E"} & 1.2 \\ 2.7 & 30 & 5 & 2.0 & 11 & 1 & \text{"E"} & 1.2 \end{pmatrix}$$

$$R_{y_2} = \begin{pmatrix} 0.2 \\ 0.2 \end{pmatrix} \quad Screw\_strap_{y_2} = \begin{pmatrix} 4.5 \\ 4.5 \end{pmatrix} \quad End\_stud_{y_2} = \begin{pmatrix} 14.85 \\ 14.85 \end{pmatrix} \quad Anchor\_force_{y_2} = \begin{pmatrix} 9.915 \\ 9.915 \end{pmatrix}$$

R_{y_2} matrix gives the ratio of the force applied to the strap to the strength of strap. If the value is zero, it means either the force applied is greater than the strength or the deflection of the X braced portion is greater than 1/500.

$Screw\_strap_{y_2}$ matrix gives the required screws at the end of the straps. When calculating the screw strength the diameter is assumed to be 4.8mm. The thinner gauge is considered (of stud or strap) for pull-over strength which is assumed to be governing failure mode for the screws.

$End\_stud_{y_2}$ matrix gives the force at the ends of the strap caused by the lateral force only. It needs to be checked against the axial load that can be carried by each stud. Usually providing B2B studs at the ends of the straps gives the required strength.

$Anchor\_force_{y_2}$ matrix gives the required force to hold down the ends of strap. So the end studs need to be fixed to an anchor to resist these forces.

$$\boxed{\sum_{i=2}^{rows(W\_L2_y)} \left(W\_L2_{y_{i,3}} \cdot W\_L2_{y_{i,5}}\right) = 110}$$

Total Lateral Force carried by the X braces only.
The plasterboard and effect of the structural boards are not taken into account.

The load need to be taken by the lateral bracing system is 102kN according to the above wind load calculations. Since the building is symmetrical there is no need to make calculations for torsional rigidity of the building. Assuming that the floor will act as a diaphram and distribute these loads to the walls according to their stiffenesses, adding the capacities of individual walls will be the overall strength of the building and can be compared against the total shear on the building.

NOTE: None of the internal walls are taken into account in this direction of the building. Hence the internal walls along y direction does not need to have any X braces inside. And the floor diaphram assumed to be able to transfer the wind loads accross to the external side walls only.

Serdar Dundar
20/08/2006
07866731125

Walls taking the lateral load acting along X direction in the second floor:

$$W\_L3_x := \begin{pmatrix} "H" & "L" & "No.X" & "Length\ X" & "F\_req\_per\_X" & "Floor" & "Int./Ext" & "Strap\_t" \\ 2.7 & 9.7 & 3 & 1.6 & 10 & 2 & "E" & 1.0 \\ 2.7 & 9.7 & 3 & 2 & 10 & 2 & "I" & 1.0 \\ 2.7 & 9.7 & 3 & 2 & 10 & 2 & "I" & 1.0 \\ 2.7 & 5.3 & 2 & 1.6 & 10 & 2 & "I" & 1.0 \\ 2.7 & 4.4 & 2 & 1.6 & 10 & 2 & "I" & 1.0 \\ 2.7 & 5.3 & 2 & 1.6 & 10 & 2 & "I" & 1.0 \\ 2.7 & 9.7 & 3 & 2 & 10 & 2 & "I" & 1.0 \\ 2.7 & 9.7 & 3 & 2 & 10 & 2 & "I" & 1.0 \\ 2.7 & 9.7 & 3 & 1.6 & 10 & 2 & "E" & 1.0 \end{pmatrix}$$

$$R_{x_3} = \begin{pmatrix} 0.2 \\ 0.2 \\ 0.2 \\ 0.2 \\ 0.2 \\ 0.2 \\ 0.2 \\ 0.2 \\ 0.2 \end{pmatrix} \quad Screw\_strap_{x_3} = \begin{pmatrix} 4.1 \\ 4.1 \\ 4.1 \\ 4.1 \\ 4.1 \\ 4.1 \\ 4.1 \\ 4.1 \\ 4.1 \end{pmatrix} \quad End\_stud_{x_3} = \begin{pmatrix} 16.88 \\ 13.5 \\ 13.5 \\ 16.88 \\ 16.88 \\ 16.88 \\ 13.5 \\ 13.5 \\ 16.88 \end{pmatrix} \quad Anchor\_force_{x_3} = \begin{pmatrix} 12.19 \\ 10.11 \\ 10.11 \\ 13.48 \\ 13.48 \\ 13.48 \\ 10.11 \\ 10.11 \\ 12.19 \end{pmatrix}$$

R_{x_3} matrix gives the ratio of the force applied to the strap to the strength of strap. If the value is zero, it means either the force applied is greater than the strength or the deflection of the X braced portion is greater than 1/500.

$Screw\_strap_{x_3}$ matrix gives the required screws at the end of the straps. When calculating the screw strength the diameter is assumed to be 4.8mm. The thinner gauge is considered (of stud or strap) for pull-over strength which is assumed to be governing failure mode for the screws.

$End\_stud_{x_3}$ matrix gives the force at the ends of the strap caused by the lateral force only. It needs to be checked against the axial load that can be carried by each stud. Usually providing B2B studs at the ends of the straps gives the required strength.

$Anchor\_force_{x_3}$ matrix gives the required force to hold down the ends of strap. So the end studs need to be fixed to an anchor to resist these forces.

$$\boxed{\sum_{i=2}^{rows(W\_L3_x)} \left(W\_L3_{x_{i,3}} \cdot W\_L3_{x_{i,5}}\right) = 240}$$

Total Lateral Force carried by the X braces only.
The plasterboard and effect of the structural boards are not taken into account.

Serdar Dundar
20/08/2006
07866731125

The load need to be taken by the lateral bracing system is 207kN according to the above wind load calculations. Since the building is symmetrical there is no need to make calculations for torsional rigidity of the building. Assuming that the floor will act as a diaphram and distribute these loads to the walls according to their stiffenesses, adding the capacities of individual walls will be the overall strength of the building and can be compared against the total shear on the building.

Walls taking the lateral load acting along Y direction in the second floor:

$$W\_L3_y := \begin{pmatrix} \text{"H"} & \text{"L"} & \text{"No.X"} & \text{"Length X"} & \text{"F\_req\_per\_X"} & \text{"Floor"} & \text{"Int./Ext"} & \text{"Strap\_t"} \\ 2.7 & 30 & 5 & 2.0 & 7 & 2 & \text{"E"} & 1.0 \\ 2.7 & 30 & 5 & 2.0 & 7 & 2 & \text{"E"} & 1.0 \end{pmatrix}$$

$$R_{y_3} = \begin{pmatrix} 0.1 \\ 0.1 \end{pmatrix} \quad Screw\_strap_{y_3} = \begin{pmatrix} 2.9 \\ 2.9 \end{pmatrix} \quad End\_stud_{y_3} = \begin{pmatrix} 9.45 \\ 9.45 \end{pmatrix} \quad Anchor\_force_{y_3} = \begin{pmatrix} 4.762 \\ 4.762 \end{pmatrix}$$

R_{y_3} matrix gives the ratio of the force applied to the strap to the strength of strap. If the value is zero, it means either the force applied is greater than the strength or the deflection of the X braced portion is greater than 1/500.

$Screw\_strap_{y_3}$ matrix gives the required screws at the end of the straps. When calculating the screw strength the diameter is assumed to be 4.8mm. The thinner gauge is considered (of stud or strap) for pull-over strength which is assumed to be governing failure mode for the screws.

$End\_stud_{y_3}$ matrix gives the force at the ends of the strap caused by the lateral force only. It needs to be checked against the axial load that can be carried by each stud. Usually providing B2B studs at the ends of the straps gives the required strength.

$Anchor\_force_{y_3}$ matrix gives the required force to hold down the ends of strap. So the end studs need to be fixed to an anchor to resist these forces.

$$\boxed{\sum_{i=2}^{rows(W\_L3_y)} \left(W\_L3_{y_{i,3}} \cdot W\_L3_{y_{i,5}}\right) = 70}$$

Total Lateral Force carried by the X braces only.
The plasterboard and effect of the structural boards are not taken into account.

The load need to be taken by the lateral bracing system is 67kN according to the above wind load calculations. Since the building is symmetrical there is no need to make calculations for torsional rigidity of the building. Assuming that the floor will act as a diaphram and distribute these loads to the walls according to their stiffenesses, adding the capacities of individual walls will be the overall strength of the building and can be compared against the total shear on the building.

NOTE: None of the internal walls are taken into account in this direction of the building. Hence the internal walls along y direction does not need to have any X braces inside. And the floor diaphram assumed to be able to transfer the wind loads accross to the external side walls only.

Walls taking the lateral load acting along X direction in the third floor:

$$W\_L4_x := \begin{pmatrix} "H" & "L" & "No.X" & "Length X" & "F\_req\_per\_X" & "Floor" & "Int./Ext" & "Strap\_t" \\ 2.7 & 9.7 & 3 & 1.6 & 5 & 3 & "E" & 1.0 \\ 2.7 & 9.7 & 3 & 2 & 5 & 3 & "I" & 1.0 \\ 2.7 & 9.7 & 3 & 2 & 5 & 3 & "I" & 1.0 \\ 2.7 & 5.3 & 2 & 1.6 & 5 & 3 & "I" & 1.0 \\ 2.7 & 4.4 & 2 & 1.6 & 5 & 3 & "I" & 1.0 \\ 2.7 & 5.3 & 2 & 1.6 & 5 & 3 & "I" & 1.0 \\ 2.7 & 9.7 & 3 & 2 & 5 & 3 & "I" & 1.0 \\ 2.7 & 9.7 & 3 & 2 & 5 & 3 & "I" & 1.0 \\ 2.7 & 9.7 & 3 & 1.6 & 5 & 3 & "E" & 1.0 \end{pmatrix}$$

$$R_{x_4} = \begin{pmatrix} 0.1 \\ 0.1 \\ 0.1 \\ 0.1 \\ 0.1 \\ 0.1 \\ 0.1 \\ 0.1 \\ 0.1 \end{pmatrix} \quad Screw\_strap_{x_4} = \begin{pmatrix} 3.1 \\ 3.1 \\ 3.1 \\ 3.1 \\ 3.1 \\ 3.1 \\ 3.1 \\ 3.1 \\ 3.1 \end{pmatrix} \quad End\_stud_{x_4} = \begin{pmatrix} 8.44 \\ 6.75 \\ 6.75 \\ 8.44 \\ 8.44 \\ 8.44 \\ 6.75 \\ 6.75 \\ 8.44 \end{pmatrix} \quad Anchor\_force_{x_4} = \begin{pmatrix} 6.46 \\ 4.95 \\ 4.95 \\ 6.63 \\ 6.63 \\ 6.63 \\ 4.95 \\ 4.95 \\ 6.46 \end{pmatrix}$$

R_{x_4} matrix gives the ratio of the force applied to the strap to the strength of strap. If the value is zero, it means either the force applied is greater than the strength or the deflection of the X braced portion is greater than 1/500.

$Screw\_strap_{x_4}$ matrix gives the required screws at the end of the straps. When calculating the screw strength the diameter is assumed to be 4.8mm. The thinner gauge is considered (of stud or strap) for pull-over strength which is assumed to be governing failure mode for the screws.

$End\_stud_{x_4}$ matrix gives the force at the ends of the strap caused by the lateral force only. It needs to be checked against the axial load that can be carried by each stud. Usually providing B2B studs at the ends of the straps gives the required strength.

$Anchor\_force_{x_4}$ matrix gives the required force to hold down the ends of strap. So the end studs need to be fixed to an anchor to resist these forces.

$$\boxed{\sum_{i=2}^{rows(W\_L4_x)} \left(W\_L4_{x_{i,3}} \cdot W\_L4_{x_{i,5}} \right) = 120}$$

Total Lateral Force carried by the X braces only.
The plasterboard and effect of the structural boards are not taken into account.

Serdar Dundar
20/08/2006
07866731125

The load need to be taken by the lateral bracing system is 100kN according to the above wind load calculations. Since the building is symmetrical there is no need to make calculations for torsional rigidity of the building. Assuming that the floor will act as a diaphram and distribute these loads to the walls according to their stiffenesses, adding the capacities of individual walls will be the overall strength of the building and can be compared against the total shear on the building.

Walls taking the lateral load acting along Y direction in the third floor:

$$W\_L4_y := \begin{pmatrix} \text{"H"} & \text{"L"} & \text{"No.X"} & \text{"Length X"} & \text{"F\_req\_per\_X"} & \text{"Floor"} & \text{"Int./Ext"} & \text{"Strap\_t"} \\ 2.7 & 30 & 5 & 2.0 & 4 & 3 & \text{"E"} & 1.0 \\ 2.7 & 30 & 5 & 2.0 & 4 & 3 & \text{"E"} & 1.0 \end{pmatrix}$$

$$R_{y_4} = \begin{pmatrix} 0.1 \\ 0.1 \end{pmatrix} \quad Screw\_strap_{y_4} = \begin{pmatrix} 2.5 \\ 2.5 \end{pmatrix} \quad End\_stud_{y_4} = \begin{pmatrix} 5.4 \\ 5.4 \end{pmatrix} \quad Anchor\_force_{y_4} = \begin{pmatrix} 3.426 \\ 3.426 \end{pmatrix}$$

R_{y_4} matrix gives the ratio of the force applied to the strap to the strength of strap. If the value is zero, it means either the force applied is greater than the strength or the deflection of the X braced portion is greater than 1/500.

$Screw\_strap_{y_4}$ matrix gives the required screws at the end of the straps. When calculating the screw strength the diameter is assumed to be 4.8mm. The thinner gauge is considered (of stud or strap) for pull-over strength which is assumed to be governing failure mode for the screws.

End_stud_y matrix gives the force at the ends of the strap caused by the lateral force only. It needs to be checked against the axial load that can be carried by each stud. Usually providing B2B studs at the ends of the straps gives the required strength.

$Anchor\_force_{y_4}$ matrix gives the required force to hold down the ends of strap. So the end studs need to be fixed to an anchor to resist these forces.

$$\boxed{\sum_{i=2}^{rows(W\_L4_y)} \left(W\_L4_{y_{i,3}} \cdot W\_L4_{y_{i,5}} \right) = 40}$$

Total Lateral Force carried by the X braces only.
The plasterboard and effect of the structural boards are not taken into account.

The load need to be taken by the lateral bracing system is 32kN according to the above wind load calculations. Since the building is symmetrical there is no need to make calculations for torsional rigidity of the building. Assuming that the floor will act as a diaphram and distribute these loads to the walls according to their stiffenesses, adding the capacities of individual walls will be the overall strength of the building and can be compared against the total shear on the building.

NOTE: None of the internal walls are taken into account in this direction of the building. Hence the internal walls along y direction does not need to have any X braces inside. And the floor diaphram assumed to be able to transfer the wind loads accross to the external side walls only.

Serdar Dundar
07/05/2006
07866731125

PAGE 1 ;

This page explains the make up of the walls floors and ceilings, also the loads of every element. The first column of every matrix is the name of the element the second column is the kN/m2 load of that element.

The second column of every matrix added together to find out the load per meter square of that part of the structure.

The **Live loads** are enteren in Live_Loads matrix.

Wind Loads are entered in Wind_loads matrix, such as;

Stud wind load is the max. local wind pressure on the building, it is used to design an individual stud, assuming that it is in the max. local pressure zone.

Rafter wind load is the max. pressure on the roof members.

Rafter suction is the max. wind load suction on the roof members

Overall Walls is the sum of absolute value of the windward and leeward wind loads on the walls, this is used to analyse the lateral bracing system of the building.

Also on this page the steel quality which is used throughout the building is defined.

PAGE 2 ;

This page starts with the summary of the loads, which are calculated by referring back to the first page and adding them together as explained in the first page.
For example;

Function used to calculate the roof dead load is :

$$\mathrm{DEAD\_LOADS\_R} := \left(\sum_{N=1}^{\mathrm{rows(DEAD\_LOADS\_R)}} \mathrm{DEAD\_LOADS\_R}_{N,2} \right) \mathrm{kN \cdot m^{-2}}$$

The above function simply adds all the members of the second column in DEAD_LOADS_R matrix together to find the total dead load per meter square of the roof. Same principle applies for calculation of the other dead loads.

Live loads and wind loads are then taken from the relevant position in the matrices.
For example;

$$\mathrm{LIVE\_LOAD\_R} := \max\left[\left(\mathrm{LIVE\_LOADS}_{4,2}\right), \mathrm{LIVE\_LOADS}_{5,2}\right] \cdot \left(\mathrm{kN \cdot m^{-2}}\right)$$

This function simply compares the snow load and the imposed roof live load and takes the max. as the LIVE_LOAD_R, which is used in rafter design as the live load on the roof.

Similarly; $\mathrm{LIVE\_LOAD\_F} := \left(\mathrm{LIVE\_LOADS}_{1,2} + \mathrm{LIVE\_LOADS}_{3,2}\right)\mathrm{kN \cdot m^{-2}}$

LIVE_LOAD_F function adds, the live load of the floor, and the service loads together to find the live load on the floor.

Serdar Dundar
07/05/2006
07866731125

Then the joist load combinations are defined. In the form of LCJ_1 which stands stands for " first load combination for joists"

The rest of the page is self explanatory. The moment and shear is calculated for a given span spacing and depth of a joist for the most critical load combination. And then the deflection requirements are calculated in terms of the moment of inertia required.

PAGE 3 ;

This page shows the design of the floor joists.

The max. of the moment of inertia required is calculated comparing the requirements for different deflection criterias.

Then the strength criterias are calculated, in terms of max. chord foce and max. diagonal force. These forces are compared against the section property tables for a proposed joist section (by the user), and the section used for the joists is written in bold letters.***SCS_500_90_350_0.75mm***

Below that, in the middle of the page, an equivalent moment of inertia for the joists is calculated, including an option to provide a plate on the sides of the joists.

PAGE 4 ;

This page starts with the joist design capacity check.

The equivalent moment of inertia and the section modulus are used to compare against the applied max. moment and required moment of inertia (calculated through the deflection criterias), then the capacity ratios are calculated, RW_j1 and RI_j1 should be less than 1, for the safe design.

Next the max. diagonal force is used to find out the amount of tek screws/rivets required at the end f the diagonals, the screw strength is also calculated for the given joist thickness.

The rafters are designed as simple supported beams, all the calculations are self explanatory.

PAGE 5 ;

Rafter design continues, self explanatory.

PAGE 6 ;

Rafter design continues and ceiling members are designed in the same way as rafters - simple supported beam-, functions and equations are self explanatory.

PAGE 7 ;

Ceiling design continues.

Serdar Dundar
07/05/2006
07866731125

PAGE 8 ;

Wall Stud calculations.

Two vertical load paths are taken into account, one of them is through the external walls, and the other is through the internal walls. Description of the load path is written at the top right of the page (side wall receiving joist type 1 at every floor level and rafter type 1 at roof level.)All the walls are lined up throughout the building and all the joists at every level are spanning the same direction.

Load combinations are defined as STUD LOAD COMBINATIONS.

Trial and error procedure is taken to design the walls, so a proposed section is entered for a wall in the load path and then the stud is checked against the load combinations defined. If the load applied vs. strength ratios are all less than 1 for all the combinations then the stud passes.

WALLS_E matrix defines the height and spacing of the studs at every floor level (1 stands for ground floor, 2 stands for first and so on)

$$\text{Walls\_E} := \begin{pmatrix} \text{"storey"} & \text{"Height"} & \text{"Stud Spacing"} \\ 4 & 2.7 & 600 \\ 3 & 2.7 & 600 \\ 2 & 2.7 & 400 \\ 1 & 2.7 & 400 \end{pmatrix}$$

Then SECTION_LP1 matrix (sections along load path1) is entered as below, the stud dimensions and the section properties according to the predefined steel quality are all entered into the below matrix, the Scottsdale section property tables are used as reference. Axial and moment capacities as well as effective section properties are all entered.

$$\text{Section\_LP1} := \begin{pmatrix} \text{"storey"} & \text{"Stud\_depth"} & \text{"Thickness"} & \text{"Axial"} & \text{"Moment"} & \text{"Ieff"} & \text{"Weff"} & \text{"Nog\_centers"} \\ 4 & 140 & 0.75 & 17.0 & 1.62 & 466799 & 5707 & 1350 \\ 3 & 140 & 1.0 & 24.0 & 2.43 & 671801 & 8757 & 1350 \\ 2 & 140 & 1.0 & 24.0 & 2.43 & 671801 & 8757 & 1350 \\ 1 & 140 & 1.2 & 32.0 & 3.70 & 831211 & 11125 & 1350 \end{pmatrix}$$

PAGE 9 ;

Wall Stud calculations.

Vertical loads are calculated per meter of wall per floor level.
Dead loads and live loads are calculated seperately.
For example; the subroutine below calculates the distribution of the dead load from the roof members to the walls.

$$\text{R\_r1}_{DL} := \left| \begin{array}{l} \text{R\_r}_{DL_1} \leftarrow 0 \\ \text{for } i \in 2 .. \text{rows(Walls\_E)} \\ \quad \text{R\_r}_{DL_i} \leftarrow \dfrac{(\text{DEAD\_LOADS\_R}) \cdot \text{Span\_r1} \cdot \cos(\text{slope})^{-1}}{2} \\ \text{submatrix}\left(\text{R\_r}_{DL}, 2, \text{rows(Walls\_E)}, 1, 1\right) \end{array} \right.$$

<<< Repeat for every floor level

<<< Dead load of the roof is multiplied by the span horizontal span and divided into 2 to find the end reaction on the wall, and assigned to the columns of R_r1.DL.

Phone: 020 8956 2373
Fax: 020 8956 2374

500 Chiswick High Road
Center 500 Suite 35
W4 5RG

Serdar Dundar
07/05/2006
07866731125

$$R\_r1_{DL} := \begin{pmatrix} 0.577 \\ 0.577 \\ 0.577 \\ 0.577 \end{pmatrix} kN \cdot m^{-1}$$

Results are tabulated to show the load per meter on every wall along the load path, from thop to bottom.

Same principle is used on the rest of the page, and on the other subroutines.

PAGE 10 ;

Wall Stud calculations.

Load per meter of walls on every floor along the load path is calculated. For example;

Dead load of the Walls_E:

$$R\_w_{DL} := \left| \begin{array}{l} R\_w_{DL_1} \leftarrow 0 \\ \text{for } i \in 2 .. \text{rows(Walls\_E)} \\ \quad R\_w_{DL_i} \leftarrow Walls\_E_{i,2} \cdot DEAD\_LOADS\_EW \cdot m \\ \text{submatrix}\left(R\_w_{DL}, 2, \text{rows(Walls\_E)}, 1, 1\right) \end{array} \right.$$

$$R\_w_{DL} := \begin{pmatrix} 2.43 \\ 2.43 \\ 2.43 \\ 2.43 \end{pmatrix} \cdot kN \cdot m^{-1}$$

Dead Loads Transferred from Joists:

$$R\_j1_{DL} := \left| \begin{array}{l} R\_j1_{DL_1} \leftarrow 0 \\ \text{for } i \in 2 .. \text{rows(Walls\_E)} \\ \quad \left| \begin{array}{l} R\_j1_{DL_i} \leftarrow \dfrac{(DEAD\_LOADS\_F) \cdot Span\_j1}{2} \\ R\_j1_{DL_2} \leftarrow 0 \end{array} \right. \\ \text{submatrix}\left(R\_j1_{DL}, 2, \text{rows(Walls\_E)}, 1, 1\right) \end{array} \right.$$

$$R\_j1_{DL} := \begin{pmatrix} 0 \\ 2.6 \\ 2.6 \\ 2.6 \end{pmatrix} \cdot kN \cdot m^{-1}$$

In the Rw_DL subroutine, the dead load per meter meter square of a wall is taken from the DEAD_LOADS_EW and the height of the walls are taken from Walls_E matrix to be multiplied together to find the dead load per meter of the wall itself on every floor. The R_w_DL matrix results are arranged to show the dead load on every floor level, since the wall height is the same and the load per meter square is the same all the members of the matrix are the same value.

In the R_j1_DL subroutine, the dead load per meter square of floor is multiplied by the span of joist and divided by two to find the reaction on the walls along this load path, since joists are spanning the same distance on very floor the values in the matrix are the same, except the top floor does not have any joists but it has only the ceiling frames carrying plasterboard.

Live loads are calculated the same way as above.

Serdar Dundar
07/05/2006
07866731125

Then the loads transferred from the roof,ceiling,floors,walls etc. are all added together to find the vertical load distribution from top to bottom, using the subroutine below;

Total Vertical Loads on Walls_E from top to bottom:

$$
\begin{aligned}
\text{Vertical\_LP1} := \;& LP1_{1,1} \leftarrow (\text{rows}(\text{Walls\_E}) - 1)\cdot kN\cdot m^{-1} \\
& LP1_{1,2} \leftarrow R\_r1_{DL_1} + R\_c1_{DL_1} + R\_w_{DL_1} \\
& LP1_{1,3} \leftarrow R\_r1_{LL_1} + R\_c1_{LL_1} \\
& \text{for } i \in 2 .. \text{rows}(\text{Walls\_E}) - 1 \\
& \quad LP1_{i,2} \leftarrow R\_w_{DL_i} + R\_j1_{DL_i} + LP1_{i-1,2} \\
& \quad LP1_{i,3} \leftarrow R\_j1_{LL_i} + LP1_{i-1,3} \\
& \quad LP1_{i,1} \leftarrow (\text{rows}(\text{Walls\_E}) - i)\cdot kN\cdot m^{-1} \\
& LP1
\end{aligned}
$$

<<<Dead load acting on the 3rd floor walls

<<<Live load acting on the 3rd floor walls

<<< For every floor level

<<<DL on the floor above+ DL from the joists + DL from the wall itself

<<<LL on the floor above + LL from the joists on the floor in question

So the subroutine first calculates the load at the highest floor, and goes to the floor below adding the loads transferred from the wall above to the floor loads at that level until the loads on the ground floor wall are calculated.

PAGE 11 ;

$$
\begin{array}{cccc}
 & & \text{DEAD} & \text{LIVE} \\
\text{Vertical\_LP1} = & \left(\begin{array}{ccc} 4 & 3.65 & 1.88 \\ 3 & 8.68 & 5.02 \\ 2 & 13.71 & 8.15 \\ 1 & 18.74 & 11.29 \end{array}\right) & \frac{1}{m} kN &
\end{array}
$$

Dead load and live loads per meter of the walls along this wall path are tabulated in the form of a matrix to show from top to bottom floor (4 stands for the top floor, 1 stands for the ground floor)

Then the wind load on the wall studs is calculated at every floor level, the max. local pressure/suction is used to design all the wall studs individually, since the spacing of the studs may differ from floor to floor, the unfactored moment per stud is calculated using the stud height (referring backto Walls_E matrix), WIND_S (taken out of the WIND_LOADS matrix), spacing of the studs (from Walls_E matrix)

Then V_LOADS_LP1 subroutine is written to find out the vertical load per stud at every floor level, for each load combination. The spacings of the studs are taken from the Walls_E matrix and loads are taken from Vertical_LP1.

PAGE 12 ;

The results of the V_LOADS_LP1 is presented, to show the vertical load on the walls at every floor level along this wall path for every load combination.

$$\begin{array}{cccccc} & LC1 & LC2 & LC3 & LC4 & LC5 \\ \text{V\_LOADS\_LP1} = & \left(\begin{array}{ccccc} 4.88 & 3.07 & 3.99 & 2.19 & 0.00 \\ 12.11 & 7.29 & 9.86 & 5.21 & 0.00 \\ 12.90 & 7.68 & 10.49 & 5.48 & 0.00 \\ 17.72 & 10.49 & 14.41 & 7.49 & 0.00 \end{array}\right) \text{kN} \end{array}$$

<<<Load Combinations

<<< Vertical load per stud along this load path at the Top floor

<<< Vertical load per stud along this load path at the Ground Floor

Since the 5th load combination is 1.4WL there is no vertical load taken into account for this combination.
Same principle is applied to horizontal loads on the studs, but now it is the moment which is taken into consideration.
H_LOADS_LP1 is calculated and the results are tabulated to show the moments applied per stud at every floor level for every load combination.

Stud capacity checks are done with the subroutine **Check_LP1.**

```
Check_LP1 := | for i ∈ 1 .. rows(Walls_E) − 1
             |   for j ∈ 1 .. cols(H_LOADS_LP1)
             |     M_all ← Section_LP1[i+1,5]·kN·m
             |     A_all ← Section_LP1[i+1,4]·kN
             |     M_appl ← H_LOADS_LP1[i,j]
             |     A_appl ← V_LOADS_LP1[i,j]
             |     Ratio[i,j] ← (M_appl / M_all) + (A_appl / A_all)
             | Ratio
```

<<<For every floor level
<<<For every load combination

<<<Allowable moment and axial load per stud are taken from Section_LP1 matrix using the floor level as reference.

<<<Applied vertical axial loads and applied moments due to wind are already calculated for every load combination, here they are compared against the capacities and a load vs. capacity ratio is calculated for every wall along this load path and for every load combination listed. The results of this subroutine should be always less than 1 showing that the applied loads are less than the capacity.

PAGE 13,14,15,16 ;

This is the second load path which is along the internal walls which are receiving joist type 1 at every floor level on one side and joist type two on the other side.

The joist type two can be of a smaller spanning joist so the span of this joist is also entered (in this case same as joist type 1) to calculate the total load coming from both sides.

All the matrices are self explanatory and same principles and notations are used as previous pages (refer to design of load path1).

The walls along this load path are all internal walls so there is no wind load taken into account.

PAGE 17 ;

The overall stability of the building is analysed on this page. The building dimensions roof slope and joist depths are entered, the subroutine below is used to calculated overall lateral wind load to be resisted per floor level.

It is assumed that the floors will act as a horizontal diaphram such that it will distribute the wind loads to the walls in proportion to their stiffnesses.

$$\text{Wind\_X} := \left| \begin{array}{l} \text{Wind}_1 \leftarrow \left(s_y \cdot \frac{s_x}{2} \cdot \tan(\alpha) \cdot \text{Wind}_O \right) + \frac{\text{Walls\_E}_{2,2} m + j_d}{2} \cdot s_y \cdot \text{Wind}_O \\ \text{for } i \in 2 .. \text{rows(Walls\_E)} - 1 \\ \quad \text{Wind}_i \leftarrow \text{Wind}_{i-1} + \left[\left(\text{Walls\_E}_{i,2} m + j_d \right) \cdot s_y \cdot \text{Wind}_O \right] \\ \text{Wind} \end{array} \right.$$

s_x s_y and α are all defined on page 17.

Wind_O is the overall wind load which is calculated by adding the windward and leeward wind pressures together, and it is entered in WIND_LOADS matrix on page 1.

The subroutine above simply calculates the amount of wind load acting at the top floor level (to the roof and half the storey height), and it finds the lateral wind load need to be resisted per floor level.

$$\boxed{\text{Wind\_X} = \begin{pmatrix} 100 \\ 207 \\ 314 \\ 421 \end{pmatrix} \text{kN}}$$

<<< The result matrix shows the lateral load per floor from top to bottom.

Same analysis is made for the wind loads acting on the perpendicular direction.

PAGE 18 ;

On this page the dead loads acting on the walls are calculated, and multiplied by 0.9, before there are used in the shear wall design. Obviously the dead loads will decrease holding down requirements.

It is assumed that there will be additional studs provided next to the end studs of a flat strap to take the vertical loads only. As the effect of the lateral loads on the end studs of a flat strap brace alone, usually requires a double stud.

DL_lateral_LP1 is the dead load distribution through load path 1 per floor, whereas DL_lateral_LP2 is likewise for load path 2.

PAGE 19 ;

On this page a subrotuine is written to analyse the capacity of a flat strap. The strap width is assumed to 150mm, thickness of the strap is a variable.

1 - The strength of the diagonal member is calculated and compared against the required force to be resisted
2 - Number of screws needed at the ends of the diagonal are calculated according to the minimum of stud thickness or the diagonal thickness.
3 - The axial force on the end studs resulting from the lateral load only are calculated.
4 - The anchoring requirements are calculated, by deducting the dead load acting on the stud from the uplift acting on the stud due to the lateral load.

Serdar Dundar
07/05/2006
07866731125

If the deflection requirement; $\Delta \leftarrow \dfrac{Freq\cdot\sqrt{H^2 + Lx^2}}{A\_strap\cdot E} < \dfrac{H}{500}$ and strength requirement; $Strength > Freq$ (capacity of the strap greater than applied force)

Are both satisfied then the subroutine returns a matrix with the following information for that strap bracing;

$$\begin{pmatrix} \text{Ratio} \\ \text{screws\_diag} \\ \text{end\_stud} \\ \text{anchor\_force} \end{pmatrix}$$

Where the results ar all defined in the subroutine.

PAGE 20...27 ;

On these pages the lateral load resisting system of the building is analysed floor by floor for every floor level. Below is the analysis of lateral load distribution on the ground floor along x direction;
W_L1x matrix is entered by the designer, below are the explanations of the entries;

$$W\_L1_x := \begin{pmatrix} \text{"H"} & \text{"L"} & \text{"No.X"} & \text{"Length X"} & \text{"F\_req\_per\_X"} & \text{"Floor"} & \text{"Int./Ext"} & \text{"Strap\_t"} \\ 2.7 & 9.7 & 3 & 1.6 & 18 & 0 & \text{"E"} & 1.2 \\ 2.7 & 9.7 & 3 & 2.0 & 18 & 0 & \text{"I"} & 1.2 \\ 2.7 & 9.7 & 3 & 2.0 & 18 & 0 & \text{"I"} & 1.2 \\ 2.7 & 5.3 & 2 & 1.6 & 18 & 0 & \text{"I"} & 1.2 \\ 2.7 & 4.4 & 2 & 1.6 & 18 & 0 & \text{"I"} & 1.2 \\ 2.7 & 5.3 & 2 & 1.6 & 18 & 0 & \text{"I"} & 1.2 \\ 2.7 & 9.7 & 3 & 2 & 18 & 0 & \text{"I"} & 1.2 \\ 2.7 & 9.7 & 3 & 2 & 18 & 0 & \text{"I"} & 1.2 \\ 2.7 & 9.7 & 3 & 1.6 & 18 & 0 & \text{"E"} & 1.2 \end{pmatrix}$$

H; stands for the height of the wall,
L; stands for the length of the wall
No. X ; stands for the number of X braces in this wall, all of which assumed to be of same width and height.
Length X; stands for the width of X braces in this wall
F_req_per_X; stands for the force demand from each of the braces in this wall. the theory is; by adding the required forces multiplied by the number of X braces should give a force (lateral load carrying capacity) greater than the lateral force applied (which was calculated earlier in WIND_X AND WIND_Y matrices.
Floor: The floor level at the bottom of the wall in question, this is used to find the dead loads acting on the wall, which is used to to decrease anchorage requirements.
Int/Ext: This defines if the wall is an external or internal wall, which is used to find the dead load on the wall.
Strap_t: Thickness of the flat straps -the width is assumed to be 150 by default-

The rest of the results are all self explanatory, and the definitions are given for the matrices on the pages.

Disproportionate collapse calculations are submitted seperately.