Structural calculations

Site details

Address 'Roselea' Smiths Loke
Bradwell
Great Yarmouth
Norfolk
NR31 8DG

Building description

A detatched single story timber framed house, with rooms in the roof. Overall size is 21m x 13m

Basis for design

Calculations comply with the following Eurocodes

BS EN 1990:2002 +A1:2005
BS EN 1991-1-1:2002
BS EN 1991-1-3:2003
BS EN 1991-1-4:2005 + A1:2010
BS EN 1992-1-1:2004 + A1:2010
BS EN 1995-1-1:2004 +A1:2008
BS EN 1997-1-1:2004 + A1:2009
and the national annexs thereof.

The folowing text provides much of the detail for performing the calculations

Structural Timber Design to Eurocode 5 by Jack Porteous and Adby Kermani
Printed by Blackwell Publishing 2007 ISBN:978-14051-4638-8
This book will be refered to below as the the 'Reference Book'

Spreadsheets

"Loads.xls" has been used to perform bulk calculations and concatenate data. It sheets will be refered to as appropriate in the calculations below.
"Wood Data.xls" contains material properties for many wood based materials which are used in the calculations.

Software used

Finnwood 2.1 from Metsawood is used to perform analysis of floor joists and door lintels made of Metsawood products, it is also used as a general beam analysis program to calculate bending and shear forces.
Tedds from CSC is used for calculations of concrete parts. (all results have been translated in this document)

Design constants

Partial safety factors

Partial safety factor for permanant loads $\gamma_G = 1.35$

Partial safety factor variable loads $\gamma_Q = 1.50$

Material Safety factors

Structural Steel $\gamma_{M0} = 1.0$

[EC 2 NA 2.15] $\gamma_{M1} = 1.0$

$\gamma_{M2} = 1.25$

Steel reinforcment $\gamma_S = 1.15$

Concrete $\gamma_C = 1.5$

Physical constants

Material densities $\rho_{conc} = 2500\text{kg}\cdot\text{m}^{-3}$ $\gamma_{conc} = g\cdot\rho_{conc}$

$\rho_{water} = 1000\text{kg}\cdot\text{m}^{-3}$ $\gamma_{water} = g\cdot\rho_{water}$

Some enumerated values for directions and zones $(front\ \ left\ \ back\ \ right) = (0\ \ 1\ \ 2\ \ 3)$

$(A\ \ B\ \ C\ \ D\ \ E) = (0\ \ 1\ \ 2\ \ 3\ \ 4)$

$(F\ \ G\ \ H\ \ I\ \ J\ \ K\ \ L\ \ M\ \ N) = (0\ \ 1\ \ 2\ \ 3\ \ 4\ \ 5\ \ 6\ \ 7\ \ 8)$

Standard Functions

Function to generate all possible load combinations

Parameters

ψ = matrix of ψ values for each load

ψ_0,load duration,ψ_2

l = vector of load values

permanent load,

variable load$_1$,

.

.

variable load$_n$

γ = vector of partial safety factors for each load

Results

matrix of load combinations with these columns

ULS = ULS load

SLS_i = Instantenous SLS load

SLS_f = Final SLS load

d_l = minimum load duration

mG = multiplier applied to permanent load

mQ_1 = multiplier applied to variable load$_1$

.

.

mQ_n = multiplier applied to variable load$_n$

```
Load_Combos(ψ, l, γ) = | n ← rows(l) − 1
                       | d3 ← 0
                       | s ← 5
                       | ml ← 0
                       | for r ∈ 0 .. 3^n − 1
                       |   | m[r,s] ← 1
                       |   | x ← 0
                       |   | for c ∈ 1 .. n
                       |   |   | m[r,c+s] ← 0  if  d[c] = 0
                       |   |   | m[r,c+s] ← ψ[c,0]  if  d[c] ≥ 1
                       |   |   | m[r,c+s] ← 1  if  d[c] = 2 ∧ ¬x
                       |   |   | x ← 1  if  d[c] = 2
                       |   | for c ∈ 1 .. n
                       |   |   | d[c] ← d[c] + 1
                       |   |   | d[c] ← 0  if  d[c] > 2
                       |   |   | (break)  if  d[c] ≠ 0
                       |   | ULS ← 0
                       |   | SLSi ← 0
                       |   | SLSf ← 0
                       |   | dur ← 0
                       |   | for c ∈ 0 .. n
                       |   |   | ULS ← ULS + l[c]·γ[c]·m[r,c+s]
                       |   |   | SLSi ← SLSi + l[c]·m[r,c+s]
                       |   |   | SLSf ← SLSf + l[c]·m[r,c+s]·ψ[c,2]
                       |   |   | if  ψ[c,1] > dur ∧ m[r,c+s]
                       |   |   |   | dur ← ψ[c,1]
                       |   |   |   | ψ2 ← ψ[c,2]
                       |   | m[r,0] ← ULS
                       |   | m[r,1] ← SLSi
                       |   | m[r,2] ← SLSf
                       |   | m[r,3] ← dur
                       |   | m[r,4] ← ψ2
                       | m
```

Enumerated columns in Load combo's

$$(ULS \;\; SLS_i \;\; SLS_f \;\; d_l \;\; \psi_2 \;\; mG \;\; mQ_1 \;\; mQ_2 \;\; mQ_3) = (0 \;\; 1 \;\; 2 \;\; 3 \;\; 4 \;\; 5 \;\; 6 \;\; 7 \;\; 8 \;\; 9)$$

Calculate Bending and shear for a simply supported beam with point loads applied.
Parameters (all must be unitless)
- Q = loads applied
- P = position of loads
- l = length of beam

$MV\_point\_loads(Q, P, l) =$

```
n ← rows(Q) − 1
for r ∈ 0.. n
    t_r ← Q_r·P_r
sq ← ΣQ
m_1 ← sq − ((Σt) / l)
m_2 ← sq − m_1
m_0 ← 0
for r ∈ 0.. n
    t ← m_1·P_r
    for c ∈ r − 1.. 0    if r > 0
        t ← t − Q_c·(P_r − P_c)
    m_0 ← t  if t > m_0
return m
```

$$(M_d \;\; SR_1 \;\; SR_2) = (0 \;\; 1 \;\; 2)$$

Calculate deflection factor for a simply supported bean with point loads
Parameters (all must be unitless)
Q = loads applied
P = position of loads
l = length of beam
Returns
E x I x deflection (y) [divide by EI to get y]

```
Ely_point_loads(Q, P, l) = | n ← rows(Q) − 1
                           | for r ∈ 0.. n
                           |   t_r ← Q_r·P_r
                           | sq ← ΣQ
                           | r1 ← sq − (Σt / l)
                           | A ← r1·l^3 / 6
                           | for r ∈ 0.. n
                           |   | i ← l − P_r
                           |   | A ← A − (Q_r/6)·i^3  if i > 0
                           | A ← −A / l
                           | x ← l / 2
                           | Ely ← r1·x^3/6 + A·x
                           | for r ∈ 0.. n
                           |   | i ← x − P_r
                           |   | Ely ← Ely − (Q_r/6)·i^3  if i > 0
                           | return −Ely
```

Function to calculate the value of k_c for a timber post
h = size of member perpendicluar to axis
l = effective length of member
f = $f_{c.0.k}$ for the member
E = $E_{0.05}$
From EC5 Equations 6.21-22,6.25-29

```
calc_k_c(h, l, f, E) = | λ_rel ← (l·√12)/(h·π) · √(f/E)
                       | k ← 0.5[1 + 0.2(λ_rel − 0.3) + λ_rel^2]
                       | 1 / (k + √(k^2 − λ_rel^2))
```

Function to test limiting values

```
OKifLT(value, limit) = | "O.K."  if value ≤ limit
                       | "******* VALUE OUTSIDE LIMIT *********"  otherwise
```

Function to check value is less than or equal to 1

$$\text{Check(value)} = \left| \begin{array}{l} \text{value} \leftarrow \max(\text{value}) \quad \text{if} \ \ \text{rows(value)} > 1 \\ r \leftarrow \text{"******* THIS CHECK HAS FAILED *********"} \\ r \leftarrow \text{"O.K."} \quad \text{if} \ \ \text{value} \le 1.0 \end{array} \right.$$

ψvalues for each design category

$$\psi\text{table} = \begin{pmatrix} \text{"A"} & \text{"Category A: domestic and residential areas"} & 0.7 & 0.5 & 0.3 \\ \text{"B"} & \text{"Category B: office areas"} & 0.7 & 0.5 & 0.3 \\ \text{"C"} & \text{"Category C: areas where people congregate"} & 0.7 & 0.7 & 0.6 \\ \text{"H"} & \text{"Category H: roofs"} & 0.7 & 0 & 0 \\ \text{"S"} & \text{"Snow loads for altitude <= 1000 m"} & 0.5 & 0.2 & 0 \\ \text{"W"} & \text{"Wind loads on buildings"} & 0.5 & 0.2 & 0 \end{pmatrix}$$

extract a ψ value from the table

$$\psi\text{val}(c,n) = \left| \begin{array}{l} r \leftarrow \text{match}\left(c, \psi\text{table}^{\langle 0 \rangle}\right)_0 \\ \psi\text{table}_{r,\,n+2} \end{array} \right.$$

Load duration enumeration

$$\text{durs} = \begin{pmatrix} \text{"Permanent"} \\ \text{"Long"} \\ \text{"Medium"} \\ \text{"Short"} \\ \text{"Instant"} \end{pmatrix}$$

$$(\textit{Permanent} \quad \textit{Long} \quad \textit{Medium} \quad \textit{Short} \quad \textit{Instant}) = (0 \quad 1 \quad 2 \quad 3 \quad 4)$$

Wood data

Read Table of values relating to wood based materials from Excel Spreadsheet

Materials = Wood Tables.xls

See Appendix B Wood properties

Enumerated column names for the materials table

$(name\ \ type\ \ thick) = (0\ \ 1\ \ 2)$

$(f_{m.0.k}\ \ f_{m.90.k}\ \ f_{c.0.k}\ \ f_{c.90.k}\ \ f_{t.0.k}\ \ f_{t.90.k}\ \ f_{v.k}\ \ f_{r.0.k}\ \ f_{r.90.k}) = (3\ \ 4\ \ 5\ \ 6\ \ 7\ \ 8\ \ 9\ \ 10\ \ 11)$

$(G_{mean}\ \ E_{0.mean}\ \ E_{90.mean}\ \ E_{tc.0.mean}\ \ E_{tc.90.mean}\ \ E_{0.05}) = (12\ \ 13\ \ 14\ \ 15\ \ 16\ \ 17)$

$(\rho_k\ \ \rho_{mean}\ \ k_{h.d}\ \ k_{h.s}\ \ k_{h.max}) = (18\ \ 19\ \ 20\ \ 21\ \ 22)$

Table of design factors based on material and service class.
Columns
0 = Material Type + 10 * Service class
1 .. 5 = k_{mod} for each duration
6 = k_{def}
7 = γ_M

ktable =

	0	1	2	3	4	5	6	7
0	11	0.6	0.7	0.8	0.9	1.1	0.8	1.2
1	12.1	0.3	0.45	0.65	0.85	1.1	2.25	1.2
2	12.2	0.5	0.5	0.7	0.9	1.1	1.5	1.2
3	13.1	0.3	0.45	0.65	0.85	1.1	2.25	1.3
4	13.3	0.4	0.5	0.7	0.9	1.1	1.5	1.3
5	14.1	0.6	0.7	0.8	0.9	1.1	0.6	1.3
6	15.1	0.6	0.7	0.8	0.9	1.1	0.6	1.3
7	16.1	0.6	0.7	0.8	0.9	1.1	0.6	1.25
8	16.2	0.6	0.7	0.8	0.9	1.1	0.6	1.25
9	17	0.6	0.7	0.8	0.9	1.1	0.6	1.2
10	21	0.6	0.7	0.8	0.9	1.1	1	1.2
11	22.1	0	0	0	0	0	0	1.2
12	22.2	0.3	0.4	0.55	0.7	0.9	2.25	1.2
13	23.1	0	0	0	0	0	0	1.3
14	23.2	0.2	0.3	0.45	0.6	0.8	3	1.3
15	23.3	0.3	0.4	0.55	0.7	0.9	2.25	1.3
16	24	0.6	0.7	0.8	0.9	1.1	0.8	1.3
17	26	0.6	0.7	0.8	0.9	1.1	0.8	1.25
18	27	0.6	0.7	0.8	0.9	1.1	0.8	1.2
19	31	0.5	0.55	0.65	0.7	0.9	2.5	1.2
20	32	0	0	0	0	0	0	1.2
21	33	0	0	0	0	0	0	1.3
22	34	0.5	0.55	0.65	0.7	0.9	2	1.3
23	36	0.5	0.55	0.65	0.7	0.9	2	1.25
24	37	0.5	0.55	0.65	0.7	0.9	2	1.2

$(k_{mod}\ \ k_{def}\ \ \gamma_M) = (1\ \ 6\ \ 7)$

Function to extract value from ktable
Returns value from row which is closest match
i.e mat=4.2 and c=2 will return value from row 16
Parameters
mat = material name or row number
sc = strength class
c = column number required

get_k(mat, sc, c) =

$mat \leftarrow match\left(mat, Materials^{\langle 0 \rangle}\right)_0$ if IsString(mat)

$m \leftarrow sc \times 10 + Materials_{mat,\, \textit{type}}$

for $r \in 1 .. rows(ktable) - 1$

  (break) if $ktable_{r,0} > m$

$r \leftarrow r - 1$

return 0 if $r = rows(ktable) - 1$

$ktable_{r,c}$

Return a TimberCharacteristic from the wood properties table
Parameters
m=material name or row number
c=charateristic required (column number)

Tc(m, c) =

$m \leftarrow match\left(m, Materials^{\langle 0 \rangle}\right)_0$ if IsString(m)

$Materials_{m,c}$

Site design values

Wind forces

See Spreadsheet 'Tables'

Site altitude $Alt = 12$

Principal direction $Dir = 50$

Base wind velocity $v_{b.map} = 23\frac{m}{s} = 51.45 \cdot \frac{mi}{hr}$

Building height $h = 6m$ constant for all faces

Altitude Factor $c_{alt} = 1 + 0.001Alt = 1.01$

Seasonal factor $c_{season} = 1.0$ Permanent design

Fundamental wind velocity $v_{b.0} = v_{b.map} \cdot c_{alt} = 23.28\frac{m}{s}$

Terrain orthography $c_o = 1.0$ No significant orthography

Structural factors $c_s = 1.0$ $c_d = 1.0$ As h<15m

Building width $b = \begin{pmatrix} 21.7 \\ 14.4 \\ 21.7 \\ 14.4 \end{pmatrix} \cdot m$ $\begin{pmatrix} front \\ left \\ back \\ right \end{pmatrix}$

Building depth $d = \begin{pmatrix} 14.4 \\ 21.7 \\ 14.4 \\ 21.7 \end{pmatrix} \cdot m$ $\begin{pmatrix} front \\ left \\ back \\ right \end{pmatrix}$

Direction factors; adjacent buildings: Dist to, Average height; Displacement height; Wind height

$$\begin{pmatrix} front \\ left \\ back \\ right \end{pmatrix} \quad c_{dir} = \begin{pmatrix} 0.77 \\ 0.86 \\ 0.98 \\ 0.98 \end{pmatrix} \quad \begin{pmatrix} 20m \\ 25m \\ 20m \\ 200m \end{pmatrix} \quad h_{ave} = \begin{pmatrix} 6m \\ 10m \\ 8m \\ 0m \end{pmatrix} \quad h_{dis} = \begin{pmatrix} \min(1.2 \cdot h_{ave_0} - 0.2m, 0.6 \cdot h) \\ \min(1.2 \cdot h_{ave_1} - 0.2m, 0.6 \cdot h) \\ \min(1.2 \cdot h_{ave_2} - 0.2m, 0.6 \cdot h) \\ 0m \end{pmatrix} \quad z = h - h_{dis} = \begin{pmatrix} 2.4 \\ 2.4 \\ 2.4 \\ 6 \end{pmatrix} m$$

Roughness, Turbulence and Exposure values from Figures NA-3 to NA-8 from NA to EC1-1-4 using z for each face.

Distance to		Terian roughness		Wind turbulence		Exposure Factor	
Sea	Town edge	Sea	Town edge	Sea	Town edge	Sea	Town edge

$$\begin{pmatrix} \text{front} \\ \text{left} \\ \text{back} \\ \text{right} \end{pmatrix} \begin{pmatrix} 2km \\ 2km \\ 30km \\ 8km \end{pmatrix} \begin{pmatrix} 2km \\ 1.3km \\ 0.8km \\ 0.8km \end{pmatrix} c_r = \begin{pmatrix} 0.8 \\ 0.8 \\ 0.76 \\ 0.95 \end{pmatrix} c_{r.T} = \begin{pmatrix} 0.61 \\ 0.62 \\ 0.63 \\ 0.76 \end{pmatrix} I_{v.flat} = \begin{pmatrix} 0.208 \\ 0.208 \\ 0.208 \\ 0.184 \end{pmatrix} k_{I.T} = \begin{pmatrix} 1.8 \\ 1.8 \\ 1.8 \\ 1.65 \end{pmatrix} c_e = \begin{pmatrix} 1.7 \\ 1.7 \\ 1.55 \\ 2.17 \end{pmatrix} c_{e.T} = \begin{pmatrix} 0.69 \\ 0.71 \\ 0.73 \\ 0.87 \end{pmatrix}$$

Calculate for each face $f = \mathit{front} .. \mathit{right}$

Basic wind velocity $v_{b_f} = v_{b.0} \cdot c_{dir_f} \cdot c_{season}$ $v_b = \begin{pmatrix} 17.92 \\ 20.02 \\ 22.81 \\ 22.81 \end{pmatrix} \frac{m}{s}$

Mean wind velocity $v_{m_f} = c_{r_f} \cdot c_{r.T_f} \cdot c_o \cdot v_{b_f}$ $v_m = \begin{pmatrix} 8.75 \\ 9.93 \\ 10.92 \\ 16.47 \end{pmatrix} \frac{m}{s}$

Turbulence intensity $I_{v_f} = I_{v.flat_f} \cdot k_{I.T_f}$ $I_v = \begin{pmatrix} 0.37 \\ 0.37 \\ 0.37 \\ 0.3 \end{pmatrix}$

Basic velocity pressure $q_{b_f} = 0.5 \cdot 1.226 \cdot \frac{kg}{m^3} \cdot \left(v_{b_f}\right)^2$ $q_b = \begin{pmatrix} 197 \\ 246 \\ 319 \\ 319 \end{pmatrix} \cdot \frac{N}{m^2}$

Peak velocity pressure $q_{p_f} = c_{e_f} \cdot c_{e.T_f} \cdot q_{b_f}$ $q_p = \begin{pmatrix} 231 \\ 296 \\ 361 \\ 602 \end{pmatrix} \cdot \frac{N}{m^2}$

External wind pressures

From Table NA.4 in EC1-1-4NA

External pressure coefficients

$$c_{pe.10.w} = \begin{array}{c} \begin{array}{ccccc} A & B & C & D & E \end{array} \\ \begin{pmatrix} -1.2 & -0.8 & -0.5 & 0.8 & -0.5 \\ -1.2 & -0.8 & -0.5 & 0.8 & -0.5 \\ -1.2 & -0.8 & -0.5 & 0.8 & -0.5 \\ -1.2 & -0.8 & -0.5 & 0.8 & -0.5 \end{pmatrix} \end{array} \quad \text{given} \quad \frac{h}{d} = \begin{pmatrix} 0.42 \\ 0.28 \\ 0.42 \\ 0.28 \end{pmatrix}$$

For each zone $z = \mathit{A} .. \mathit{E}$

External wind pressure on walls

$$w_{e.w_{f,z}} = q_{p_f} \cdot c_{pe.10.w_{f,z}} \qquad w_{e.w} = \begin{array}{c} \begin{array}{ccccc} A & B & C & D & E \end{array} \\ \begin{pmatrix} -0.28 & -0.18 & -0.12 & 0.18 & -0.12 \\ -0.36 & -0.24 & -0.15 & 0.24 & -0.15 \\ -0.43 & -0.29 & -0.18 & 0.29 & -0.18 \\ -0.72 & -0.48 & -0.3 & 0.48 & -0.3 \end{pmatrix} \end{array} \cdot \frac{kN}{m^2}$$

Internal wind pressures

opening area due to leakage

$$a_{o.l} = h \cdot b \cdot 0.0004 = \begin{pmatrix} 0.05 \\ 0.03 \\ 0.05 \\ 0.03 \end{pmatrix} m^2$$

doors

$$a_{o.d} = \begin{pmatrix} 0.9m \cdot 2m \\ 1.15m \cdot 2m \\ 0m^2 \\ 0.9m \cdot 2m \end{pmatrix} = \begin{pmatrix} 1.8 \\ 2.3 \\ 0 \\ 1.8 \end{pmatrix} m^2$$

no opening windows

$$a_{o.w} = \begin{pmatrix} 0 \\ 0 \\ 0 \\ 0 \end{pmatrix}$$

total open areas

$$a_{o_f} = a_{o.l_f} + a_{o.d_f} + a_{o.w_f} \qquad a_o = \begin{pmatrix} 1.85 \\ 2.33 \\ 0.05 \\ 1.83 \end{pmatrix} m^2$$

opening ratios

$$\mu = \begin{pmatrix} \dfrac{a_{o_1} + a_{o_2} + a_{o_3}}{\sum a_o} \\ \dfrac{a_{o_0} + a_{o_2} + a_{o_3}}{\sum a_o} \\ \dfrac{a_{o_0} + a_{o_1} + a_{o_3}}{\sum a_o} \\ \dfrac{a_{o_0} + a_{o_1} + a_{o_2}}{\sum a_o} \end{pmatrix} \qquad \mu = \begin{pmatrix} 0.695 \\ 0.6156 \\ 0.9914 \\ 0.6979 \end{pmatrix}$$

From EC1-1-4 Figure 7.13 for h/d>1 $\quad c_{pi.1}(u) = \max[0.35 - (u - 0.33) \times 1.37, -0.5]$

From EC1-1-4 Figure 7.13 for h/d<0.25 $\quad c_{pi.25}(u) = \max[0.35 - (u - 0.33) \times 1.14, -0.3]$

$$c_{pi_f} = \begin{array}{|l} r \leftarrow \dfrac{h}{d_f} \\ c_{pi.25}(\mu_f) \ \text{ if } \ r \le 0.25 \\ c_{pi.1}(\mu_f) \ \text{ if } \ r > 1 \\ c_{pi.1}(\mu_f) + \left(c_{pi.25}(\mu_f) - c_{pi.1}(\mu_f)\right) \cdot \dfrac{(r - 0.25)}{0.75} \ \text{ otherwise} \end{array}$$

$$c_{pi} = \begin{pmatrix} -0.13 \\ -0.04 \\ -0.46 \\ -0.15 \end{pmatrix}$$

Internal wind pressure

$$w_{i_f} = q_{p_f} \cdot c_{pi_f} \qquad w_i = \begin{pmatrix} -0.03 \\ -0.01 \\ -0.16 \\ -0.09 \end{pmatrix} \cdot \frac{kN}{m^2}$$

Wind forces on the walls

For each zone

$$z = \boldsymbol{A} .. \boldsymbol{E}$$

Net Wind pressure on walls

$$w_{w_{f,z}} = w_{e.w_{f,z}} - w_{i_f}$$

$$w_w = \begin{array}{c} \begin{array}{ccccc} A & B & C & D & E \end{array} \\ \begin{pmatrix} -0.25 & -0.15 & -0.09 & 0.22 & -0.09 \\ -0.34 & -0.23 & -0.14 & 0.25 & -0.14 \\ -0.27 & -0.12 & -0.02 & 0.45 & -0.02 \\ -0.63 & -0.39 & -0.21 & 0.57 & -0.21 \end{pmatrix} \end{array} \cdot \frac{kN}{m^2}$$

Racking force on the walls

Wall height

$$h_w = 3m$$

Net pressure coefficient from EC1-1-4NA Table NA.4 Note f

$$c_{p_f} = \begin{array}{|l} r \leftarrow \dfrac{h}{d_f} \\ 1.1 + 0.1243 \cdot \ln(r) \ \text{ if } \ 5 > r > 1 \\ (1.1 + 0.2164 \cdot \ln(r)) \ \text{ if } \ 1 \ge r > 0.25 \\ c_{pe.10.w_{f,\boldsymbol{D}}} - c_{pe.10.w_{f,\boldsymbol{E}}} \ \text{ otherwise} \end{array}$$

$$c_p = \begin{pmatrix} 0.91 \\ 0.82 \\ 0.91 \\ 0.82 \end{pmatrix}$$

Racking forces on building

$$F_{rack_f} = c_s \cdot c_d \cdot h_w \cdot b_f \cdot q_{p_f} \cdot c_{p_f} \qquad F_{rack} = \begin{pmatrix} 13.7 \\ 10.5 \\ 21.4 \\ 21.4 \end{pmatrix} \cdot kN$$

Wind forces on the roof

Building has full hipped roof structure.

External pressure coefieciens from Table NA.8 in EC1-1-4NA. Values interpolated from table.

For zones

$z = F .. N$ and windward roof angle $a = 0..4$

$c_{pe.10.r.n} =$

-0.43	-0.43	-0.17	-0.6	-1.22	-0.75	-1.01	-0.6	-0.49
-0.4	-0.4	-0.16	-0.6	-1.18	-0.72	-1.02	-0.6	-0.48
-0.17	-0.17	-0.07	-0.6	-0.9	-0.53	-1.07	-0.6	-0.43
0	0	0	-0.6	-0.7	-0.4	-1.1	-0.6	-0.4
0	0	0	-0.7	-0.6	-0.3	-1.2	-0.7	-0.6

given windward roof angles of $\begin{pmatrix} 32 \\ 33 \\ 40 \\ 45 \\ 60 \end{pmatrix}$ degrees

$c_{pe.10.r.p} =$

0.8	0.51	0.44	-0.6	-1.22	-0.75	0	0	0
0.8	0.52	0.46	-0.6	-1.18	-0.72	0	0	0
0.8	0.57	0.6	-0.6	-0.9	-0.53	0	0	0
0.8	0.6	0.7	-0.6	-0.7	-0.4	0	0	0
0.8	0.8	0.8	-0.7	-0.6	-0.3	-1.2	-0.7	-0.6

Not all roof angles are windward to all wind directions so a matrix of relevence is needed

$$awd = \begin{pmatrix} 1 & 0 & 0 & 0 & 1 \\ 0 & 1 & 1 & 0 & 0 \\ 1 & 0 & 1 & 1 & 0 \\ 0 & 1 & 1 & 0 & 0 \end{pmatrix}$$

External wind pressure on roof zones (negative)

$$w_{e.r.n_{a\cdot 4+f,\,z}} = q_{p_f} \cdot c_{pe.10.r.n_{a,\,z}} \cdot awd_{f,\,a}$$

External wind pressure on roof zones (positive)

$$w_{e.r.p_{a\cdot 4+f,\,z}} = q_{p_f} \cdot c_{pe.10.r.p_{a,\,z}} \cdot awd_{f,\,a}$$

Maximum uplift on the roof surface

$$F_{r.up.max} = -\min\left(w_{e.r.n}\right) = 0.71 \cdot \frac{kN}{m^2}$$

Maximum down force on the roof surface

$$F_{r.down.max} = \max\left(w_{e.r.p}\right) = 0.48 \cdot \frac{kN}{m^2}$$

Snow

From Figure NA.1 in the National Annex to EC1 Part 3 the site is in Zone 3

Ground snow load

$$Q_{snow.k} = 0.5 \frac{kN}{m^2}$$

Roof design

The roof of the building is fully hipped and has a skylight replacing the full length of the central ridge. The roof covering will be cedar shingles but the design will allow for replacement with standard tiling (extra 40kg/m^2). Central skylight will be structural and supported by columns from the foundations. All roof framing will be formed from box beams to allow a insulation depth of 400mm.

Calculations for the box beams are taken from chapter 7 of Reference book

Drawings **6 - Roof design**
7 - Roof and Wall wind load zones

Design conditions

Service class

Overall service class can be set to 1 (warm roof) but for safety will be set to $\text{Class} = 2$

Roof loads

Dead load of the roof has a minimum value of $G_{roof.min.k} = 0.53 \cdot kN \cdot m^{-2}$ with cedar shingles

and a maximum value of $G_{roof.max.k} = 0.93 kN \cdot m^{-2}$ for clay/concrete tiles

this value include self weight of rafters, ceiling and insulation.

The skylight has a dead load of $G_{skyl.k} = 0.65 kN \cdot m^{-2}$ including quad glazing and framing

psi values for roof loads of dead load,snow and wind

$$\psi s = \begin{pmatrix} 1 & \textit{Permanent} & 1 \\ \psi val(\text{"S"}, 0) & \textit{Short} & \psi val(\text{"S"}, 2) \\ \psi val(\text{"W"}, 0) & \textit{Instant} & \psi val(\text{"W"}, 2) \end{pmatrix} = \begin{pmatrix} 1 & 0 & 1 \\ 0.5 & 3 & 0 \\ 0.5 & 4 & 0 \end{pmatrix}$$

Uplift forces on roof

Tiling batten material

$\text{Material} = \text{"Softwood C14"}$

Density of tiling batten $\rho_k = Tc(\text{Material}, \rho_k) = 290$

k_{mod} for batten $k_{mod} = get\_k(\text{Material}, 3, k_{mod} + \textit{Instant}) = 0.9$ Class 3 with Instant duration

Safety factor $\gamma_M = 1.3$ For metal dowel fixings

Check fixing strength for the roof shingles

Number of tiles per m^2 $n_t = \dfrac{110}{9.3m^2} = 11.83\dfrac{1}{m^2}$ (approx. 110 shingles per 100 square feet)

Fixings per tile $n_f = 2$

Shingles will be held by 16 gauge Stainless steel staples

Leg diameter $d = \sqrt{1.6 \cdot 1.4} \cdot mm = 1.5 \cdot mm$

Leg length $l_{leg} = 38mm$

Tile thickness $t_t = 12mm$

Tile self weight $G_k = 20\frac{kg}{m^2} \cdot g = 196.13 \cdot \frac{N}{m^2}$

Pointside pentration $t_p = l_{leg} - t_t = 26 \cdot mm$ $OKifLT\left(14d, t_p\right) = "O.K."$

Characteristic Axial withrawal strenth per leg $f_{ax.Rk} = 20 \times 10^{-6} \cdot \rho_k^{\,2} \cdot N \cdot mm^{-2} \cdot d \cdot t_p = 65.45\,N$

Withdrawal resistance of a staple $F_{ax.Rd} = \frac{2f_{ax.Rk} \cdot k_{mod}}{\gamma_M} = 90.63\,N$

Uplift resistance for tiling $F_{t.up} = n_t \cdot n_f \cdot F_{ax.Rd} + G_k = 2.34 \cdot \frac{kN}{m^2}$ $OKifLT\left(F_{r.up.max}, F_{t.up}\right) = "O.K."$

Tiling battens to counter battens

Tiling batten spacing $d_s = 125mm$

Counter batten spacing $d_c = 612mm$

Batten junctions per m² $n_j = \frac{1}{d_s} \cdot \frac{1}{d_c} = 13.07 \cdot m^{-2}$

Thickness of both battens $t = 25mm$

Nail length $l_{nail} = 50mm$ Using Hot dipped Ring shank nail

Nail diameter $d = 2.8mm$

Nail pointside penetration $t_2 = l_{nail} - t = 25 \cdot mm$

As both penetration distances are the same pointside withdrawal will be the lesser value

nail penetration for full strength $8 \cdot d = 22.4 \cdot mm$ Greater than t_2 so full withdrawal resistance allowed

Characteristic withdrawal resitance $f_{ax.k} = 7.79 \cdot \left(\frac{\rho_k}{350}\right)^2 \cdot N \cdot mm^{-2} = 5.35 \cdot N \cdot mm^{-2}$ From data sheet (350 is density of test material)

Characteristic withdrawal strength $F_{ax.Rk} = f_{ax.k} \cdot d \cdot t_2 = 374\,N$

Axial withdrawl strength of fixings $F_{ax.Rd} = \frac{F_{ax.Rk} \cdot k_{mod}}{\gamma_M} = 259\,N$

Uplift resitance of battens $F_{tb.up} = F_{ax.Rd} \cdot n_j = 3.39 \cdot \frac{kN}{m^2}$ $OKifLT\left(F_{r.up.max}, F_{tb.up}\right) = "O.K."$

Rafter design

See Drawings **8 - Roof load distribution**
9 - Roof construction details

Spreadsheet **Rafters**
Rafter Max M&V

In the Rafters spreadsheet the following calculations are performed for each rafter in the building

Column	Value	Alternative value for hip/valley rafters
A & B	Wall/Roof letter and rafter number	curent and ajacent wall letters
C	Distance of the rafter from left end of wall when viewed from outside	
D	Roof angle (θ_1)	
E	Radian value of angle	angle of ajacent roof (θ_2)
F	Length of eaves projection (angled length) to give level facia	plan span of current roof (l_1)
G	Plan span of the rafter excluding eaves	plan span of ajacent roof (l_2)
H	Snow coeficient μ	
I & K	Area of the roof imposed on the exterior wall angled/eaves/plan	$l_1 l_2/4\cos\theta_1 + l_1 l_2/4\cos\theta_2$ and $l_1 l_2/2$
L	Snow load imposed on the exterior wall	
M & N	Minimum and maximum dead load imposed on the exterior wall	
O & P	Minimum and maximum total load imposed on the exterior wall including wind forces	
Q to AJ	Wind forces on each rafter for each wind direction (5 columns for each direction)	
Q	Direction of the wind for the wall this rafter is loading (relative to wall n=none) [To calculate racking loads]	
R	Roof pressure load zone	
S	Windward roof angle	
T & U	Positive and negative pressure. Looks up $c_{pe.10}$ from table using angle and zone them applies it to q_p	
AK & AL	Min and max value for wind pressure	
AM to AQ	Calculate approximate bending & shear forces in the rafters main span (slightly to large)	
AM	Angled length of the main span (clear span) (l_c)	
AN	Maximum design UDL $w = G_{roof.max.k} \times \gamma_G + Q_{snow.k} \times \gamma_Q + Q_{wind.max.k} \times \gamma_Q \times \psi_{0.wind}$ (x Rafter spacing if standard rafter)	
AO	Area of roof loading this rafter (A)	
AP	Bending moment $M = w\, l_c^2 \;/\; 8$	$M=0.128Awl_c$ from $M=0.064xl_c^2$, $W=xl_c/2$ and W=Aw as load is a variable linear load from x to 0
AQ	Shear force $V=wl_c/2$	V=2/3 Aw

In the Max M&V spreadsheet the bending and shear values are sorted into descending order so as to show the maximum values. White on Black values are for hip/valley rafters.

Standard rafters

The standard rafters are all of the same basic construction. The main span is a box beam and the eaves projection is an extension of the the top flange. Seperate calculations will be need for the two parts.

Calculate Main span

A number of the rafters have equal maximum bending moments and shear forces. These are M10-M14 and I6 and they have the following parameters.

Main span length $L_s = 4.7m$

Roof angle $\theta = 32$

Rafter spacing $R_s = 612mm$

Number of flanges $f_n = 1$

Number of webs $w_n = 2$

Beam depth $h_b = 400mm$

Beam flanges are fully restrained $k_c = 1$

Load sharing is active $k_{sys} = 1.1$

Flange

although top flange is C24 and 89mm deep the lesser values are choosen for simplicity

Flange material $Material_f$ = "Softwood C16"

Height of flange $h_f = 63mm$

Width of flange $b_f = 38mm$

Web

Web material $Material_w$ = "Plywood Canadian Softwood 12.5mm 5 ply"

Actions

Wind coeffiecient for zone H $c_{pe.10} = 0.44$ The majority of the load is from the main roof zone

Variable wind load $Q_{w.k} = q_{p_{back}} \cdot c_{pe.10} = 0.16 \cdot kN \cdot m^{-2}$

Variable snow load $Q_{s.k} = Q_{snow.k} \cdot \cos(\theta \cdot deg) \cdot 1.2 \frac{(60 - \theta)}{30} = 0.47 \cdot kN \cdot m^{-2}$

Find critical load combination

Create a list of the load combinations

$$Loads = Load\_Combos\left[\psi s, \begin{pmatrix} G_{roof.max.k} \\ Q_{s.k} \\ Q_{w.k} \end{pmatrix} \div Pa, \begin{pmatrix} \gamma_G \\ \gamma_Q \\ \gamma_Q \end{pmatrix}\right]$$

Then iterate the calculations for all combinations $c = 0 .. rows(Loads) - 1$

Load duration for this load combo $LoadDuration_c = Loads_{c, d_l}$

ULS Actions

Design load $F_{d_c} = Loads_{c, ULS} \cdot Pa \cdot R_s$

Design moment $M_d = \frac{F_d \cdot L_s^2}{8}$

Design shear force $V_d = \frac{F_d \cdot L_s}{2}$

Values for bending and shear to evaluate load combos

$$F_d = \begin{pmatrix} 0.77 \\ 0.99 \\ 1.2 \\ 0.84 \\ 1.06 \\ 1.28 \\ 0.91 \\ 1.13 \\ 1.28 \end{pmatrix} \cdot \frac{kN}{m} \qquad M_d = \begin{pmatrix} 2.12 \\ 2.72 \\ 3.33 \\ 2.32 \\ 2.92 \\ 3.53 \\ 2.52 \\ 3.13 \\ 3.53 \end{pmatrix} \cdot kN \cdot m \qquad V_d = \begin{pmatrix} 1.81 \\ 2.32 \\ 2.83 \\ 1.98 \\ 2.49 \\ 3 \\ 2.15 \\ 2.66 \\ 3 \end{pmatrix} \cdot kN$$

SLS Actions

Design load at instantaneous SLS $F_{SLS.i_c} = Loads_{c, \boldsymbol{SLS_i}} \cdot Pa \cdot R_s$

Design load at final SLS $F_{SLS.f_c} = Loads_{c, \boldsymbol{SLS_f}} \cdot Pa \cdot R_s$

Material characteristics

Flange material

Material safety factor $\gamma_{M.f} = get\_k(Material_f, Class, \gamma_M) = 1.3$

Duration modification factors for each load combo $k_{mod.f_c} = get\_k(Material_f, Class, k_{mod} + LoadDuration_c)$

Final deformation factor $k_{def.f} = get\_k(Material_f, Class, k_{def}) = 0.8$

Material design characteristics

$f_{m.f.k} = Tc(Material_f, f_{m.0.k})\ MPa$

$f_{c.0.f.k} = Tc(Material_f, f_{c.0.k})\ MPa$

$f_{t.0.f.k} = Tc(Material_f, f_{t.0.k})\ MPa$

$E_{0.mean.f} = Tc(Material_f, E_{0.mean})\ MPa$

Effective flange width $b_{f.ef} = b_f \cdot f_n = 38 \cdot mm$

Web material

Material safety factor $\gamma_{M.w} = get\_k(Material_w, Class, \gamma_M) = 1.2$

Duration modification factors for each load combo $k_{mod.w_c} = get\_k(Material_w, Class, k_{mod} + LoadDuration_c)$

Final deformation factor $k_{def.w} = get\_k(Material_w, Class, k_{def}) = 1$

Material design characteristics

$f_{v.w.k} = Tc(Material_w, f_{v.k})\ MPa$

$f_{r.w.k} = Tc(Material_w, f_{r.90.k})\ MPa$

$f_{c.90.w.k} = Tc(Material_w, f_{c.90.k})\ MPa$

$f_{t.90.w.k} = Tc(Material_w, f_{t.90.k})\ MPa$

$E_{c.90.mean.w} = Tc\left(Material_w, \boldsymbol{E_{tc.90.mean}}\right) MPa$

$G_{mean.w} = Tc\left(Material_w, \boldsymbol{G_{mean}}\right) MPa$

Web thickness $b_w = Tc\left(Material_w, \boldsymbol{thick}\right) mm = 12.5 \cdot mm$

Effective web thickness $b_{w.ef} = \begin{cases} \frac{b_w}{2} & \text{if } w_n = 1 \\ b_w & \text{otherwise} \end{cases} = 12.5 \cdot mm$

Clear height of the web $h_w = h_b - 2h_f = 0.27\,m$

Area of the web $A_w = h_b \cdot b_w \cdot w_n = 0.01\,m^2$

Material characteristics - design

Flange

Height modification $k_{h.f} = \max\left[1, \min\left[\left(\frac{Tc\left(Material_f, \boldsymbol{k_{h.d}}\right)}{h_f \div mm}\right)^{Tc\left(Material_f, \boldsymbol{k_{h.s}}\right)}, Tc\left(Material_f, \boldsymbol{k_{h.max}}\right)\right]\right] = 1$

Design characteristics $f_{m.f.d} = \frac{f_{m.f.k} \cdot k_{mod.f} \cdot k_{h.f} \cdot k_{sys}}{\gamma_{M.f}}$

$f_{t.0.f.d} = \frac{f_{t.0.f.k} \cdot k_{mod.f} \cdot k_{h.f} \cdot k_{sys}}{\gamma_{M.f}}$

$f_{c.0.f.d} = \frac{f_{c.0.f.k} \cdot k_{mod.f} \cdot k_{sys}}{\gamma_{M.f}}$

Web

$f_{t.90.w.d} = \frac{f_{t.90.w.k} \cdot k_{mod.w} \cdot k_{sys}}{\gamma_{M.w}}$

$f_{c.90.w.d} = \frac{f_{c.90.w.k} \cdot k_{mod.w} \cdot k_{sys}}{\gamma_{M.w}}$

$f_{v.w.d} = \frac{f_{v.w.k} \cdot k_{mod.w} \cdot k_{sys}}{\gamma_{M.w}}$

$f_{r.w.d} = \frac{f_{r.w.k} \cdot k_{mod.w} \cdot k_{sys}}{\gamma_{M.w}}$

Geometric properties – transformed sections

Instantaneous – transformed section properties:

Second moment of area of flanges $I_{f.ef} = \frac{b_{f.ef}}{12} \cdot \left(h_b^3 - h_w^3\right) = 1.38 \times 10^8 \cdot mm^4$

Transformed web thickness (into flange)

$b_{w.tfd.i} = b_w \cdot \frac{E_{c.90.mean.w}}{E_{0.mean.f}} = 6.19 \cdot mm$

Second moment of area of web $I_{ef.w.i} = \frac{b_{w.tfd.i}}{12} \cdot h_b^3 = 3.3 \times 10^{-5} m^4$

Instantaneous second moment of area of the transformed section

$$I_{ef.i} = I_{ef.w.i} + I_{f.ef} = 1.71 \times 10^{-4} m^4$$

Final – transformed section properties:

of web thickness $b_{w.tfd.f_c} = b_{w.tfd.i} \cdot \frac{1 + Loads_{c,\psi_2} \cdot k_{def.f}}{1 + Loads_{c,\psi_2} \cdot k_{def.w}}$

Second moment of area of web $I_{ef.w.f} = \frac{b_{w.tfd.f}}{12} \cdot h_b^3$

Final second moment of area of the transformed section $I_{ef.f} = I_{ef.w.f} + I_{f.ef}$

Bending stress check in the flanges and the web

Because the mean modulus of elasticity of the flange material is greater than that of the web,only check the stresses in the flanges at the final deformation condition and those in the web at the instantaneous condition.

Stress in flange due to bending – final condition:

Bending stress in top and bottom flange $\sigma_{m.max.f.d} = \frac{M_d}{I_{ef.f}} \cdot \frac{h_b}{2}$

Test against bending strength $r_{b.f} = \max\left(\frac{\sigma_{m.max.f.d}}{f_{m.f.d}}\right) = 0.27$

Stress in web due to bending – instananeous condition:

Bending stress in the web $\sigma_{m.w.d} = \frac{M_d}{I_{ef.i}} \cdot \frac{h_b}{2} \cdot \left(\frac{E_{c.90.mean.w}}{E_{0.mean.f}}\right)$

Test against bending strength in compression $r_{b.w.c} = \max\left(\frac{\sigma_{m.w.d}}{f_{c.90.w.d}}\right) = 0.24$

Test against bending strength in tension $r_{b.w.t} = \max\left(\frac{\sigma_{m.w.d}}{f_{t.90.w.d}}\right) = 0.32$

Stress in the flange due to axial stress – final condition:

Axial stress in top and bottom flange $\sigma_{ax.f.d} = \frac{M_d}{I_{ef.f}} \cdot \left(\frac{h_b}{2} - \frac{h_f}{2}\right)$

Test against axial strength in compression $r_{ax.f.c} = \max\left(\frac{\sigma_{ax.f.d}}{f_{c.0.f.d} \cdot k_c}\right) = 0.25$

Test against axial strength in tension $r_{ax.f.t} = \max\left(\frac{\sigma_{ax.f.d}}{f_{t.0.f.d}}\right) = 0.36$

Buckling and shear stress check in the web

Buckling condition in the web in EC5

$$ratio = \frac{h_w}{b_w} = 21.92$$

Maxiumum value of the ratio is 70

$$r_{b.w} = \frac{ratio}{70} = 0.31 \qquad Check(r_{b.w}) = \text{"O.K."}$$

Shear strength of the web

Design shear force able to be taken by each web; EC5, equation (9.9))

$$F_{v.w.Ed} = \begin{vmatrix} b_w \cdot h_w \cdot \left(1 + \frac{h_f}{h_w}\right) \cdot f_{v.w.d} & \text{if } ratio \le 35 \\ 35 b_w^{\,2} \left(1 + \frac{h_f}{h_w}\right) \cdot f_{v.w.d} & \text{otherwise} \end{vmatrix}$$

Design shear force able to be taken by the beam

$$F_{v.Ed} = F_{v.w.Ed} \cdot w_n$$

Test shear force in web

$$r_{v.w} = \max\left(\frac{V_d}{F_{v.Ed}}\right) = 0.12$$

Shear strength of the glued joint between the web and the flanges

First moment of area of a flange about the NA,

$$S_f = b_{f.ef} \cdot h_f \cdot \left(\frac{h_b}{2} - \frac{h_f}{2}\right) = 403.39 \cdot cm^3$$

Total length of the glue line in the flange

$$l_g = 2h_f = 0.13\,m$$

Shear stress in the glue line (instant.)

$$\tau_{mean.d.i} = \frac{V_d \cdot S_f}{I_{ef.i} \cdot l_g}$$

Shear stress in the glue line (final)

$$\tau_{mean.d.f} = \frac{V_d \cdot S_f}{I_{ef.f} \cdot l_g}$$

EC5 takes into account the effect of stress concentrations at the web/flange interface in the vicinity of position of the join to web when the height of the flange is greater than $4b_{w.ef}$

$$f_{v.90.d} = \begin{vmatrix} f_{r.w.d} & \text{if } h_f \le 4b_{w.ef} \\ f_{r.w.d} \cdot \left(\frac{4b_{w.ef}}{h_f}\right)^{0.8} & \text{otherwise} \end{vmatrix}$$

$$r_{v.g} = \max\left(\frac{\tau_{mean.d.i}}{f_{v.90.d}}, \frac{\tau_{mean.d.f}}{f_{v.90.d}}\right) = 0.12$$

Deflection of the beam at the SLS

At the instantaneous condition:

Instantaneous deflection at mid-span

$$\mu_{inst} = \max\left(\frac{5 \cdot F_{SLS.i} \cdot L_s^{4}}{384 \cdot E_{0.mean.f} \cdot I_{ef.i}} + \frac{F_{SLS.i} \cdot L_s^{2}}{8 \cdot G_{mean.w} \cdot A_w}\right) = 4.81 \cdot mm$$

Allowable Instantaneous deflection at mid-span

$$\mu_{inst.allow} = \frac{L_s}{300} = 15.67 \cdot mm$$

$$r_{d.i} = \frac{\mu_{inst}}{\mu_{inst.allow}} = 0.31$$

$$\text{Check}\left(r_{d.i}\right) = \text{"O.K."}$$

At the final deformation condition:

transform of web thickness

$$b_{w.tfd.f} = b_{w.tfd.i} \cdot \frac{1 + k_{def.f}}{1 + k_{def.w}} = 5.57 \cdot mm$$

Second moment of area of web

$$I_{ef.w.f} = \frac{b_{w.tfd.f}}{12} \cdot h_b^{3} = 2.97 \times 10^{7} \cdot mm^{4}$$

Second moment of area of beam

$$I_{ef.f} = I_{ef.w.f} + I_{f.ef} = 1.67 \times 10^{-4} m^{4}$$

Final deflection at mid-span

$$\mu_{final} = \max\left[\frac{5 \cdot F_{SLS.f} \cdot L_s^{4}\left(1 + k_{def.f}\right)}{384 \cdot E_{0.mean.f} \cdot I_{ef.f}} + \frac{F_{SLS.f} \cdot L_s^{2}\left(1 + k_{def.w}\right)}{8 \cdot G_{mean.w} \cdot A_w}\right] = 5.6 \cdot mm$$

Final Instantaneous deflection at mid-span

$$\mu_{final.allow} = \frac{L_s}{250} = 18.8 \cdot mm$$

$$r_{d.f} = \frac{\mu_{final}}{\mu_{final.allow}} = 0.3$$

$$\text{Check}\left(r_{d.f}\right) = \text{"O.K."}$$

Results of calculation

Maximum utility rate

$$\max\left(r_{b.f}, r_{b.w.c}, r_{b.w.t}, r_{ax.f.c}, r_{ax.f.t}, r_{b.w}, r_{v.w}, r_{v.g}, r_{d.i}, r_{d.f}\right) = 36 \cdot \%$$

Calculate eaves

The load on the eaves is different in both dead and live loads. Dead load is reduced by being only one flange and tiling. The wind load is a function of the combined effect of the wind on the topside and the underside. Underside load is equal to wind force on the wall below.

Spreadsheet Rafter Eaves

Looking at the Rafter Eaves Spreadsheet it shows that the eaves on roof E at the corner of E&F has the most load on it when the wind is from the right.

Eves length $L_e = 0.8m$

Roof angle $\theta = 32$

Beam width $b = 38 \cdot mm$

Beam depth $h = 89 \cdot mm$

Beam material Material = "Softwood C24"

Section modulus $W_y = \frac{h^2 \cdot b}{6} = 50.17 \cdot cm^3$

Dead load on the eaves $G_{eaves.max.k} = 0.63kN \cdot m^{-2}$

Wind coeffiecient for zone A $c_{pe.10.A} = c_{pe.10.w_{right, A}} = -1.2$ The wall zone

Wind coeffiecient for zone L $c_{pe.10.L} = 0.0$ The roof zone

Wind load on the eaves

$$Q_{w.k} = q_{p_{right}} \cdot \left(c_{pe.10.L} - c_{pe.10.A}\right) = 0.72 \cdot kN \cdot m^{-2}$$

Variable snow load $Q_{s.k} = Q_{snow.k} \cdot \cos(\theta \cdot deg) \cdot 1.2 \frac{(60 - \theta)}{30} = 0.47 \cdot kN \cdot m^{-2}$

Facia and gutters provide a point load of $G_{fg.k} = 0.05kN \cdot m^{-1}$

and a variable load of $Q_{fg.k} = 0.15kN \cdot m^{-1}$ (full gutters)

The variable action can be ignored as it produces less moment or shear than the snow load and is mutually exclusive

Find critical load combination

Generate load combinations

$$Loads = Load\_Combos\left[\psi s, \begin{pmatrix} G_{eaves.max.k} \\ Q_{s.k} \\ Q_{w.k} \end{pmatrix} \div Pa, \begin{pmatrix} \gamma_G \\ \gamma_Q \\ \gamma_Q \end{pmatrix}\right]$$

Then iterate the calculations for all combinations $c = 0 .. rows(Loads) - 1$

Load duration for this load combo $LoadDuration_c = Loads_{c, d_l}$

ULS Actions

Design load due to critical load combination $F_{d_c} = Loads_{c, 0} \cdot Pa \cdot R_s$

Design load of the facia $F_{fg.d} = G_{fg.k} \cdot R_s \cdot \gamma_G = 41.31\,N$

Design moment $M_d = \frac{F_d \cdot L_e^2}{2} + F_{fg.d} \cdot L_e$

Design shear force $V_d = F_d \cdot L_e + F_{fg.d}$

Material characteristics

Material safety factor $\gamma_M = get\_k(Material, Class, \gamma_M) = 1.3$

Duration modification factors for each load combo $k_{mod_c} = get\_k(Material, Class, k_{mod} + LoadDuration_c)$

Final deformation factor $k_{def} = get\_k(Material, Class, k_{def}) = 0.8$

Material characteristics $f_{m.k} = Tc(Material, f_{m.0.k})\,MPa$

$f_{v.k} = Tc(Material, f_{v.k})\,MPa$

Height modification $k_h = max\left[1, min\left[\left(\frac{Tc(Material, k_{h.d})}{h \div mm}\right)^{Tc(Material, k_{h.s})}, Tc(Material, k_{h.max})\right]\right] = 1.11$

Bending strength

Design bending stress $\sigma_{m.y.d} = \frac{M_d}{W_y}$

Design bend strength $f_{m.d} = \frac{f_{m.k} \cdot k_{mod} \cdot k_h \cdot k_{sys}}{\gamma_M}$

Test Bending Strength $r_b = \frac{\sigma_{m.y.d}}{f_{m.d}}$ $Check(r_b) = "O.K."$

Shear strength

Design shear stress $\tau_{v.d} = \frac{3}{2} \cdot \frac{V_d}{b \cdot h}$

Design shear strength $f_{v.d} = \frac{f_{v.k} \cdot k_{mod} \cdot k_{sys}}{\gamma_M}$

Test shear strength $r_v = \frac{\tau_{v.d}}{f_{v.d}}$ $Check(r_v) = "O.K."$

Deflection

In this instance deflection is not important and can be ignored.

Hip and valley rafters

The hip and valley rafters are all of the same basic construction, the exact depth will depend on the adjacent roof angles. The main span is a box beam and any eaves projection is an extension of the the top flange. Any eves will very little load on them due to the geometery.

Calculate Main span

The rafter which has the maximum bending moment and shear force is from the junction of walls I &J to point e on the skylight when the wind is from the right. All loads on the rafter are variable linear load declining from IJ to e. The load is equivalent to half of the total area enclosing the rafter.

Roof angle left $\theta_l = 32$

Roof angle right $\theta_r = 33$

Roof plan span left $L_l = 3.99m$

Roof plan span right $L_r = 3.71m$

Area of roof bearing on the rafter $$A_r = \frac{L_l \cdot L_r}{4\cos(\theta_l \cdot deg)} + \frac{L_l \cdot L_r}{4\cos(\theta_r \cdot deg)} = 8.78\,m^2$$

Main span length $$L_s = \sqrt{\left(\frac{L_l}{\cos(\theta_l \cdot deg)}\right)^2 + L_r^2} = 5.99\,m$$

Box beam design

Number of flanges $f_n = 1$

Number of webs $w_n = 2$

Beam depth $h_b = 425mm$

Beam flanges are fully restrained $k_c = 1$

Load sharing is not available $k_{sys} = 1.0$

Flange

Flange material $Material_f =$ "Kerto S Edgewise"

Height of flange $h_f = 90mm$ nominal 100mm but allow shaping for intersections

Width of flange $b_f = 39mm$

Web

Web material $Material_w =$ "Plywood Canadian Softwood 12.5mm 5 ply"

Actions

Wind coeffiecient for zone M $c_{pe.10.M} = 0.0$ The roof zone paralell to wind

Wind coeffiecient for zone H $c_{pe.10.H} = 0.46$ The roof zone perpendicular to wind

Wind load on the eaves $$Q_{w.k} = \frac{q_{p_{right}} \cdot (c_{pe.10.M} + c_{pe.10.H})}{2} = 0.14 \cdot kN \cdot m^{-2}$$

Variable snow load $Q_{s.k} = Q_{snow.k} \cdot \cos\left[\left(\frac{\theta_l + \theta_r}{2}\right) \cdot deg\right] \cdot 1.2 \frac{\left(60 - \frac{\theta_l + \theta_r}{2}\right)}{30} = 0.46 \cdot kN \cdot m^{-2}$

Find load combinations

Generate load combinations $Loads = Load\_Combos\left[\psi s, \begin{pmatrix} G_{roof.max.k} \\ Q_{s.k} \\ Q_{w.k} \end{pmatrix} \div Pa, \begin{pmatrix} \gamma_G \\ \gamma_Q \\ \gamma_Q \end{pmatrix}\right]$

Then iterate the calculations for all combinations $c = 0 .. rows(Loads) - 1$

Load duration for this load combo $LoadDuration_c = Loads_{c, \boldsymbol{d_l}}$

ULS Actions

Design load due to load combination $Fa_{d_c} = Loads_{c, \boldsymbol{ULS}} \cdot Pa$

Design moment $M_d = 0.128 \cdot A_r \, Fa_d \cdot L_s$ from M_d=0.064wL_s2 substitute A_rF_d =wL_s/2⟶ w=2A_rF_d/L_s

Design shear force $V_d = \frac{2}{3} Fa_d \cdot A_r$

SLS Actions

Design load at SLS $F_{SLS.i_c} = Loads_{c, \boldsymbol{SLS_i}} \cdot Pa$

$F_{SLS.f_c} = Loads_{c, \boldsymbol{SLS_f}} \cdot Pa$

Material characteristics

Flange material

Material safety factor $\gamma_{M.f} = get\_k\left(Material_f, Class, \gamma_M\right) = 1.2$

Duration modification factors for each load combo $k_{mod.f_c} = get\_k\left(Material_f, Class, \boldsymbol{k_{mod}} + LoadDuration_c\right)$

Final deformation factor $k_{def.f} = get\_k\left(Material_f, Class, \boldsymbol{k_{def}}\right) = 0.8$

Material design characteristics $f_{m.f.k} = Tc\left(Material_f, \boldsymbol{f_{m.0.k}}\right) MPa$

$f_{c.0.f.k} = Tc\left(Material_f, \boldsymbol{f_{c.0.k}}\right) MPa$

$f_{t.0.f.k} = Tc\left(Material_f, \boldsymbol{f_{t.0.k}}\right) MPa$

$E_{0.mean.f} = Tc\left(Material_f, \boldsymbol{E_{0.mean}}\right) MPa$

Effective flange width $b_{f.ef} = b_f \cdot f_n = 39 \cdot mm$

Web material

Material safety factor $\gamma_{M.w} = get\_k\left(Material_w, Class, \gamma_M\right) = 1.2$

Duration modification factors for each load combo $k_{mod.w_c} = get\_k\left(Material_w, Class, \boldsymbol{k_{mod}} + LoadDuration_c\right)$

Final deformation factor $k_{def.w} = get\_k\left(Material_w, Class, \boldsymbol{k_{def}}\right) = 1$

Material design characteristics

$f_{v.w.k} = Tc\left(Material_w, \boldsymbol{f_{v.k}}\right) MPa$

$f_{r.w.k} = Tc\left(Material_w, \boldsymbol{f_{r.90.k}}\right) MPa$

$f_{c.90.w.k} = Tc\left(Material_w, \boldsymbol{f_{c.90.k}}\right) MPa$

$f_{t.90.w.k} = Tc\left(Material_w, \boldsymbol{f_{t.90.k}}\right) MPa$

$E_{c.90.mean.w} = Tc\left(Material_w, \boldsymbol{E_{tc.90.mean}}\right) MPa$

$G_{mean.w} = Tc\left(Material_w, \boldsymbol{G_{mean}}\right) MPa$

Web thickness $b_w = Tc\left(Material_w, \boldsymbol{thick}\right) mm = 12.5 \cdot mm$

Effective web thickness

$$b_{w.ef} = \begin{vmatrix} \frac{b_w}{2} \text{ if } w_n = 1 \\ b_w \text{ otherwise} \end{vmatrix} = 12.5 \cdot mm$$

Clear height of the web $h_w = h_b - 2h_f = 0.25\,m$

Area of the web $A_w = h_b \cdot b_w \cdot w_n = 0.01\,m^2$

Material characteristics - design

Flange

Height modification

$$k_{h.f} = max\left[1, min\left[\left(\frac{Tc\left(Material_f, \boldsymbol{k_{h.d}}\right)}{h_f \div mm}\right)^{Tc\left(Material_f, \boldsymbol{k_{h.s}}\right)}, Tc\left(Material_f, \boldsymbol{k_{h.max}}\right)\right]\right] = 1$$

Design characteristics

$$f_{m.f.d} = \frac{f_{m.f.k} \cdot k_{mod.f} \cdot k_{h.f} \cdot k_{sys}}{\gamma_{M.f}}$$

$$f_{t.0.f.d} = \frac{f_{t.0.f.k} \cdot k_{mod.f} \cdot k_{h.f} \cdot k_{sys}}{\gamma_{M.f}}$$

$$f_{c.0.f.d} = \frac{f_{c.0.f.k} \cdot k_{mod.f} \cdot k_{sys}}{\gamma_{M.f}}$$

Web

$$f_{t.90.w.d} = \frac{f_{t.90.w.k} \cdot k_{mod.w} \cdot k_{sys}}{\gamma_{M.w}}$$

$$f_{c.90.w.d} = \frac{f_{c.90.w.k} \cdot k_{mod.w} \cdot k_{sys}}{\gamma_{M.w}}$$

$$f_{v.w.d} = \frac{f_{v.w.k} \cdot k_{mod.w} \cdot k_{sys}}{\gamma_{M.w}}$$

$$f_{r.w.d} = \frac{f_{r.w.k} \cdot k_{mod.w} \cdot k_{sys}}{\gamma_{M.w}}$$

Geometric properties – transformed sections

Instantaneous – transformed section properties:

Second moment of area of flanges $$I_{f.ef} = \frac{b_{f.ef}}{12}\cdot\left(h_b^3 - h_w^3\right) = 2.02\times 10^8\cdot mm^4$$

Transformed web thickness (into flange) $$b_{w.tfd.i} = b_w\cdot\frac{E_{c.90.mean.w}}{E_{0.mean.f}} = 3.59\cdot mm$$

Second moment of area of web $$I_{ef.w.i} = \frac{b_{w.tfd.i}}{12}\cdot h_b^3 = 2.29\times 10^{-5}\,m^4$$

Instantaneous second moment of area of the transformed section $$I_{ef.i} = I_{ef.w.i} + I_{f.ef} = 2.25\times 10^{-4}\,m^4$$

Final – transformed section properties:

of web thickness $$b_{w.tfd.f_c} = b_{w.tfd.i}\cdot\frac{1 + Loads_{c,\psi_2}\cdot k_{def.f}}{1 + Loads_{c,\psi_2}\cdot k_{def.w}}$$

Second moment of area of web $$I_{ef.w.f} = \frac{b_{w.tfd.f}}{12}\cdot h_b^3$$

Final second moment of area of the transformed section $$I_{ef.f} = I_{ef.w.f} + I_{f.ef}$$

Bending stress check in the flanges and the web

Because the mean modulus of elasticity of the flange material is greater than that of the web,only check the stresses in the flanges at the final deformation condition and those in the web at the instantaneous condition.

Stress in flange due to bending – final condition:

Bending stress in top and bottom flange $$\sigma_{m.max.f.d} = \frac{M_d}{I_{ef.f}}\cdot\frac{h_b}{2}$$

Test against bending strength $$r_{b.f} = \max\left(\frac{\sigma_{m.max.f.d}}{f_{m.f.d}}\right) = 0.33$$

Stress in web due to bending – instananeous condition:

Bending stress in the web $$\sigma_{m.w.d} = \frac{M_d}{I_{ef.i}}\cdot\frac{h_b}{2}\cdot\left(\frac{E_{c.90.mean.w}}{E_{0.mean.f}}\right)$$

Test against bending strength in compression $$r_{b.w.c} = \max\left(\frac{\sigma_{m.w.d}}{f_{c.90.w.d}}\right) = 0.49$$

Test against bending strength in tension $$r_{b.w.t} = \max\left(\frac{\sigma_{m.w.d}}{f_{t.90.w.d}}\right) = 0.64$$

Stress in the flange due to axial stress – final condition:

Axial stress in top and bottom flange $$\sigma_{ax.f.d} = \frac{M_d}{I_{ef.f}}\cdot\left(\frac{h_b}{2} - \frac{h_f}{2}\right)$$

Test against axial strength in compression

$$r_{ax.f.c} = \max\left(\frac{\sigma_{ax.f.d}}{f_{c.0.f.d} \cdot k_c}\right) = 0.37$$

Test against axial strength in tension

$$r_{ax.f.t} = \max\left(\frac{\sigma_{ax.f.d}}{f_{t.0.f.d}}\right) = 0.32$$

Buckling and shear stress check in the web

Buckling condition in the web in EC5

$$ratio = \frac{h_w}{b_w} = 19.6$$

Maxiumum value of the ratio is 70

$$r_{b.w} = \frac{ratio}{70} = 0.28$$

Shear strength of the web

Design shear force able to be taken by each web; EC5, equation (9.9))

$$F_{v.w.Ed} = \begin{cases} b_w \cdot h_w \cdot \left(1 + \frac{h_f}{h_w}\right) \cdot f_{v.w.d} & \text{if } ratio \le 35 \\ 35 b_w^2 \left(1 + \frac{h_f}{h_w}\right) \cdot f_{v.w.d} & \text{otherwise} \end{cases}$$

Design shear force able to be taken by the beam

$$F_{v.Ed} = F_{v.w.Ed} \cdot w_n$$

Test shear force in web

$$r_{v.w} = \max\left(\frac{V_d}{F_{v.Ed}}\right) = 0.52$$

Shear strength of the glued joint between the web and the flanges

First moment of area of a flange about the NA,

$$S_f = b_{f.ef} \cdot h_f \cdot \left(\frac{h_b}{2} - \frac{h_f}{2}\right) = 587.92 \cdot cm^3$$

Total length of the glue line in the flange

$$l_g = 2h_f = 0.18\,m$$

Shear stress in the glue line (instant.)

$$\tau_{mean.d.i} = \frac{V_d \cdot S_f}{I_{ef.i} \cdot l_g}$$

Shear stress in the glue line (final)

$$\tau_{mean.d.f} = \frac{V_d \cdot S_f}{I_{ef.f} \cdot l_g}$$

EC5 takes into account the effect of stress concentrations at the web/flange interface in the vicinity of position of the join to web when the height of the flange is greater than $4b_{w.ef}$

$$f_{v.90.d} = \begin{cases} f_{r.w.d} & \text{if } h_f \le 4b_{w.ef} \\ f_{r.w.d} \cdot \left(\frac{4b_{w.ef}}{h_f}\right)^{0.8} & \text{otherwise} \end{cases}$$

$$r_{v.g} = \max\left(\frac{\tau_{mean.d.i}}{f_{v.90.d}}, \frac{\tau_{mean.d.f}}{f_{v.90.d}}\right) = 0.55$$

Deflection of the beam at the SLS

At the instantaneous condition:

Instantaneous deflection at mid-span

$$\mu_{inst} = \max\left(\frac{0.01304 \cdot A_r \cdot F_{SLS.i} \cdot L_s^3}{E_{0.mean.f} \cdot I_{ef.i}}\right) \qquad \mu_{inst} = 11.62 \cdot mm$$

Allowable Instantaneous deflection at mid-span

$$\mu_{inst.allow} = \frac{L_s}{300} = 19.97 \cdot mm$$

$$r_{d.i} = \frac{\mu_{inst}}{\mu_{inst.allow}} = 0.58$$

At the final deformation condition:

transform of web thickness

$$b_{w.tfd.f} = b_{w.tfd.i} \cdot \frac{1 + k_{def.f}}{1 + k_{def.w}} = 3.23 \cdot mm$$

Second moment of area of web

$$I_{ef.w.f} = \frac{b_{w.tfd.f}}{12} \cdot h_b^3 = 2.07 \times 10^7 \cdot mm^4$$

Second moment of area of beam

$$I_{ef.f} = I_{ef.w.f} + I_{f.ef} = 2.22 \times 10^{-4} m^4$$

Final deflection at mid-span

$$\mu_{final} = \max\left[\frac{0.01304 \cdot A_r \cdot F_{SLS.f} \cdot L_s^3 \left(1 + k_{def.f}\right)}{E_{0.mean.f} \cdot I_{ef.f}}\right] = 13.43 \cdot mm$$

Allowable final deflection at mid-span

$$\mu_{final.allow} = \frac{L_s}{250} = 23.97 \cdot mm$$

$$r_{d.f} = \frac{\mu_{final}}{\mu_{final.allow}} = 0.56$$

Results of calculation

$$r_{max} = \max\left(r_{b.f}, r_{b.w.c}, r_{b.w.t}, r_{ax.f.c}, r_{ax.f.t}, r_{b.w}, r_{v.w}, r_{v.g}, r_{d.i}, r_{d.f}\right) = 0.64$$

$$\mathrm{Check}\left(r_{max}\right) = \text{"O.K."}$$

Skylight surrounds

The skylight surround is rectangular steel frame and supported on seven columns. See Drawings 'Skylight Steel' & 'Skylight Design'. All beams are of the same section size to simplify construction. The design can be treated as two separate long beams at the back and front. The loading on these beams is caluclated and combined in the spreadsheet 'Skylight'. These loads are input into the Finnwood program to perform Bending, Shear and Deflection calculations for the Back and Front Beams.

See Drawings **11 - Skylight design**
12 - Skylight steel frame
Spreadsheet **Skylight**

Selected results from :-
Finnwood 2.1 (2.1.0.23)
STRUCTURAL INFORMATION:

Type of structure: Roof beam
Material: KERTO-S
Profile: 2x45x450 (B=90 mm, H=450 mm) (Selected for EI value near to that of the Steel)

Cantilever/span lengths:

Cantilever/Span:	Horizontal [mm]:
Left cantilever	2650.0
Span 1	4830.0
Span 2	3790.0
Span 3	4350.0
Total:	15620.0

Support:	Position x [mm]:	Width [mm]:	Type:
A:	2650	100	Pinned support (X,Y)
B:	7480	100	Pinned support (Y)
C:	11270	100	Pinned support (Y)
D:	15620	100	Pinned support (Y)

Load on Back Beam:

Dead load (Dead load, Permanent, ULS-movability = 25.9 %):

Point load: 1:	FY = 12.58 kN	x = 0.0 mm	(c)
Point load: 2:	FY = 9.15 kN	x = 11020.0 mm	(d)
Point load: 3:	FY = 7.41 kN	x = 15620.0 mm	(e)
Line load: 1:	QY = 1.850 kN/m	x = 2860 - 7310 mm	(cd2)
Line load: 2:	QY = 1.850 - 0.000 kN/m	x = 7310 - 11020 mm	(cd3)
Line load: 3:	QY = 0.000 - 1.850 kN/m	x = 11020 - 14730 mm	(de1)
Line load: 4:	QY = 1.850 kN/m	x = 14730 - 15620 mm	(de2)
Line load: 5:	QY = 0.300 kN/m	x = 0 - 15620 mm	(beam)
Line load: 6:	QY = 0.000 - 1.850 kN/m	x = 0 - 2860 mm	(cd1)
Line load: 7:	QY = 0.810 kN/m	x = 0 - 15620 mm	(Glazing)

Snow load (Snow load h<1000 m, Short-term, ULS/SLS-movability = 100.0 %):

Point load: 1:	FY = 4.38 kN	x = 0.0 mm	(c)
Point load: 2:	FY = 4.53 kN	x = 11020.0 mm	(d)
Point load: 3:	FY = 2.94 kN	x = 15620.0 mm	(e)
Line load: 1:	QY = 1.090 kN/m	x = 2860 - 7310 mm	(cd2)
Line load: 2:	QY = 1.090 - 0.000 kN/m	x = 7310 - 11020 mm	(cd3)
Line load: 3:	QY = 0.000 - 1.090 kN/m	x = 11020 - 14730 mm	(de1)
Line load: 4:	QY = 1.090 kN/m	x = 14730 - 15620 mm	(de2)

Line load: 5:	QY = 0.000 - 1.090 kN/m	x = 0 - 2860 mm	(cd1)
Line load: 6:	QY = 0.500 kN/m	x = 0 - 15620 mm	(Glazing)

Wind load (down) (Wind load, Instantaneous):

Point load: 1:	FY = 0.19 kN	x = 0.0 mm	(c)
Point load: 2:	FY = 0.74 kN	x = 11020.0 mm	(d)
Point load: 3:	FY = 1.42 kN	x = 15620.0 mm	(e)
Line load: 1:	QY = 0.320 kN/m	x = 2860 - 7310 mm	(cd2)
Line load: 2:	QY = 0.320 - 0.000 kN/m	x = 7310 - 11020 mm	(cd3)
Line load: 3:	QY = 0.000 - 0.320 kN/m	x = 11020 - 14730 mm	(de1)
Line load: 4:	QY = 0.320 kN/m	x = 14730 - 15620 mm	(de2)
Line load: 5:	QY = 0.000 - 0.320 kN/m	x = 0 - 2860 mm	(cd1)
Line load: 6:	QY = 0.090 kN/m	x = 0 - 15620 mm	(Glazing)

Wind load (upp) (Wind load, Instantaneous):

Point load: 1:	FY = -2.32 kN	x = 0.0 mm	(c)
Point load: 2:	FY = -2.29 kN	x = 11020.0 mm	(d)
Point load: 3:	FY = -1.73 kN	x = 15620.0 mm	(e)
Line load: 1:	QY =0.000 - -0.720 kN/m	x = 0 - 2860 mm	(cd1)
Line load: 2:	QY = -0.720 kN/m	x = 2860 - 7310 mm	(cd2)
Line load: 3:	QY =-0.720 - 0.000 kN/m	x = 7310 - 11020 mm	(cd3)
Line load: 4:	QY =0.000 - -0.720 kN/m	x = 11020 - 14730 mm	(de1)
Line load: 5:	QY = -0.720 kN/m	x = 14730 - 15620 mm	(de2)
Line load: 6:	QY = -0.450 kN/m	x = 0 - 15620 mm	(Glazing)

FRONT LOADING INFORMATION:

Dead load (Dead load, Permanent, ULS-movability = 25.9 %):

Point load: 1:	FY = 7.41 kN	x = 0.0 mm	(f)
Point load: 2:	FY = 11.19 kN	x = 11020.0 mm	(g)
Point load: 3:	FY = 7.41 kN	x = 15620.0 mm	(h)
Line load: 1:	QY = 1.850 kN/m	x = 0 - 7310 mm	(f-g1)
Line load: 2:	QY = 1.850 - 0.000 kN/m	x = 7310 - 11020 mm	(f-g2)
Line load: 3:	QY = 0.000 - 1.850 kN/m	x = 11020 - 14730 mm	(g-h1)
Line load: 4:	QY = 1.850 kN/m	x = 14730 - 15620 mm	(g-h2)
Line load: 5:	QY = 0.300 kN/m	x = 0 - 15620 mm	(beam)
Line load: 6:	QY = 0.810 kN/m	x = 0 - 15620 mm	(Glazing)

Snow load (Snow load h<1000 m, Short-term, ULS/SLS-movability = 100.0 %):

Point load: 1:	FY = 2.94 kN	x = 0.0 mm	(f)
Point load: 2:	FY = 5.53 kN	x = 11020.0 mm	(g)
Point load: 3:	FY = 2.94 kN	x = 15620.0 mm	(h)
Line load: 1:	QY = 1.090 kN/m	x = 0 - 7310 mm	(f-g1)
Line load: 2:	QY = 1.090 - 0.000 kN/m	x = 7310 - 11020 mm	(f-g2)
Line load: 3:	QY = 0.000 - 1.090 kN/m	x = 11020 - 14730 mm	(g-h1)
Line load: 4:	QY = 1.090 kN/m	x = 14730 - 15620 mm	(g-h2)
Line load: 5:	QY = 0.500 kN/m	x = 0 - 15620 mm	(Glazing)

Wind load (down) (Wind load, Instantaneous):

Point load: 1:	FY = 0.70 kN	x = 0.0 mm	(f)
Point load: 2:	FY = 0.00 kN	x = 11020.0 mm	(g)
Point load: 3:	FY = 1.42 kN	x = 15620.0 mm	(h)
Line load: 1:	QY = 0.260 kN/m	x = 0 - 7310 mm	(f-g1)
Line load: 2:	QY = 0.260 - 0.000 kN/m	x = 7310 - 11020 mm	(f-g2)
Line load: 3:	QY = 0.000 - 0.260 kN/m	x = 11020 - 14730 mm	(g-h1)

Line load: 4:	QY = 0.260 kN/m	x = 14730 - 15620 mm	(g-h2)
Line load: 5:	QY = 0.060 kN/m	x = 0 - 15620 mm	(Glazing)

Wind load (upp) (Wind load, Instantaneous):

Point load: 1:	FY = -2.63 kN	x = 0.0 mm	(f)
Point load: 2:	FY = -2.78 kN	x = 11020.0 mm	(g)
Point load: 3:	FY = -1.41 kN	x = 15620.0 mm	(h)
Line load: 1:	QY = -0.720 kN/m	x = 0 - 7310 mm	(fg-1)
Line load: 2:	QY =-0.720 - 0.000 kN/m	x = 7310 - 11020 mm	(fg-2)
Line load: 3:	QY =0.000 - -0.720 kN/m	x = 11020 - 14730 mm	(gh-1)
Line load: 4:	QY = -0.720 kN/m	x = 14730 - 15620 mm	(gh-2)
Line load: 5:	QY = -0.590 kN/m	x = 0 - 15620 mm	(Glazing)

DEFLECTIONS: (Will need a small adjustment for different EI values)

Back	Left cant., Utot,inst:	36.21 mm	
Back	Span 1, Utot,inst:		-5.17 mm
Back	Span 2, Utot,inst:		1.40 mm
Back	Span 3, Utot,inst:		1.42 mm
Front	Left cant., Utot,inst:	29.02 mm	
Front	Span 1, Utot,inst:		-3.98 mm
Front	Span 2, Utot,inst:		1.28 mm
Front	Span 3, Utot,inst:		1.42 mm

EXTREME FORCES:

Result:		Maximum value:	Location x:
Back	Vy,max	35.20 kN	2650 mm
Front	Vy,max	33.68 kN	11270 mm
Back	Mz,max	75.67 kNm	2650 mm
Front	Mz,max	62.84 kNm	2650 mm

SUPPORT REACTIONS:

Support:		ULSmax:	ULSmin:
Back	A	69.36 kN	20.15 kN
Back	B	18.58kN	-13.85 kN
Back	C	43.43 kN	9.49 kN
Back	D	26.32 kN	5.88 kN
Front	A	62.25 kN	11.08 kN
Front	B	21.08 kN	-8.83 kN
Front	C	45.80 kN	8.71 kN
Front	D	26.25 kN	6.09 kN

- Upplift occurs, make sure of the anchoring

The beams are dividing into 10 sections for Buckling analysis and the following values have been extracted from the Bending Moment diagrams in Finnwood. The beam is actually partially restrained by all rafters.

Section	Length	M_{max} [kNm]	
		Front	Back
1	1.25	24.3	35.0
2	1.40	62.4	75.7
3	1.49	62.4	75.7
4	1.67	26.5	34.7
5	1.67	3.0	5.0
6	2.03	6.4	8.3
7	1.76	11.5	12.2
8	1.55	11.5	12.2
9	1.55	7.5	7.9
10	1.25	7.4	7.7

$$L = \begin{pmatrix} 1.25 \\ 1.4 \\ 1.49 \\ 1.67 \\ 1.67 \\ 2.03 \\ 1.76 \\ 1.55 \\ 1.55 \\ 1.25 \end{pmatrix} \cdot m \qquad M_{max} = \begin{pmatrix} 35 \\ 75.7 \\ 75.7 \\ 34.7 \\ 5 \\ 8.3 \\ 12.2 \\ 12.2 \\ 7.9 \\ 7.7 \end{pmatrix} \cdot kN \cdot m$$

The anaylysis shows all the maxium forces occur in the back beam.

Maximum bending moment $M_{Ed} = 75.67 kN \cdot m$

Maxium Shear force $V_{Ed} = 35.20 kN$

Material characteristics

Steel section selected 305 x 102 x 25 UB S355

Steel strength $f_y = 355 N \cdot mm^{-2}$

Elastic Modulus $E = 210 GPa$

Shear modulus $G = 81 GPa$

Beam height $h = 305.1 mm$

Height between fillets $d = 275.9 mm$

Beam width $b = 101.6 mm$

Web thickness $t_w = 5.8 mm$

Flange thickness $t_f = 7 mm$

Height of web $h_w = h - 2t_f = 291.1 \cdot mm$

Root radius $r = 7.6 mm$

Second moments $I_y = 4460 cm^4 \qquad I_z = 123 cm^4$

Plastic first moment $W_{pl.y} = 342 cm^3$

Elastic first moment $W_{el.y} = 292 cm^3$

Section Area $A_s = 31.6 cm^2$

Shear Area $A_v = \min\left[h_w \cdot t_w, A_s - 2 \cdot b \cdot t_f + \left(t_w + 2r\right) t_f\right] = 16.88 \cdot cm^2$

Warping constant $I_w = 27300 cm^6$

Torsional constant $I_T = 4.77 cm^4$

Beam classification

$$\varepsilon = \sqrt{\frac{235 \cdot N \cdot mm^{-2}}{f_y}} = 0.81$$

Flange $\frac{t_f}{\varepsilon \cdot mm} = 8.6$ < 9 so Class 1 Plastic

Web $\frac{d}{t_w \cdot \varepsilon} = 58$ < 72 so also Class 1 Plastic

Bending strength

Bending strength $M_{c.y.Rd} = f_y \cdot W_{pl.y} = 121.41 \cdot kN \cdot m$ $OKifLT\left(M_{Ed}, M_{c.y.Rd}\right) = "O.K."$

Shear strength

Shear resistance $V_{pl.Rd} = \left(\frac{A_V \cdot f_y}{\sqrt{3}}\right) \div \gamma_{M0} = 346 \cdot kN$ $OKifLT\left(V_{Ed}, V_{pl.Rd}\right) = "O.K."$

Shear buckling $\frac{h_w}{t_w \cdot \varepsilon} = 61.7$ less than 72 so check not required [EC3 6.2.6(6)]

Lateral torsional buckling

$\frac{h}{b} = 3$ so use buckling curve c:

$\alpha_{LT} = 0.49$ [EC3 Tables 6.3/6.4 NA 2.18]

$\bar{\lambda}_{LT.0} = 0.4$

$\beta = 0.75$ [EC3 UK NA 2.17]

Calculate $M_{b.Rd}$ for all M_{cr} values

$C_1 = 1.00$ Conservative value

$s = 0 .. rows(L) - 1$

$$M_{cr_s} = C_1 \cdot \left[\frac{\pi^2 E \cdot I_z}{\left(L_s\right)^2}\right] \cdot \sqrt{\frac{I_w}{I_z} + \frac{\left(L_s\right)^2 G \cdot I_T}{\pi^2 \cdot \left(E \cdot I_z\right)}}$$ From [SN003]

$$\bar{\lambda}_{LT} = \sqrt{\frac{f_y \cdot W_{pl.y}}{M_{cr}}}$$

$$\phi_{LT} = 0.5\left[1 + \alpha_{LT} \cdot \left(\bar{\lambda}_{LT} - \bar{\lambda}_{LT.0}\right) + \beta \cdot \bar{\lambda}_{LT}^{2}\right]$$

$$\chi_{LT} = \frac{1}{\left(\phi_{LT} + \sqrt{\phi_{LT}^{2} - \beta \cdot \bar{\lambda}_{LT}^{2}}\right)}$$

$$k_c = \frac{1}{\sqrt{C_1}} = 1$$

$$f = 1 - 0.5\left(1-k_c\right)\left[1-2\left(\bar{\lambda}_{LT}-0.8\right)^2\right]$$

$$\chi_{LT.mod} = \frac{\chi_{LT}}{f}$$

$$M_{b.Rd} = \chi_{LT.mod} \cdot M_{c.y.Rd}$$

$$r_m = \frac{M_{max}}{M_{b.Rd}} =$$

	0
0	0.35
1	0.80
2	0.83
3	0.41
4	0.06
5	0.12
6	0.15
7	0.14
8	0.09
9	0.08

$\text{Check}\left(r_m\right) = \text{"O.K."}$

Beam Deflections

The deflection values calculated by the Finwood program will need to be adjusted to allow for different EI values and reduced shear deflection.

EI value for the beam in finnwood is $EI_T = 13.8GPa \cdot 90mm \cdot \frac{(450mm)^3}{12} = 9.43 \times 10^3 \cdot kN \cdot m^2$

EI value for the steel beams $EI_S = E \cdot I_y = 9.37 \times 10^3 \cdot kN \cdot m^2$

ratio of values $\frac{EI_T}{EI_S} = 1.01$

Ratio of shear values $\frac{13800}{600} \div \frac{E}{G} = 8.87$

Also in the actual roof, deflections in the beams will be resisted by the rafters, resulting in reduced deflections.

Frame supports

The frame is supported by 7 columns, 6 spaced in pairs along the main beams and the last centered on the right hand end. Only the front support @ C and the end support are not completely axial loads. The columns are restrained by the first floor. The buckling length will be treated as half the total length.

Calculations are using simplified equation from NCCI SN048b for Buckling and Bending analysis of the columns.

ULS Load on each column
$$N_{Ed} = \begin{pmatrix} 69.36 \\ 18.58 \\ 43.43 \\ 62.25 \\ 21.08 \\ 45.8 \\ 26.25 + 26.32 + 1 \end{pmatrix} \cdot kN$$

load offsets from center

$$e_y = \begin{pmatrix} 0 \\ 0 \\ 0 \\ 0 \\ 0 \\ 0 \\ 35 \end{pmatrix} mm \quad e_z = \begin{pmatrix} 0 \\ 0 \\ 0 \\ 0 \\ 0 \\ 25 \\ 0 \end{pmatrix} mm$$

Bending moments due load eccentricity

$$M_{y.Ed_s} = N_{Ed_s} \cdot e_{y_s}$$

$$M_{z.Ed_s} = N_{Ed_s} \cdot e_{z_s}$$

$$M_{y.Ed} = \begin{pmatrix} 0 \\ 0 \\ 0 \\ 0 \\ 0 \\ 0 \\ 1.87 \end{pmatrix} \cdot kN \cdot m \quad M_{z.Ed} = \begin{pmatrix} 0 \\ 0 \\ 0 \\ 0 \\ 0 \\ 1.15 \\ 0 \end{pmatrix} \cdot kN \cdot m$$

Section size 100 x 50 x 3 CF RHS S235

Steel strength $f_y = 235N \cdot mm^{-2}$

Column depth $h = 100mm$

Column width $b = 50mm$

Web thickness $t = 3.0mm$

Second moments $I_y = 106cm^4$ $I_z = 36.1cm^4$

Radius of gyration $i_y = 3.56cm$ $i_z = 2.07cm$

Plastic first moment $W_{pl.y} = 26.7cm^3$ $W_{pl.z} = 16.4cm^3$

Elastic first moment $W_{el.y} = 21.3cm^3$ $W_{el.z} = 14.4cm^3$

Section Area $A_s = 8.41cm^2$

Torsional constant $I_T = 88.6cm^4$

Classification

$$\varepsilon = \sqrt{\frac{235}{\frac{f_y}{N \cdot mm^{-2}}}} = 1$$

Flange $\frac{b - 3t}{t \cdot \varepsilon} = 14$ < 33 so Class 1 Plastic

Web $\frac{h - 3t}{t \cdot \varepsilon} = 30$ < 38 so Class 1 Plastic

Column height $L_c = 5.4m$

Effective length $L_{E.y} = L_c \div 2$ $\quad L_{E.z} = 0.5L_c \div 2$

Slenderness $\lambda_y = \frac{L_{E.y}}{i_y} = 75.84$ $\quad \lambda_z = \frac{L_{E.z}}{i_z} = 65.22$

Compression

Design Compression resistance $N_{c.Rd} = \frac{A_s \cdot f_y}{\gamma_{M0}} = 197.63 \cdot kN$ $\quad \text{Check}\left(\frac{N_{Ed}}{N_{c.Rd}}\right) = \text{"O.K."}$

Buckling and Bending

For cold formed RHS sections need to use buckling curve c $\alpha = 0.49$

Buckling about y-y (major) axis

$$\_\lambda_y = \frac{\lambda_y}{93.9\varepsilon} = 0.81$$

$$\phi_y = 0.5\left[1 + \alpha \cdot \left(\_\lambda_y - 0.2\right) + \_\lambda_y^{\,2}\right] = 0.98$$

$$\chi_y = \frac{1}{\left(\phi_y + \sqrt{\phi_y^{\,2} - \_\lambda_y^{\,2}}\right)} = 0.657$$

Design buckling resistance $N_{b.y.Rd} = \frac{\chi_y \cdot A_s \cdot f_y}{\gamma_{M1}} = 129.91 \cdot kN$

Buckling about z-z (minor) axis

$$\_\lambda_z = \frac{\lambda_z}{93.9\varepsilon} = 0.69$$

$$\phi_z = 0.5\left[1 + \alpha \cdot \left(\_\lambda_z - 0.2\right) + \_\lambda_z^{\,2}\right] = 0.86$$

$$\chi_z = \frac{1}{\left(\phi_z + \sqrt{\phi_z^{\,2} - \_\lambda_z^{\,2}}\right)} = 0.728$$

Design buckling resistance $N_{b.z.Rd} = \frac{\chi_z \cdot A_s \cdot f_y}{\gamma_{M1}} = 143.89 \cdot kN$

Minimum design buckling resistance $N_{b.min.Rd} = \min\left(N_{b.y.Rd}, N_{b.z.Rd}\right) = 129.91 \cdot kN$

Bending about y-y (major) axis

$$M_{c.y.Rd} = \frac{f_y \cdot W_{pl.y}}{\gamma_{M0}} = 6.27 \cdot kN \cdot m$$

$$M_{cr.y} = C_1 \cdot \left[\frac{\pi^2 E \cdot I_z}{\left(L_{E.y}\right)^2} \right] \cdot \sqrt{\frac{I_w}{I_z} + \frac{\left(L_{E.y}\right)^2 G \cdot I_T}{\pi^2 \cdot \left(E \cdot I_z\right)}} = 90.35 \cdot kN \cdot m$$ From [SN003]

$$\bar{\lambda}_{LT.y} = \sqrt{\frac{M_{c.y.Rd}}{M_{cr.y}}} = 0.26$$

$$\phi_{LT.y} = 0.5\left[1 + \alpha_{LT} \cdot \left(\bar{\lambda}_{LT.y} - \bar{\lambda}_{LT.0}\right) + \beta \cdot \bar{\lambda}_{LT.y}^{\;2}\right] = 0.49$$

$$\chi_{LT.y} = \min\left[1, \frac{1}{\left(\phi_{LT.y} + \sqrt{\phi_{LT.y}^{\;2} - \beta \cdot \bar{\lambda}_{LT.y}^{\;2}}\right)}\right] = 1$$

$$k_c = \frac{1}{\sqrt{C_1}} = 1$$

$$f = 1 - 0.5\left(1 - k_c\right)\left[1 - 2\left(\bar{\lambda}_{LT.y} - 0.8\right)^2\right] = 1$$

$$\chi_{LT.mod.y} = \frac{\chi_{LT.y}}{f}$$

$$M_{b.y.Rd} = \chi_{LT.mod.y} \cdot M_{c.y.Rd} = 6.27 \cdot kN \cdot m$$

Bending about z-z (minor) axis

$$M_{b.z.Rd} = \frac{f_y \cdot W_{pl.z}}{\gamma_{M1}} = 3.85 \cdot kN \cdot m$$

Test against expression from SN048b

$$r_{bb} = \frac{N_{Ed}}{N_{b.min.Rd}} + \frac{M_{y.Ed}}{M_{b.y.Rd}} + 1.5\left(\frac{M_{z.Ed}}{M_{b.z.Rd}}\right) = \begin{pmatrix} 0.53 \\ 0.14 \\ 0.33 \\ 0.48 \\ 0.16 \\ 0.8 \\ 0.71 \end{pmatrix}$$ Check$\left(r_{bb}\right)$ = "O.K."

Column base plates

All the columns rest on a standard sized base plate. The plates bear on the concrete foundation. Analysys in foundation design section

Maximum design load on a base plate $N_{Ed.Base} = \max\left(N_{Ed}\right) = 69.36 \cdot kN$

Connecting Beam ends to Cross pieces

The right hand end on the main beams is connected to the cross piece and transfers the reaction from the beams to column D. The connection is formed from a right angled joining plate bolted to the webs of beams. Test how many bolts are required.

Maximum force to be transfered $F_j = 25.36kN$

Minimum steel thickness $t_s = 5mm$ Joiner plate

Using a M12 Bolt in Grade 8.8 through 5mm thickness gives a Bearing capacity of

$F_b = 27.6kN$ [Tata Steel Blue Book Table 14.2.3]

So only one bolt is needed to carry the load, design will use 2 bolts for stability.

Beams from middle of wall O to 'a' and from middle of wall K to 'd'

There are two beams which form the ridges on the rear of the building which are formed from two parts. First a rafter from the exterior wall to the confulence of the roof angled at 45° (points 'a' and 'b') and the second horizontal from there to the skylight. This combined rafter will be spliced together by steel splice plates glued on both sides and have a continuous steel flange reinforcement glued on both top and bottom. Analyse beams as continuous to find maxiumum bending and shear values. Check these against strength of the beam and then calculate strength of splice and its connection to the beams. Loads on these beams are derived from the spreadsheet 'Skylight' and show that the beam from 'O' to 'a' has the highest loading so this is the beam which is analyised below.

See Drawing 10 - Ridge to Rafter splice plates

Finnwood 2.1 was used to calculate forces.

Span: 7.5m

Support:	Position x [mm]:	Width [mm]:	Type:
1:	0	90	Pinned support (X,Y)
2:	7500	50	Pinned support (Y)

LOADING INFORMATION:

Dead load (Dead load, Permanent, ULS-movability = 25.9 %):

Point load: 1:	FY = 3.90 kN	x = 3400.0 mm	(Hip Rafter NO-a)
Point load: 2:	FY = 3.90 kN	x = 3400.0 mm	(Hip Rafter OP-a)
Line load: 1:	QY = 0.560 kN/m	x = 0 - 3400 mm	(Rafter O to a)
Line load: 2:	QY = 0.150 kN/m	x = 0 - 7500 mm	(Self weight)
Line load: 3:	QY = 3.500 - 1.750 kN/m	x = 3400 - 7500 mm	(Ridge a-c)

Snow load (Snow load h<1000 m, Short-term, ULS/SLS-movability = 100.0 %):

Point load: 1:	FY = 1.10 kN	x = 3400.0 mm	(Hip Rafter NO-a)
Point load: 2:	FY = 1.10 kN	x = 3400.0 mm	(Hip Rafter OP-a)
Line load: 1:	QY = 0.140 kN/m	x = 0 - 3400 mm	(Rafter O to a)
Line load: 2:	QY = 1.200 - 0.600 kN/m	x = 3400 - 7500 mm	(Ridge a-c)

Wind load (down) (Wind load, Instantaneous):

Point load: 1:	FY = 0.40 kN	x = 3400.0 mm	(Hip Rafter NO-a)
Point load: 2:	FY = 0.40 kN	x = 3400.0 mm	(Hip Rafter OP-a)
Line load: 1:	QY = 0.110 kN/m	x = 0 - 3400 mm	(Rafter O to a)

LOAD COMBINATIONS:

Combination 1 (ULS)

1.35*Dead load

Combination 2 (ULS)

1.35*Dead load + 1.50*Snow load

Combination 3 (ULS)

1.35*Dead load + 1.50*Snow load + 1.50*0.50*Wind load (down)

Combination 4 (ULS)

1.35*Dead load + 1.50*0.50*Snow load + 1.50*Wind load (down)

Combination 6 (ULS)

1.35*Dead load + 1.50*0.50*Snow load

Combination 7 (ULS)

1.35*Dead load + 1.50*Wind load (down)

Combination 10 (SLS, Characteristic)

1.00*Dead load

Combination 11 (SLS, Characteristic)

1.00*Dead load + 1.00*Snow load

Combination 12 (SLS, Characteristic)

1.00*Dead load + 1.00*Snow load + 1.00*0.50*Wind load (down)

Combination 13 (SLS, Characteristic)

1.00*Dead load + 1.00*0.50*Snow load + 1.00*Wind load (down)

Combination 15 (SLS, Characteristic)

1.00*Dead load + 1.00*0.50*Snow load

Find critical load combination

Then iterate the calculations for all combinations $c = 0..\,2$

$$\begin{array}{l} \qquad\qquad\;\; \text{ULS} \;\; \text{SLS} \\ \text{Loads} = \begin{pmatrix} 29.1 & 21.6 & 21.6 & \textit{Permanent} & 1 \\ 38.7 & 28.0 & 28.0 & \textit{Short} & 0 \\ 39.6 & 28.5 & 28.5 & \textit{Instant} & 0 \end{pmatrix} \end{array}$$

Combination 1 + 10
Combination 2 + 11
Combination 3 + 12

Load duration for this load combo $\text{LoadDuration} = \begin{pmatrix} \textit{Permanent} \\ \textit{Short} \\ \textit{Instant} \end{pmatrix}$

ULS Actions

Design moment due to critical load combinations $M_d = \begin{pmatrix} 38.4 \\ 50.9 \\ 52.3 \end{pmatrix} \cdot kN \cdot m$

Design shear force due to critical load combinations $V_d = \begin{pmatrix} 12.0 \\ 21.8 \\ 22.1 \end{pmatrix} \cdot kN$

Support reactions $Sr_O = \begin{pmatrix} 13.0 \\ 17.0 \\ 17.5 \end{pmatrix} \cdot kN$ $Sr_c = V_d$

Design conditions

Load sharing

$k_{sys} = 1.0$

Beam Design

Number of flanges $f_n = 1$

Number of webs $w_n = 2$

Effective span $L_e = 4.5m$

Beam depth $h_b = 400mm$ minimum depth i.e. the Rafter

Compression edge of beam will be continuously restrained so $k_c = 1.0$

Material choices & sizes

Flange

$Material_f = \text{"Kerto S Edgewise"}$

Height of flange $h_f = 100mm$

Width of flange $b_f = 39mm$

Web

web material $Material_w = \text{"Plywood Finnish Birch 12mm 9 ply"}$

Steel

Thickness of the steel $h_s = 5mm$

Width of steel $b_s = b_f + 2\,Tc\left(Material_w, \boldsymbol{thick}\right)mm = 63 \cdot mm$

Steel grade $Material_s = \text{"S275"}$

Material characteristics

Flange material

Material safety factor $\gamma_{M.f} = get\_k\left(Material_f, Class, \gamma_M\right) = 1.2$

Duration modification factors for each load combo $k_{mod.f_c} = get\_k\left(Material_f, Class, \boldsymbol{k_{mod}} + LoadDuration_c\right)$

Final deformation factor $k_{def.f} = get\_k\left(Material_f, Class, \boldsymbol{k_{def}}\right) = 0.8$

Material design characteristics

$f_{m.f.k} = Tc\left(Material_f, \boldsymbol{f_{m.0.k}}\right) MPa$

$f_{c.0.f.k} = Tc\left(Material_f, \boldsymbol{f_{c.0.k}}\right) MPa$

$f_{c.90.f.k} = Tc\left(Material_f, \boldsymbol{f_{c.90.k}}\right) MPa$

$f_{t.0.f.k} = Tc\left(Material_f, \boldsymbol{f_{t.0.k}}\right) MPa$

$E_{0.mean.f} = Tc\left(Material_f, \boldsymbol{E_{0.mean}}\right) MPa$

Effective flange width $b_{f.ef} = b_f \cdot f_n = 39 \cdot mm$

Web material

Material safety factor $\gamma_{M.w} = \text{get\_k}(\text{Material}_w, \text{Class}, \gamma_M) = 1.2$

Duration modification factors for each load combo $k_{mod.w_c} = \text{get\_k}(\text{Material}_w, \text{Class}, k_{mod} + \text{LoadDuration}_c)$

Final deformation factor $k_{def.w} = \text{get\_k}(\text{Material}_w, \text{Class}, k_{def}) = 1$

Material design characteristics

$$f_{v.w.k} = \text{Tc}(\text{Material}_w, f_{v.k})\,\text{MPa}$$

$$f_{r.w.k} = \text{Tc}(\text{Material}_w, f_{r.90.k})\,\text{MPa}$$

$$f_{c.90.w.k} = \text{Tc}(\text{Material}_w, f_{c.90.k})\,\text{MPa}$$

$$f_{t.90.w.k} = \text{Tc}(\text{Material}_w, f_{t.90.k})\,\text{MPa}$$

$$E_{c.90.mean.w} = \text{Tc}(\text{Material}_w, E_{tc.90.mean})\,\text{MPa}$$

$$G_{mean.w} = \text{Tc}(\text{Material}_w, G_{mean})\,\text{MPa}$$

Web thickness $b_w = \text{Tc}(\text{Material}_w, thick)\,\text{mm} = 12\cdot\text{mm}$

Effective web thickness

$$b_{w.ef} = \begin{cases} \dfrac{b_w}{2} & \text{if } w_n = 1 \\ b_w & \text{otherwise} \end{cases} = 12\cdot\text{mm}$$

Clear height of the web $h_w = h_b - 2h_f = 0.2\,\text{m}$

Area of the web $A_w = h_b \cdot b_w \cdot w_n = 9.6 \times 10^{-3}\,\text{m}^2$

Steel band

Yield strength $f_{y.s.k} = 275\text{MPa}$

Modulus of elasticity $E_s = 210\text{GPa}$

Material factor $\gamma_{M0.s} = \gamma_{M0}$

Material characteristics - design

Flange

Height modification

$$k_{h.f} = \max\left[1, \min\left[\left(\frac{\text{Tc}(\text{Material}_f, k_{h.d})}{h_f \div \text{mm}}\right)^{\text{Tc}(\text{Material}_f, k_{h.s})}, \text{Tc}(\text{Material}_f, k_{h.max})\right]\right] = 1$$

Design characteristics

$$f_{m.f.d} = \frac{f_{m.f.k} \cdot k_{mod.f} \cdot k_{h.f} \cdot k_{sys}}{\gamma_{M.f}}$$

$$f_{t.0.f.d} = \frac{f_{t.0.f.k} \cdot k_{mod.f} \cdot k_{h.f} \cdot k_{sys}}{\gamma_{M.f}}$$

$$f_{c.0.f.d} = \frac{f_{c.0.f.k} \cdot k_{mod.f} \cdot k_{sys}}{\gamma_{M.f}}$$

$$f_{c.90.f.d} = \frac{f_{c.90.f.k} \cdot k_{mod.f} \cdot k_{sys}}{\gamma_{M.f}}$$

Web

$$f_{t.90.w.d} = \frac{f_{t.90.w.k} \cdot k_{mod.w} \cdot k_{sys}}{\gamma_{M.w}}$$

$$f_{c.90.w.d} = \frac{f_{c.90.w.k} \cdot k_{mod.w} \cdot k_{sys}}{\gamma_{M.w}}$$

$$f_{v.w.d} = \frac{f_{v.w.k} \cdot k_{mod.w} \cdot k_{sys}}{\gamma_{M.w}}$$

$$f_{r.w.d} = \frac{f_{r.w.k} \cdot k_{mod.w} \cdot k_{sys}}{\gamma_{M.w}}$$

Steel

$$f_{y.s.d} = \frac{f_{y.s.k}}{\gamma_{M0}} = 275 \cdot \mathrm{MPa}$$

Geometric properties – transformed sections

Instantaneous – transformed section properties:

Second moment of area of flanges

$$I_{f.ef} = \frac{b_{f.ef}}{12} \cdot \left[h_b^3 - \left(h_b - 2h_f \right)^3 \right] = 1.82 \times 10^4 \cdot \mathrm{cm}^4$$

Second moment of area of steel

$$I_s = \frac{b_s}{12} \left[\left(h_b + 2h_s \right)^3 - h_b^3 \right] = 2.58 \times 10^3 \cdot \mathrm{cm}^4$$

Transform the steel into flange

$$I_{s.ef} = I_s \cdot \frac{E_s}{E_{0.mean.f}} = 3.93 \times 10^4 \cdot \mathrm{cm}^4$$

Transformed web thickness (into flange)

$$b_{w.tfd.i} = b_w \cdot \frac{E_{c.90.mean.w}}{E_{0.mean.f}} = 7.1 \cdot \mathrm{mm}$$

Second moment of area of web

$$I_{ef.w.i} = \frac{b_{w.tfd.i}}{12} \cdot h_b^3 = 3.79 \times 10^{-5}\, \mathrm{m}^4$$

Instantaneous second moment of area of the transformed section

$$I_{ef.i} = I_{ef.w.i} + I_{f.ef} = 2.2 \times 10^{-4}\, \mathrm{m}^4$$

Final – transformed section properties:

of web thickness

$$b_{w.tfd.f_c} = b_{w.tfd.i} \cdot \frac{1 + \mathrm{Loads}_{c,\,\psi_2} \cdot k_{def.f}}{1 + \mathrm{Loads}_{c,\,\psi_2} \cdot k_{def.w}}$$

Second moment of area of web

$$I_{ef.w.f} = \frac{b_{w.tfd.f}}{12} \cdot h_b^3$$

Final second moment of area of the transformed section

$$I_{ef.f} = I_{ef.w.f} + I_{f.ef}$$

Bending stress check in the flanges and the web

Because the mean modulus of elasticity of the flange material is greater than that of the web,only check the stresses in the flanges at the final deformation condition and those in the web at the instantaneous condition.

Stress in flange due to bending – final condition:

Bending stress in top and bottom flange

$$\sigma_{m.max.f.d} = \frac{M_d}{I_{ef.f} + I_{s.ef}} \cdot \frac{h_b}{2}$$

Test against bending strength

$$r_{b.f} = \max\left(\frac{\sigma_{m.max.f.d}}{f_{m.f.d}}\right) = 0.5 \qquad \text{Check}\left(r_{b.f}\right) = \text{"O.K."}$$

Stress in steel due to bending – final condition:

Bending stress in top and bottom flange

$$\sigma_{m.max.s.d} = \frac{M_d}{I_{ef.f} + I_{s.ef}} \cdot \frac{\left(h_b + 2h_s\right)}{2} \cdot \frac{E_s}{E_{0.mean.f}}$$

Test against bending strength

$$r_{b.s} = \max\left(\frac{\sigma_{m.max.s.d}}{f_{y.s.d}}\right) = 0.97 \qquad \text{Check}\left(r_{b.s}\right) = \text{"O.K."}$$

Stress in web due to bending – instananeous condition:

Bending stress in the web

$$\sigma_{m.w.d} = \frac{M_d}{I_{ef.i} + I_{s.ef}} \cdot \frac{h_b - 2h_s}{2} \cdot \left(\frac{E_{c.90.mean.w}}{E_{0.mean.f}}\right)$$

Test against bending strength in compression

$$r_{b.w.c} = \max\left(\frac{\sigma_{m.w.d}}{f_{c.90.w.d}}\right) = 0.59 \qquad \text{Check}\left(r_{b.w.c}\right) = \text{"O.K."}$$

Test against bending strength in tension

$$r_{b.w.t} = \max\left(\frac{\sigma_{m.w.d}}{f_{t.90.w.d}}\right) = 0.41 \qquad \text{Check}\left(r_{b.w.t}\right) = \text{"O.K."}$$

Stress in the flange due to axial stress – final condition:

Axial stress in top and bottom flange

$$\sigma_{ax.f.d} = \left[\frac{M_d}{I_{ef.f} + I_{s.ef}} \cdot \left(\frac{h_b - 2h_s}{2} - \frac{h_f}{2}\right)\right]$$

Test against axial strength in compression

$$r_{ax.f.c} = \max\left(\frac{\sigma_{ax.f.d}}{f_{c.0.f.d} \cdot k_c}\right) = 0.52 \qquad \text{Check}\left(r_{ax.f.c}\right) = \text{"O.K."}$$

Test against axial strength in tension

$$r_{ax.f.t} = \max\left(\frac{\sigma_{ax.f.d}}{f_{t.0.f.d}}\right) = 0.46 \qquad \text{Check}\left(r_{ax.f.t}\right) = \text{"O.K."}$$

Buckling and shear stress check in the web

Buckling condition in the web in EC5

$$ratio = \frac{h_w}{b_w} = 16.67$$

Maximum ratio is 70

$$r_{b.w} = \frac{ratio}{70} = 0.24 \qquad Check(r_{b.w}) = \text{"O.K."}$$

Shear strength of the web

Design shear force able to be taken by each web; EC5, equation (9.9))

$$F_{v.w.Ed} = \begin{cases} b_w \cdot h_w \cdot \left(1 + \frac{h_f}{h_w}\right) \cdot f_{v.w.d} & \text{if } ratio \le 35 \\ 35 b_w^2 \left(1 + \frac{h_f}{h_w}\right) \cdot f_{v.w.d} & \text{otherwise} \end{cases}$$

Design shear force able to be taken by the beam

$$F_{v.Ed} = F_{v.w.Ed} \cdot w_n$$

Test shear force in web

$$r_{v.w} = \max\left(\frac{V_d}{F_{v.Ed}}\right) = 0.42 \qquad Check(r_{v.w}) = \text{"O.K."}$$

Shear strength of the glued joint between the web and the flanges

First moment of area of a flange about the NA,

$$S_f = b_{f.ef} \cdot h_f \cdot \left(\frac{h_b - 2h_s}{2} - \frac{h_f}{2}\right) = 565.5 \cdot cm^3$$

Total length of the glue line in the flange

$$l_g = 2h_f = 0.2\,m$$

Shear stress in the glue line (instant.)

$$\tau_{mean.d.i} = \frac{V_d \cdot S_f}{I_{ef.i} \cdot l_g}$$

Shear stress in the glue line (final)

$$\tau_{mean.d.f} = \frac{V_d \cdot S_f}{I_{ef.f} \cdot l_g}$$

EC5 takes into account the effect of stress concentrations at the web/flange interface in the vicinity of position of the join to web when the height of the flange is greater than $4b_{w.ef}$

$$f_{v.90.d} = \begin{cases} f_{r.w.d} & \text{if } h_f \le 4b_{w.ef} \\ f_{r.w.d} \cdot \left(\frac{4b_{w.ef}}{h_f}\right)^{0.8} & \text{otherwise} \end{cases}$$

$$r_{v.g.t} = \max\left(\frac{\tau_{mean.d.i}}{f_{v.90.d}}, \frac{\tau_{mean.d.f}}{f_{v.90.d}}\right) = 0.3 \qquad Check(r_{v.g.t}) = \text{"O.K."}$$

Shear strength of the glued joint between the flanges and the steel

From EC5 on thin flanged beams

First moment of area of the steel about the NA, $S_f = b_s \cdot h_s \cdot \left(\frac{h_b}{2}\right) \cdot \frac{E_s}{E_{0.mean.f}} = 958.7 \cdot cm^3$

Total width of the glue line to the flange $l_g = b_f \cdot f_n = 0.04\,m$

Shear stress in the glue line (instant.) $\tau_{mean.d.i} = \frac{V_d \cdot S_f}{I_{ef.i} \cdot l_g}$

Shear stress in the glue line (final) $\tau_{mean.d.f} = \frac{V_d \cdot S_f}{I_{ef.f} \cdot l_g}$

Research on internet finds glue bond strength when paralell to grain in softwood to be > 10 N/mm^2 (wood failure)

$$f_{t.g} = 10N \cdot mm^{-2}$$

EC5 takes into account the effect of stress concentrations at the interface in the vicinity of position of the join to flange

$$f_{v.d} = \begin{cases} f_{t.g} & \text{if } b_s \le 4h_s \\ f_{t.g} \cdot \left(\frac{4h_s}{b_s}\right)^{0.8} & \text{otherwise} \end{cases} = 3.99 \cdot N \cdot mm^{-2}$$

$$r_{v.g.s} = \max\left(\frac{\tau_{mean.d.i}}{f_{v.d}}, \frac{\tau_{mean.d.f}}{f_{v.d}}\right) = 0.62 \qquad \mathrm{Check}(r_{v.g.s}) = \text{"O.K."}$$

Bending stress check in the splice plate

Second moment of splice inc. flange: $I_{sp} = \frac{2h_s}{12} \cdot h_b^3 + I_s = 7.92 \times 10^3 \cdot cm^4$

Stress in splice plate $\sigma_{sp.d} = \frac{M_d}{I_{sp}} \cdot \frac{(h_b + 2h_s)}{2}$

$$r_{sp} = \max\left(\frac{\sigma_{sp.d}}{f_{y.s.d}}\right) = 0.49 \qquad \mathrm{Check}(r_{sp}) = \text{"O.K."}$$

Stress in glue between plate and beam

I am unshure of how to calculate this value.

Support reactions

Support at the skylight (point 'c')

Support reaction is shared by steel supporting frames welded to beam with a minimum bearing of 75mm

Support area $a = 75 \cdot mm \cdot b_f = 2.92 \times 10^3 \cdot mm^2$

Support load $f_s = \frac{Sr_c}{2a} = \begin{pmatrix} 2.05 \times 10^6 \\ 3.73 \times 10^6 \\ 3.78 \times 10^6 \end{pmatrix} Pa$ $f_{c.90.f.d} = \begin{pmatrix} 3 \\ 4.5 \\ 5.5 \end{pmatrix} \cdot MPa$

ratio $r_{sr.a} = \max\left(\frac{f_s}{f_{c.90.f.d}}\right) = 0.83$ $Check\left(r_{sr.a}\right) = "O.K."$

Support at the external wall

This will be evaluated later in the external wall section

First floor design

Design conditions

Floor construction

First floor is constructed of timber I-beams of 240mm depth spaced at 400mm intervals with 18mm OSB3 decking glued to joists and 10mm Fermacell ceilings (also glued to joists). In the living areas additional sound reduction layers are added (25mm mineral wool,25mm plasterboard and 18mm OSB deck) This gives permanent loads of 0.29kN/m² for storage areas and gallery and 0.69kN/m² for Living areas.
Joists are supported by the ground floor internal walls and the inner leaf of the outside walls.
All walls on the first floor are non load bearing partions.

Drawings **5 - First flooor plan**
13 - First floor Joists
Spreadsheet **Joists**

Gallery

The gallery is cantilevered out from the first floor over the Great room. It has a spiral staircase to the ground floor. A bookcase will placed against the inside wall. Allowance will be made for a glass balustrade. Bending strength is analysed in each joist calculation.

Test the equilibrium of the gallery cantilever

Gallery has a lighter floor stucture but has a bookcase and a railing as point loads.

	length		floor	bookcase	railing
Supported span	$l_s = 4.86$	UDL	$G_{s.f} = 0.75$		
Cantilever span	$l_c = 2.34$	UDL	$G_{c.f} = 0.25$	$G_{c.b} = 1.2$ @ $d_{c.b} = 0.2$	$G_{c.r} = 0.3$ @ $d_{c.r} = l_c$
Live load	$Q = 1.5$				

For EQU testing the following partial factors are used

$\gamma_{G.s} = 0.9$ $\gamma_{G.c} = 1.1$

$\gamma_{Q.s} = 0$ $\gamma_{Q.c} = 1.5$

Calculate balance of forces on the two spans

$$r_b = \frac{\gamma_{G.c} \cdot G_{c.f} \cdot l_c \cdot l_c + \gamma_{G.c} \cdot G_{c.b} \cdot d_{c.b} + \gamma_{G.c} \cdot G_{c.r} \cdot d_{c.r} + \gamma_{Q.c} \cdot Q \cdot l_c \cdot l_c}{\gamma_{G.s} \cdot G_{s.f} \cdot l_s \cdot l_s + \gamma_{Q.s} \cdot Q \cdot l_s \cdot l_s} = 0.9322$$

$\text{Check}\left(r_b\right) =$ "O.K."

Floor Joists

All the floor joists have been evaluated by the Finnwood 2.1 program and the results for each is show below.

J01A Over Kitchen - Loft Loads

Profile: FJI 45/240 (B=45 mm, H=240 mm)

Cantilever/Span: Horizontal [mm]:
Span 1 4850.0
Total: 4850.0

Support: Position x [mm]: Width [mm]: Type:

1:	0	45	Pinned support (X,Y)
2:	4850	45	Pinned support (Y)

LOADING INFORMATION:
Dead load (Dead load, Permanent, ULS-movability = 25.9 %):
Beam weight: QY = 0.026 kN/m x = 0 - 4850 mm
Surface load: 1: QY = 0.700 kN/m2 x = 0 - 4850 mm

Live load (Imposed load A-residential areas, Medium-term, ULS/SLS-movability = 100.0 %):
Surface load: 1: QY = 1.000 kN/m2 x = 0 - 4850 mm

DESIGN RESULTS:
Maximum utility rate: 96.1 %

GOVERNING DESIGN RESULTS:

Check:	Actual:	Allowable:	% allowable:	Location x:	
Shear (y):	2.19 kN	7.51 kN	29.2 %	4588 mm	Medium-term
Bending (Mz):	2.98 kNm	7.02 kNm	42.4 %	2425 mm	Medium-term
(without kcrit):	2.98 kNm	7.02 kNm	42.4 %	2425 mm	Medium-term
Utot,fin: (characteristic)	16.25 mm	19.40 mm	83.8 %	2425 mm	
Utot,inst: (characteristic)	11.53 mm	12.00 mm	96.1 %	2425 mm	

SUPPORT REACTIONS:

Support:	ULSmax:	ULSmin:	SLSmax:	SLSmin:	Rd/A:	Bearing:
1:	2.46 kN	0.74 kN	1.71 kN	0.74 kN	1.21 N/mm2	37.5 %
2:	2.46 kN	0.74 kN	1.71 kN	0.74 kN	1.21 N/mm2	37.5 %

J01B Over Kitchen - Loft Loads

Profile: FJI 58/240 (B=58 mm, H=240 mm)

Cantilever/Span:	Horizontal [mm]:
Span 1	4860.0
Total:	4860.0

Support:	Position x [mm]:	Width [mm]:	Type:
1:	0	63	Pinned support (X,Y)
2:	4860	63	Pinned support (Y)

LOADING INFORMATION:
Dead load (Dead load, Permanent, ULS-movability = 25.9 %):
Beam weight: QY = 0.031 kN/m x = 0 - 4860 mm
Surface load: 1: QY = 0.700 kN/m2 x = 0 - 4860 mm

Partition load (Dead load, Permanent, ULS/SLS-movability = 100.0 %):
Line load: 1: QY = 0.200 kN/m x = 0 - 4850 mm (Short (Half))

Live load (Imposed load A-residential areas, Medium-term, ULS/SLS-movability = 100.0 %):
Surface load: 1: QY = 1.000 kN/m2 x = 0 - 4860 mm

Maximum utility rate: 99.0 %

GOVERNING DESIGN RESULTS:

Check:	Actual:	Allowable:	% allowable:	Location x:	
Shear (y):	2.78 kN	8.02 kN	34.7 %	4588 mm	Medium-term
Bending (Mz):	3.81 kNm	9.24 kNm	41.2 %	2430 mm	Medium-term
(without kcrit):	3.81 kNm	9.24 kNm	41.2 %	2430 mm	Medium-term
Utot,fin:	17.64 mm	19.44 mm	90.7 %	2430 mm	(characteristic)
Utot,inst:	11.88 mm	12.00 mm	99.0 %	2430 mm	(characteristic)

SUPPORT REACTIONS:

Support:	ULSmax:	ULSmin:	SLSmax:	SLSmin:	Rd/A:	
Bearing:						
1:	3.13 kN	0.76 kN	2.21 kN	0.76 kN	0.86 N/mm2	30.8 %
2:	3.13 kN	0.76 kN	2.21 kN	0.76 kN	0.86 N/mm2	30.7 %

J03 Over Kitchen - Std Loads

Profile: FJI 89/240 (B=89 mm, H=240 mm)

Cantilever/Span:	Horizontal [mm]:
Span 1	4860.0
Total:	4860.0

Support:	Position x [mm]:	Width [mm]:	Type:
1:	0	63	Pinned support (X,Y)
2:	4860	63	Pinned support (Y)

LOADING INFORMATION:

Dead load (Dead load, Permanent, ULS-movability = 25.9 %):

Beam weight:	QY = 0.043 kN/m	x = 0 - 4860 mm	
Surface load: 1:	QY = 0.750 kN/m2	x = 0 - 4860 mm	

Partition load (Dead load, Permanent, ULS/SLS-movability = 100.0 %):

Line load: 1:	QY = 0.200 kN/m	x = 0 - 4850 mm	(Short)

Live load (Imposed load A-residential areas, Medium-term, ULS/SLS-movability = 100.0 %):

Surface load: 1:	QY = 1.500 kN/m2	x = 0 - 4860 mm

Maximum utility rate: 85.2 %

GOVERNING DESIGN RESULTS:

Check:	Actual:	Allowable:	% allowable:	Location x:	
Shear (y):	3.52 kN	8.28 kN	42.6 %	4588 mm	Medium-term
Bending (Mz):	4.82 kNm	14.30 kNm	33.7 %	2430 mm	Medium-term
(without kcrit):	4.82 kNm	14.30 kNm	33.7 %	2430 mm	Medium-term
Utot,fin:	15.06 mm	19.44 mm	77.5 %	2430 mm	(characteristic)
Utot,inst:	10.23 mm	12.00 mm	85.2 %	2430 mm	(characteristic)

SUPPORT REACTIONS:

Support:	ULSmax:	ULSmin:	SLSmax:	SLSmin:	Rd/A:	
Bearing:						
1:	3.97 kN	0.83 kN	2.78 kN	0.83 kN	0.71 N/mm2	33.8 %
2:	3.97 kN	0.83 kN	2.78 kN	0.83 kN	0.71 N/mm2	33.8 %

J04 Kitchen + Gallery + Balustrade

Profile:	FJI 89/240 (B=89 mm, H=240 mm)				

Cantilever/Span:	Horizontal [mm]:
Span 1	4860.0
Right cantilever	1520.0
Total:	6380.0

Support:	Position x [mm]:	Width [mm]:	Type:
1:	0	63	Pinned support (X,Y)
2:	4860	126	Pinned support (Y)

LOADING INFORMATION:

Dead load (Dead load, Permanent, ULS-movability = 25.9 %):

Line load: 1:	QY = 0.300 kN/m	x = 4965 - 6295 mm	(Balustrade)
Surface load: 1:	QY = 0.750 kN/m2	x = 0 - 4865 mm	(Main Floor)
Surface load: 2:	QY = 0.250 kN/m2	x = 4865 - 6380 mm	(Gallery Floor)
Surface load: 3:	QY = 6.000 kN/m2	x = 4965 - 5265 mm	(BookCase)
Surface load: 4:	QY = 6.000 kN/m2	x = 6330 - 6380 mm	(Balustrade)

Live load (Imposed load A-residential areas, Medium-term, ULS/SLS-movability = 100.0 %):

Surface load: 1:	QY = 1.500 kN/m2	x = 0 - 6380 mm

Maximum utility rate: 57.2 %

GOVERNING DESIGN RESULTS:

Check:	Actual:	Allowable:	% allowable:	Location x:	
Shear (y):	3.21 kN	8.28 kN	38.8 %	4557 mm	Medium-term
Bending (Mz):	3.47 kNm	14.30 kNm	24.2 %	2233 mm	Medium-term
(without kcrit):	3.47 kNm	14.30 kNm	24.2 %	2233 mm	Medium-term
Utot,fin:	9.35 mm	19.44 mm	48.1 %	2392 mm	(characteristic)
Utot,inst:	6.86 mm	12.00 mm	57.2 %	2392 mm	(characteristic)

SUPPORT REACTIONS:

Support:	ULSmax:	ULSmin:	SLSmax:	SLSmin:	Rd/A:	
Bearing:						
1:	3.01 kN	0.30 kN	2.03 kN	0.42 kN	0.54 N/mm2	25.7 %
2:	6.85 kN	2.28 kN	4.80 kN	2.28 kN	0.61 N/mm2	28.9 %

J05 Kitchen + Gallery + Balustrade

Profile:	FJI 89/240 (B=89 mm, H=240 mm)

Cantilever/Span:	Horizontal [mm]:
Span 1	4860.0
Right cantilever	2340.0
Total:	7200.0

Support:	Position x [mm]:	Width [mm]:	Type:
1:	0	63	Pinned support (X,Y)
2:	4860	126	Pinned support (Y)

LOADING INFORMATION:

Dead load (Dead load, Permanent, ULS-movability = 25.9 %):

Surface load: 1:	QY = 0.750 kN/m2	x = 0 - 4860 mm	(Main Floor)
Surface load: 2:	QY = 0.250 kN/m2	x = 4860 - 7140 mm	(Gallery Floor)
Surface load: 3:	QY = 6.000 kN/m2	x = 4960 - 5260 mm	(BookCase)
Surface load: 4:	QY = 6.000 kN/m2	x = 7150 - 7200 mm	(Balustrade)

Live load (Imposed load A-residential areas, Medium-term, ULS/SLS-movability = 100.0 %):
Surface load: 1: QY = 1.500 kN/m2 x = 0 - 7200 mm

Maximum utility rate: 80.4 %

GOVERNING DESIGN RESULTS:

Check:	Actual:	Allowable:	% allowable:	Location x:	
Shear (y):	3.48 kN	8.28 kN	42.1 %	4557 mm	Medium-term
Bending (Mz):	3.50 kNm	14.30 kNm	24.5 %	2340 mm	Medium-term
(without kcrit):	3.50 kNm	14.30 kNm	24.5 %	2340 mm	Medium-term
Utot,fin:	13.82 mm	18.72 mm	73.8 %	7200 mm	(characteristic)
Utot,inst:	10.75 mm	13.37 mm	80.4 %	7200 mm	(characteristic)

SUPPORT REACTIONS:

Support:	ULSmax:	ULSmin:	SLSmax:	SLSmin:	Rd/A:	
Bearing:						
1:	3.02 kN	0.02 kN	2.04 kN	0.24 kN	0.54 N/mm2	25.8 %
2:	7.43 kN	1.95 kN	5.14 kN	1.94 kN	0.66 N/mm2	31.4 %

J06 Hall+ Gallery + Balustrade

Profile: FJI 89/240 (B=89 mm, H=240 mm)

Cantilever/Span:	Horizontal [mm]:
Span 1	4860.0
Right cantilever	2000.0
Total:	6860.0

Support:	Position x [mm]:	Width [mm]:	Type:	
1:	0	50	Pinned support (X,Y)	ITT49.5
2:	4860	126	Pinned support (Y)	

LOADING INFORMATION:

Dead load (Dead load, Permanent, ULS-movability = 25.9 %):

Surface load: 1:	QY = 0.750 kN/m2	x = 0 - 4860 mm	(Main Floor)
Surface load: 2:	QY = 0.250 kN/m2	x = 4860 - 6860 mm	(Gallery Floor)
Surface load: 3:	QY = 6.000 kN/m2	x = 4960 - 5260 mm	(BookCase)
Surface load: 4:	QY = 6.000 kN/m2	x = 6810 - 6860 mm	(Balustrade)

Live load (Imposed load A-residential areas, Medium-term, ULS/SLS-movability = 100.0 %):
Surface load: 1: QY = 1.500 kN/m2 x = 0 - 6860 mm

Maximum utility rate: 59.3 %

GOVERNING DESIGN RESULTS:

Check:	Actual:	Allowable:	% allowable:	Location x:	
Shear (y):	3.32 kN	8.28 kN	40.1 %	4557 mm	Medium-term
Bending (Mz):	3.55 kNm	14.30 kNm	24.8 %	2401 mm	Medium-term

(without kcrit):	3.55 kNm	14.30 kNm	24.8 %	2401 mm	Medium-term
Utot,fin:	9.76 mm	19.44 mm	50.2 %	2401 mm	(characteristic)
Utot,inst:	7.12 mm	12.00 mm	59.3 %	2401 mm	(characteristic)

SUPPORT REACTIONS:

Support:	ULSmax:	ULSmin:	SLSmax:	SLSmin:	Rd/A:	
Bearing:						
1:	3.04 kN	0.19 kN	2.06 kN	0.36 kN	--	--
2:	6.92 kN	1.90 kN	4.80 kN	1.90 kN	0.62 N/mm2	29.2 %

- "--" indicates that hangers are used
- See HANGERS for hanger design results

HANGERS:

Support:	Hanger:	Bearing:	Hanger name:
1	57.7 %	33.0 %	ITT49.5, to rectangle header

J07 Hall+ Gallery + Balustrade

Profile: FJI 58/240 (B=58 mm, H=240 mm)

Cantilever/Span:	Horizontal [mm]:
Span 1	4900.0
Right cantilever	1620.0
Total:	6520.0

Support:	Position x [mm]:	Width [mm]:	Type:
1:	0	0	Pinned support (X,Y)
2:	4900	126	Pinned support (Y)

LOADING INFORMATION:

Dead load (Dead load, Permanent, ULS-movability = 25.9 %):

Surface load: 1:	QY = 0.750 kN/m2	x = 0 - 4900 mm	(Main Floor)
Surface load: 2:	QY = 0.250 kN/m2	x = 4900 - 6520 mm	(Gallery Floor)
Surface load: 3:	QY = 6.000 kN/m2	x = 5000 - 5400 mm	(BookCase)
Surface load: 4:	QY = 6.000 kN/m2	x = 6470 - 6520 mm	(Balustrade)

Live load (Imposed load A-residential areas, Medium-term, ULS/SLS-movability = 100.0 %):

Surface load: 1: QY = 1.500 kN/m2 x = 0 - 6520 mm

Maximum utility rate: 88.7 %

GOVERNING DESIGN RESULTS:

Check:	Actual:	Allowable:	% allowable:	Location x:	
Shear (y):	3.21 kN	8.02 kN	40.0 %	4597 mm	Medium-term
Bending (Mz):	3.61 kNm	9.24 kNm	39.1 %	2282 mm	Medium-term
(without kcrit):	3.61 kNm	9.24 kNm	39.1 %	2282 mm	Medium-term
Utot,fin:	14.28 mm	19.60 mm	72.9 %	2445 mm	(characteristic)
Utot,inst:	10.64 mm	12.00 mm	88.7 %	2445 mm	(characteristic)

SUPPORT REACTIONS:

Support:	ULSmax:	ULSmin:	SLSmax:	SLSmin:	Rd/A:	
Bearing:						
1:	3.07 kN	0.33 kN	2.08 kN	0.45 kN	--	--

2:	6.74 kN	2.10 kN	4.70 kN	2.10 kN	0.92 N/mm2	33.1 %

J08 Es1 + Bed1+ Gallery + Balustrade

Profile: FJI 45/240 (B=45 mm, H=240 mm)

Cantilever/Span:	Horizontal [mm]:
Span 1	1870.0
Span 2	4860.0
Right cantilever	1430.0
Total:	8160.0

Support:	Position x [mm]:	Width [mm]:	Type:
1:	0	63	Pinned support (X,Y)
2:	1870	126	Pinned support (Y)
3:	6730	126	Pinned support (Y)

LOADING INFORMATION:

Dead load (Dead load, Permanent, ULS-movability = 25.9 %):

Surface load: 1:	QY = 0.750 kN/m2	x = 0 - 6730 mm	(Main Floor)
Surface load: 2:	QY = 0.250 kN/m2	x = 6730 - 8160 mm	(Gallery Floor)
Surface load: 3:	QY = 6.000 kN/m2	x = 6830 - 7130 mm	(BookCase)
Surface load: 4:	QY = 6.000 kN/m2	x = 8110 - 8160 mm	(Balustrade)

Live load (Imposed load A-residential areas, Medium-term, ULS/SLS-movability = 100.0 %):

Surface load: 1: QY = 1.500 kN/m2 x = 0 - 8160 mm

Maximum utility rate: 74.7 %

GOVERNING DESIGN RESULTS:

Check:	Actual:	Allowable:	% allowable:	Location x:	
Shear (y):	3.21 kN	7.51 kN	42.8 %	2173 mm	Medium-term
Bending (Mz):	2.58 kNm	7.02 kNm	36.8 %	1870 mm	Medium-term
(without kcrit):	2.58 kNm	7.02 kNm	36.8 %	1870 mm	Medium-term
Utot,fin:	-7.30 mm	11.44 mm	63.8 %	8160 mm	(characteristic)
Utot,inst:	-6.10 mm	8.17 mm	74.7 %	8160 mm	(characteristic)

SUPPORT REACTIONS:

Support:	ULSmax:	ULSmin:	SLSmax:	SLSmin:	Rd/A:	
Bearing:						
1:	1.08 kN	-1.04 kN	0.65 kN	-0.61 kN	0.38 N/mm2	13.0 %
2:	6.21 kN	0.80 kN	4.22 kN	1.01 kN	1.10 N/mm2	37.3 %
3:	5.66 kN	1.68 kN	3.94 kN	1.69 kN	1.00 N/mm2	34.0 %

- Upplift occurs, make sure of the anchoring

J09A Es1 + Bed1

Profile: FJI 45/240 (B=45 mm, H=240 mm)

Cantilever/Span:	Horizontal [mm]:
Span 1	1870.0
Span 2	4865.0
Total:	6735.0

Support:	Position x [mm]:	Width [mm]:	Type:
1:	0	63	Pinned support (X,Y)

2:	1870	126	Pinned support (Y)
3:	6735	63	Pinned support (Y)

LOADING INFORMATION:
Dead load (Dead load, Permanent, ULS-movability = 25.9 %):
Surface load: 1: QY = 0.750 kN/m2 x = 0 - 6665 mm (Main Floor)

Live load (Imposed load A-residential areas, Medium-term, ULS/SLS-movability = 100.0 %):
Surface load: 1: QY = 1.500 kN/m2 x = 0 - 6735 mm

Maximum utility rate: 78.2 %

GOVERNING DESIGN RESULTS:

Check:	Actual:	Allowable:	% allowable:	Location x:	
Shear (y):	3.34 kN	7.51 kN	44.5 %	2173 mm	Medium-term
Bending (Mz):	2.73 kNm	7.02 kNm	38.9 %	1870 mm	Medium-term
(without kcrit):	2.73 kNm	7.02 kNm	38.9 %	1870 mm	Medium-term
Utot,fin:	13.18 mm	19.46 mm	67.8 %	4546 mm	(characteristic)
Utot,inst:	9.38 mm	12.00 mm	78.2 %	4546 mm	(characteristic)

SUPPORT REACTIONS:

Support:	ULSmax:	ULSmin:	SLSmax:	SLSmin:	Rd/A:	
Bearing:						
1:	0.82 kN	-1.12 kN	0.47 kN	-0.69 kN	0.29 N/mm2	9.9 %
2:	6.42 kN	1.48 kN	4.43 kN	1.47 kN	1.13 N/mm2	38.5 %
3:	2.61 kN	0.56 kN	1.79 kN	0.57 kN	0.92 N/mm2	31.3 %

- Upplift occurs, make sure of the anchoring

J09B Es1 + Bed1

Profile: FJI 45/240 (B=45 mm, H=240 mm)

Cantilever/Span:	Horizontal [mm]:
Span 1	1870.0
Span 2	4865.0
Total:	6735.0

Support:	Position x [mm]:	Width [mm]:	Type:
1:	0	63	Pinned support (X,Y)
2:	1870	126	Pinned support (Y)
3:	6735	63	Pinned support (Y)

LOADING INFORMATION:
Dead load (Dead load, Permanent, ULS-movability = 25.9 %):
Surface load: 1: QY = 0.750 kN/m2 x = 0 - 6665 mm (Main Floor)

Partition load (Dead load, Permanent, ULS/SLS-movability = 100.0 %):
Line load: 1: QY = 0.200 kN/m x = 0 - 6665 mm (Short)

Live load (Imposed load A-residential areas, Medium-term, ULS/SLS-movability = 100.0 %):
Surface load: 1: QY = 1.500 kN/m2 x = 0 - 6735 mm

Maximum utility rate: 95.7 %

GOVERNING DESIGN RESULTS:

Check:	Actual:	Allowable:	% allowable:	Location x:	
Shear (y):	4.03 kN	7.51 kN	53.7 %	2173 mm	Medium-term
Bending (Mz):	3.30 kNm	7.02 kNm	47.0 %	1870 mm	Medium-term
(without kcrit):	3.30 kNm	7.02 kNm	47.0 %	1870 mm	Medium-term
Utot,fin:	16.90 mm	19.46 mm	86.8 %	4546 mm	(characteristic)
Utot,inst:	11.49 mm	12.00 mm	95.7 %	4546 mm	(characteristic)

SUPPORT REACTIONS:

Support:	ULSmax:	ULSmin:	SLSmax:	SLSmin:	Rd/A:	
Bearing:						
1:	1.06 kN	-1.41 kN	0.64 kN	-0.90 kN	0.37 N/mm2	12.7 %
2:	7.74 kN	1.48 kN	5.41 kN	1.47 kN	1.37 N/mm2	46.4 %
3:	3.14 kN	0.55 kN	2.19 kN	0.56 kN	1.11 N/mm2	37.6 %

- Upplift occurs, make sure of the anchoring

J11A Es1 + Bed1 - Loft Load

Profile: FJI 38/240 (B=38 mm, H=240 mm)

Cantilever/Span:	Horizontal [mm]:
Span 1	1800.0
Span 2	4850.0
Total:	6650.0

Support:	Position x [mm]:	Width [mm]:	Type:
1:	0	45	Pinned support (X,Y)
2:	1800	115	Pinned support (Y)
3:	6650	45	Pinned support (Y)

LOADING INFORMATION:

Dead load (Dead load, Permanent, ULS-movability = 25.9 %):

Beam weight: QY = 0.023 kN/m x = 0 - 6650 mm

Surface load: 1: QY = 0.700 kN/m2 x = 0 - 6650 mm

Partition load (Dead load, Permanent, ULS/SLS-movability = 100.0 %):

Line load: 1: QY = 0.200 kN/m x = 0 - 6650 mm (Short)

Live load (Imposed load A-residential areas, Medium-term, ULS/SLS-movability = 100.0 %):

Surface load: 1: QY = 1.000 kN/m2 x = 0 - 6650 mm

Maximum utility rate: 89.3 %

GOVERNING DESIGN RESULTS:

Check:	Actual:	Allowable:	% allowable:	Location x:	
Shear (y):	3.28 kN	7.25 kN	45.2 %	2098 mm	Medium-term
Bending (Mz):	2.70 kNm	5.83 kNm	46.3 %	1800 mm	Medium-term
(without kcrit):	2.70 kNm	5.83 kNm	46.3 %	1800 mm	Medium-term
Utot,fin:	16.21 mm	19.40 mm	83.5 %	4322 mm	(characteristic)
Utot,inst:	10.71 mm	12.00 mm	89.3 %	4489 mm	(characteristic)

SUPPORT REACTIONS:

Support:	ULSmax:	ULSmin:	SLSmax:	SLSmin:	Rd/A:	
Bearing:						
1:	0.74 kN	-1.17 kN	0.42 kN	-0.75 kN	0.43 N/mm2	12.8 %
2:	6.31 kN	1.49 kN	4.45 kN	1.49 kN	1.44 N/mm2	46.0 %
3:	2.57 kN	0.58 kN	1.81 kN	0.59 kN	1.50 N/mm2	44.5 %

- Upplift occurs, make sure of the anchoring

J11B Es1 + Bed1 - Loft Load

Profile: FJI 45/240 (B=45 mm, H=240 mm)

Cantilever/Span:	Horizontal [mm]:
Span 1	4850.0
Span 2	1870.0
Total:	6720.0

Support:	Position x [mm]:	Width [mm]:	Type:
1:	0	63	Pinned support (X,Y)
2:	4850	126	Pinned support (Y)
3:	6720	63	Pinned support (Y)

LOADING INFORMATION:

Dead load (Dead load, Permanent, ULS-movability = 25.9 %):

Beam weight:	QY = 0.026 kN/m	x = 0 - 6720 mm
Surface load: 1:	QY = 0.700 kN/m2	x = 0 - 6720 mm

Live load (Imposed load A-residential areas, Medium-term, ULS/SLS-movability = 100.0 %):

Surface load: 1:	QY = 1.000 kN/m2	x = 0 - 6720 mm

Maximum utility rate: 60.5 %

GOVERNING DESIGN RESULTS:

Check:	Actual:	Allowable:	% allowable:	Location x:	
Shear (y):	2.58 kN	7.51 kN	34.4 %	4547 mm	Medium-term
Bending (Mz):	2.11 kNm	7.02 kNm	30.0 %	4850 mm	Medium-term
(without kcrit):	2.11 kNm	7.02 kNm	30.0 %	4850 mm	Medium-term
Utot,fin:	10.59 mm	19.40 mm	54.6 %	2184 mm	(characteristic)
Utot,inst:	7.26 mm	12.00 mm	60.5 %	2184 mm	(characteristic)

SUPPORT REACTIONS:

Support:	ULSmax:	ULSmin:	SLSmax:	SLSmin:	Rd/A:	Bearing:
1:	2.04 kN	0.59 kN	1.42 kN	0.60 kN	0.72 N/mm2	24.4 %
2:	4.96 kN	1.50 kN	3.46 kN	1.50 kN	0.88 N/mm2	29.8 %
3:	0.56 kN	-0.80 kN	0.30 kN	-0.47 kN	0.20 N/mm2	6.8 %

- Upplift occurs, make sure of the anchoring

J12 Front Hall + Entertainment

Profile: FJI 38/240 (B=38 mm, H=240 mm)

Cantilever/Span:	Horizontal [mm]:
Span 1	2820.0
Span 2	4170.0
Total:	6990.0

Support:	Position x [mm]:	Width [mm]:	Type:
1:	0	63	Pinned support (X,Y)
2:	2820	89	Pinned support (Y)
3:	6990	89	Pinned support (Y)

LOADING INFORMATION:

Dead load (Dead load, Permanent, ULS-movability = 25.9 %):

Point load: 1:	FY = 0.16 kN	x = 1600.0 mm	(Partition)
Point load: 2:	FY = 0.16 kN	x = 5387.0 mm	(Partition)
Surface load: 1:	QY = 0.750 kN/m2	x = 0 - 6990 mm	

Live load (Imposed load A-residential areas, Medium-term, ULS/SLS-movability = 100.0 %):

Surface load: 1: QY = 1.500 kN/m2 x = 0 - 6990 mm

GOVERNING DESIGN RESULTS:

Check:	Actual:	Allowable:	% allowable:	Location x:	
Shear (y):	2.97 kN	7.25 kN	40.9 %	3104 mm	Medium-term
Bending (Mz):	2.24 kNm	5.83 kNm	38.4 %	2820 mm	Medium-term
(without kcrit):	2.24 kNm	5.83 kNm	38.4 %	2820 mm	Medium-term
Utot,fin:	9.63 mm	16.68 mm	57.7 %	5068 mm	(characteristic)
Utot,inst:	6.81 mm	11.91 mm	57.2 %	5068 mm	(characteristic)

SUPPORT REACTIONS:

Support:	ULSmax:	ULSmin:	SLSmax:	SLSmin:	Rd/A:	Bearing:
1:	1.59 kN	-0.16 kN	1.05 kN	0.02 kN	0.66 N/mm2	21.6 %
2:	6.10 kN	1.55 kN	4.22 kN	1.55 kN	1.80 N/mm2	53.2 %
3:	2.41 kN	0.49 kN	1.66 kN	0.53 kN	0.71 N/mm2	25.1 %

- Upplift occurs, make sure of the anchoring

J14 Front Hall + Stair Framing + Entertainment + Utility

Profile: FJI 45/240 (B=45 mm, H=240 mm)

Cantilever/Span:	Horizontal [mm]:
Span 1	2820.0
Span 2	4170.0
Span 3	3900.0
Total:	10890.0

Support:	Position x [mm]:	Width [mm]:	Type:
1:	0	63	Pinned support (X,Y)
2:	2820	89	Pinned support (Y)
3:	6990	89	Pinned support (Y)
4:	10890	63	Pinned support (Y)

LOADING INFORMATION:

Dead load (Dead load, Permanent, ULS-movability = 25.9 %):

Surface load: 1: QY = 0.750 kN/m2 x = 0 - 10890 mm

Partition load (Dead load, Permanent, ULS/SLS-movability = 100.0 %):

Point load: 1:	FY = 0.16 kN	x = 5387.0 mm	(Short)
Point load: 2:	FY = -0.15 kN	x = 1650.0 mm	(Stair Edge JS01)

Live load (Imposed load A-residential areas, Medium-term, ULS/SLS-movability = 100.0 %):

Point load: 1:	FY = 1.80 kN	x = 1650.0 mm	(Stair Edge JS01)
Surface load: 1:	QY = 1.500 kN/m2	x = 0 - 10890 mm	

Maximum utility rate: 56.7 %

GOVERNING DESIGN RESULTS:

Check:	Actual:	Allowable:	% allowable:	Location x:	
Shear (y):	3.91 kN	7.51 kN	52.0 %	2536 mm	Medium-term

Bending (Mz):	2.46 kNm	7.02 kNm	35.1 %	1650 mm	Medium-term
(without kcrit):	2.46 kNm	7.02 kNm	35.1 %	1650 mm	Medium-term
Utot,fin:	6.39 mm	15.60 mm	41.0 %	8984 mm	(characteristic)
Utot,inst:	4.64 mm	11.14 mm	41.6 %	8984 mm	(characteristic)

SUPPORT REACTIONS:

Support:	ULSmax:	ULSmin:	SLSmax:	SLSmin:	Rd/A:	Bearing:
1:	2.57 kN	-0.15 kN	1.72 kN	0.02 kN	0.91 N/mm2	30.8 %
2:	7.37 kN	0.62 kN	5.01 kN	0.80 kN	1.84 N/mm2	56.7 %
3:	6.31 kN	1.03 kN	4.35 kN	1.15 kN	1.57 N/mm2	48.5 %
4:	2.23 kN	0.22 kN	1.52 kN	0.31 kN	0.79 N/mm2	26.8 %

- Upplift occurs, make sure of the anchoring

J15A Front Hall + Entertainment + Utility

Profile: FJI 38/240 (B=38 mm, H=240 mm)

Cantilever/Span:	Horizontal [mm]:
Span 1	1150.0
Span 2	4140.0
Span 3	3940.0
Total:	9230.0

Support:	Position x [mm]:	Width [mm]:	Type:	
1:	0	57	Pinned support (X,Y)	LBV240/40
2:	1150	89	Pinned support (Y)	
3:	5290	89	Pinned support (Y)	
4:	9230	63	Pinned support (Y)	

LOADING INFORMATION:

Dead load (Dead load, Permanent, ULS-movability = 25.9 %):

Point load: 1:	FY = 0.53 kN	x = 2150.0 mm	(Partition)
Point load: 2:	FY = 0.16 kN	x = 8000.0 mm	(Partition)
Surface load: 1:	QY = 0.750 kN/m2	x = 0 - 9230 mm	

Live load (Imposed load A-residential areas, Medium-term, ULS/SLS-movability = 100.0 %):

Surface load: 1: QY = 1.500 kN/m2 x = 0 - 9230 mm

GOVERNING DESIGN RESULTS:

Check:	Actual:	Allowable:	% allowable:	Location x:	
Shear (y):	2.97 kN	7.25 kN	41.0 %	1434 mm	Medium-term
Bending (Mz):	2.45 kNm	5.83 kNm	42.1 %	5290 mm	Medium-term
(without kcrit):	2.45 kNm	5.83 kNm	42.1 %	5290 mm	Medium-term
Utot,fin:	7.48 mm	15.76 mm	47.4 %	7384 mm	(characteristic)
Utot,inst:	5.37 mm	11.26 mm	47.7 %	7384 mm	(characteristic)

SUPPORT REACTIONS:

Support:	ULSmax:	ULSmin:	SLSmax:	SLSmin:	Rd/A:	Bearing:
1:	0.57 kN	-1.41 kN	0.24 kN	-0.90 kN	--	--
2:	5.70 kN	0.99 kN	3.91 kN	1.23 kN	1.69 N/mm2	49.8 %
3:	6.37 kN	1.58 kN	4.40 kN	1.58 kN	1.88 N/mm2	55.6 %
4:	2.30 kN	0.33 kN	1.57 kN	0.42 kN	0.96 N/mm2	31.4 %

- Upplift occurs, make sure of the anchoring
- "--" indicates that hangers are used
- See HANGERS for hanger design results

HANGERS:

Support:	Hanger:	Bearing:	Hanger name:
1	65.3 %	7.9 %	LBV240/40, to rectangle header all nail holes filled with web stiffeners

See construction details and refer to manufacturers literature for further information

J15B Front Hall + Entertainment + Utility

Profile: FJI 38/240 (B=38 mm, H=240 mm)

Cantilever/Span:	Horizontal [mm]:
Span 1	1150.0
Span 2	4140.0
Span 3	3940.0
Total:	9230.0

Support:	Position x [mm]:	Width [mm]:	Type:		
1 (with stiffener):		0	57	Pinned support (X,Y)	LBV240/40
2:	1150	90	Pinned support (Y)		
3:	5290	90	Pinned support (Y)		
4:	9230	45	Pinned support (Y)		

LOADING INFORMATION:

Dead load (Dead load, Permanent, ULS-movability = 25.9 %):

Point load: 1:	FY = 0.53 kN	x = 2150.0 mm	(Partition)
Point load: 2:	FY = 0.16 kN	x = 8000.0 mm	(Partition)
Surface load: 1:	QY = 0.750 kN/m2	x = 0 - 9230 mm	

Partition load (Dead load, Permanent, ULS/SLS-movability = 100.0 %):

Line load: 1:	QY = 0.200 kN/m	x = 3680 - 8000 mm	(Short partition 1/2)
Line load: 2:	QY = 0.67 - 0.2 kN/m	x = 2150 - 3680 mm	(Slope up Partition 1/2)

Live load (Imposed load A-residential areas, Medium-term, ULS/SLS-movability = 100.0 %):

Surface load: 1: QY = 1.500 kN/m2 x = 0 - 9230 mm

Maximum utility rate: 85.1 %

GOVERNING DESIGN RESULTS:

Check:	Actual:	Allowable:	% allowable:	Location x:	
Shear (y):	3.65 kN	7.25 kN	50.3 %	1435 mm	Medium-term
Bending (Mz):	2.96 kNm	5.83 kNm	50.8 %	5290 mm	Medium-term
(without kcrit):	2.96 kNm	5.83 kNm	50.8 %	5290 mm	Medium-term
Utot,fin:	9.72 mm	16.56 mm	58.7 %	3230 mm	(characteristic)
Utot,inst:	6.34 mm	11.26 mm	56.3 %	7384 mm	(characteristic)

SUPPORT REACTIONS:

Support:	ULSmax:	ULSmin:	SLSmax:	SLSmin:	Rd/A:	Bearing:
1:	0.63 kN	-1.83 kN	0.28 kN	-1.22 kN	--	--
2:	6.81 kN	0.86 kN	4.73 kN	1.14 kN	1.99 N/mm2	59.0 %
3:	7.70 kN	1.58 kN	5.39 kN	1.58 kN	2.25 N/mm2	66.8 %
4:	2.50 kN	0.26 kN	1.72 kN	0.36 kN	1.46 N/mm2	43.3 %

- Upplift occurs, make sure of the anchoring
- "--" indicates that hangers are used

HANGERS:

Support:	Hanger:	Bearing:	Hanger name:

1	85.1 %	8.7 %	LBV240/40, to rectangle header all nail holes filled with web stiffeners

See construction details and refer to manufacturers literature for further

J16 Front Hall

Profile: FJI 38/240 (B=38 mm, H=240 mm)

Cantilever/Span:	Horizontal [mm]:
Span 1	1600.0
Total:	1600.0

Support:	Position x [mm]:	Width [mm]:	Type:
1:	0	0	Pinned support (X,Y)
2:	1600	0	Pinned support (Y)

LOADING INFORMATION:
Dead load (Dead load, Permanent, ULS-movability = 25.9 %):
Surface load: 1: QY = 0.750 kN/m2 x = 0 - 1600 mm

Live load (Imposed load A-residential areas, Medium-term, ULS/SLS-movability = 100.0 %):
Surface load: 1: QY = 1.500 kN/m2 x = 0 - 1600 mm

Maximum utility rate: 10.1 %

GOVERNING DESIGN RESULTS:

Check:	Actual:	Allowable:	% allowable:	Location x:	
Shear (y):	0.73 kN	7.25 kN	10.1 %	1360 mm	Medium-term
Bending (Mz):	0.42 kNm	5.83 kNm	7.2 %	800 mm	Medium-term
(without kcrit):	0.42 kNm	5.83 kNm	7.2 %	800 mm	Medium-term
Utot,fin:	0.50 mm	6.40 mm	7.9 %	800 mm	(characteristic)
Utot,inst:	0.33 mm	4.57 mm	7.2 %	800 mm	(characteristic)

SUPPORT REACTIONS:

Support:	ULSmax:	ULSmin:	SLSmax:	SLSmin:	Rd/A:	Bearing:
1:	1.04 kN	0.24 kN	0.72 kN	0.24 kN	--	--
2:	1.04 kN	0.24 kN	0.72 kN	0.24 kN	--	--

HANGERS:

Support:	Hanger:	Bearing:	Hanger name:
1	14.8 %	16.8 %	LBV240/40, to I-joist header with backer block
2	10.2 %	16.8 %	LBV240/40, to rectangle header

J17 Right Hall + Bed3

Profile: FJI 45/240 (B=45 mm, H=240 mm)

Cantilever/Span:	Horizontal [mm]:
Span 1	4960.0
Span 2	2620.0
Total:	7580.0

Support:	Position x [mm]:	Width [mm]:	Type:
1:	0	0	Pinned support (X,Y)
2:	4960	90	Pinned support (Y)
3:	7580	45	Pinned support (Y)

LOADING INFORMATION:

Dead load (Dead load, Permanent, ULS-movability = 25.9 %):

Point load: 1:	FY = 0.53 kN	x = 500.0 mm (Partition)	
Point load: 2:	FY = 0.16 kN	x = 6350.0 mm	(Partition)
Surface load: 1:	QY = 0.750 kN/m2	x = 0 - 7580 mm	

Live load (Imposed load A-residential areas, Medium-term, ULS/SLS-movability = 100.0 %):

Surface load: 1: QY = 1.500 kN/m2 x = 0 - 7580 mm

Maximum utility rate: 91.3 %

GOVERNING DESIGN RESULTS:

Check:	Actual:	Allowable:	% allowable:	Location x:	
Shear (y):	3.54 kN	7.51 kN	47.2 %	4675 mm	Medium-term
Bending (Mz):	3.00 kNm	7.02 kNm	42.7 %	4960 mm	Medium-term
(without kcrit):	3.00 kNm	7.02 kNm	42.7 %	4960 mm	Medium-term
Utot,fin:	15.43 mm	19.84 mm	77.8 %	2274 mm	(characteristic)
Utot,inst:	10.95 mm	12.00 mm	91.3 %	2274 mm	(characteristic)

SUPPORT REACTIONS:

Support:	ULSmax:	ULSmin:	SLSmax:	SLSmin:	Rd/A:	Bearing:
1:	3.33 kN	1.01 kN	2.32 kN	1.03 kN	--	--
2:	6.87 kN	1.71 kN	4.75 kN	1.71 kN	1.70 N/mm2	52.4 %
3:	1.42 kN	-0.56 kN	0.91 kN	-0.25 kN	0.70 N/mm2	21.7 %

- Upplift occurs, make sure of the anchoring

J18 Stair Framing

Material: KERTO-S 45x240 (B=45 mm, H=240 mm)

Cantilever/Span:	Horizontal [mm]:
Span 1	3260.0
Total:	3260.0

Support:	Position x [mm]:	Width [mm]:	Type:	
1:	0	63	Pinned support (X,Y)	
2:	3260	50	Pinned support (Y)	ITT239/47

LOADING INFORMATION:

Dead load (Dead load, Permanent, ULS-movability = 25.9 %):

Line load: 1:	QY = 0.075 kN/m	x = 0 - 1650 mm	(Floor 1/2)
Line load: 2:	QY = 0.150 kN/m	x = 1650 - 3260 mm	(Floor)

Partition load (Dead load, Permanent, ULS/SLS-movability = 100.0 %):

Point load: 1: FY = 0.36 kN x = 1650.0 mm (Stair Edge)

Live load (Imposed load A-residential areas, Medium-term, ULS/SLS-movability = 100.0 %):

Point load: 1:	FY = 2.41 kN	x = 1650.0 mm	(Stair edge)
Line load: 1:	QY = 0.150 kN/m	x = 0 - 1650 mm	(Floor 1/2)
Line load: 2:	QY = 0.300 kN/m	x = 1650 - 3260 mm	(Floor)

Maximum utility rate: 49.8 %

GOVERNING DESIGN RESULTS:

Check:	Actual:	Allowable:	% allowable:	Location x:

Shear (y):	2.85 kN	21.65 kN	13.1 %	3020 mm	Medium-term
Bending (Mz):	3.99 kNm	14.32 kNm	27.9 %	1650 mm	Medium-term
(without kcrit):	3.99 kNm	14.32 kNm	27.9 %	1650 mm	Medium-term
Utot,fin:	4.98 mm	13.04 mm	38.2 %	1650 mm	(characteristic)
Utot,inst:	3.98 mm	9.31 mm	42.8 %	1650 mm	(characteristic)

SUPPORT REACTIONS:

Support:	ULSmax:	ULSmin:	SLSmax:	SLSmin:	Rd/A:	Bearing:
1:	2.69 kN	0.15 kN	1.82 kN	0.15 kN	0.95 N/mm2	13.2 %
2:	3.00 kN	0.21 kN	2.04 kN	0.21 kN	--	--

- "--" indicates that hangers are used

HANGERS:

Support:	Hanger:	Bearing:	Hanger name:
2	49.8 %	18.5 %	ITT239/47, to rectangle header

J19A Front Hall

Profile: FJI 38/240 (B=38 mm, H=240 mm)

Cantilever/Span:	Horizontal [mm]:
Span 1	3180.0
Total:	3180.0

Support:	Position x [mm]:	Width [mm]:	Type:
1:	0	0	Pinned support (X,Y)
2:	3180	0	Pinned support (Y)

LOADING INFORMATION:

Dead load (Dead load, Permanent, ULS-movability = 25.9 %):

Surface load: 1: QY = 0.750 kN/m2 x = 0 - 3180 mm

Partition load (Dead load, Permanent, ULS/SLS-movability = 100.0 %):

Point load: 1: FY = 0.53 kN x = 1650.0 mm (High)

Live load (Imposed load A-residential areas, Medium-term, ULS/SLS-movability = 100.0 %):

Surface load: 1: QY = 1.500 kN/m2 x = 0 - 3180 mm

Maximum utility rate: 53.4 %

GOVERNING DESIGN RESULTS:

Check:	Actual:	Allowable:	% allowable:	Location x:	
Shear (y):	2.13 kN	7.25 kN	29.4 %	2940 mm	Medium-term
Bending (Mz):	2.22 kNm	5.83 kNm	38.0 %	1650 mm	Medium-term
(without kcrit):	2.22 kNm	5.83 kNm	38.0 %	1650 mm	Medium-term
Utot,fin:	6.80 mm	12.72 mm	53.4 %	1650 mm	(characteristic)
Utot,inst:	4.58 mm	9.09 mm	50.4 %	1650 mm	(characteristic)

SUPPORT REACTIONS:

Support:	ULSmax:	ULSmin:	SLSmax:	SLSmin:	Rd/A:	Bearing:
1:	2.42 kN	0.48 kN	1.69 kN	0.48 kN	--	--
2:	2.45 kN	0.48 kN	1.71 kN	0.48 kN	--	--

HANGERS:

Support:	Hanger:	Bearing:	Hanger name:

1	23.5 %	39.0 %	LBV240/40, to rectangle header
2	23.8 %	39.5 %	LBV240/40, to rectangle header

J19B Front Hall

Profile: FJI 38/240 (B=38 mm, H=240 mm)

Cantilever/Span:	Horizontal [mm]:
Span 1	3180.0
Total:	3180.0

Support:	Position x [mm]:	Width [mm]:	Type:
1:	0	0	Pinned support (X,Y)
2:	3180	0	Pinned support (Y)

LOADING INFORMATION:
Dead load (Dead load, Permanent, ULS-movability = 25.9 %):
Surface load: 1: QY = 0.750 kN/m2 x = 0 - 3180 mm

Partition load (Dead load, Permanent, ULS/SLS-movability = 100.0 %):
Point load: 1: FY = 0.53 kN x = 1650.0 mm (High)
Line load: 1: QY = 1.300 kN/m x = 1650 - 3180 mm (Hall)

Live load (Imposed load A-residential areas, Medium-term, ULS/SLS-movability = 100.0 %):
Surface load: 1: QY = 1.500 kN/m2 x = 0 - 3180 mm

Maximum utility rate: 86.4 %

GOVERNING DESIGN RESULTS:

Check:	Actual:	Allowable:	% allowable:	Location x:	
Shear (y):	2.54 kN	4.14 kN	61.2 %	2940 mm	Permanent
Bending (Mz):	3.29 kNm	5.83 kNm	56.4 %	1749 mm	Medium-term
(without kcrit):	3.29 kNm	5.83 kNm	56.4 %	1749 mm	Medium-term
Utot,fin:	10.99 mm	12.72 mm	86.4 %	1670 mm	(characteristic)
Utot,inst:	6.98 mm	9.09 mm	76.8 %	1650 mm	(characteristic)

SUPPORT REACTIONS:

Support:	ULSmax:	ULSmin:	SLSmax:	SLSmin:	Rd/A:	Bearing:
1:	3.07 kN	0.48 kN	2.16 kN	0.48 kN	--	--
2:	4.49 kN	0.48 kN	3.22 kN	0.48 kN	--	--

HANGERS:

Support:	Hanger:	Bearing:	Hanger name:
1	29.8 %	49.5 %	LBV240/40, to rectangle header
2	52.0 %	72.4 %	LBV240/40, to rectangle header

J20A Left Hall + Cupboards + Bed3

Profile: FJI 38/240 (B=38 mm, H=240 mm)

Cantilever/Span:	Horizontal [mm]:
Span 1	3280.0
Span 2	1380.0
Span 3	2280.0
Span 4	3890.0
Total:	10830.0

Support:	Position x [mm]:	Width [mm]:	Type:
1:	0	0	Pinned support (X,Y)
2:	3280	90	Pinned support (Y)
3:	4660	90	Pinned support (Y)
4:	6940	90	Pinned support (Y)
5:	10830	45	Pinned support (Y)

LOADING INFORMATION:

Dead load (Dead load, Permanent, ULS-movability = 25.9 %):

Point load: 1:	FY = 0.53 kN	x = 3700.0 mm	(Partition)
Point load: 2:	FY = 0.16 kN	x = 9600.0 mm	(Partition)
Surface load: 1:	QY = 0.750 kN/m2	x = 0 - 10830 mm	

Partition load (Dead load, Permanent, ULS/SLS-movability = 100.0 %):

Point load: 1:	FY = 0.27 kN	x = 1650.0 mm	(High)
Line load: 1:	QY = 0.670 kN/m	x = 1650 - 3700 mm	(High)

Live load (Imposed load A-residential areas, Medium-term, ULS/SLS-movability = 100.0 %):

Surface load: 1:	QY = 1.500 kN/m2	x = 0 - 10830 mm

Maximum utility rate: 61.8 %

GOVERNING DESIGN RESULTS:

Check:	Actual:	Allowable:	% allowable:	Location x:	
Shear (y):	3.35 kN	7.25 kN	46.2 %	2995 mm	Medium-term
Bending (Mz):	1.88 kNm	5.83 kNm	32.3 %	9206 mm	Medium-term
(without kcrit):	1.88 kNm	5.83 kNm	32.3 %	9206 mm	Medium-term
Utot,fin:	7.61 mm	15.56 mm	48.9 %	8935 mm	(characteristic)
Utot,inst:	5.27 mm	11.11 mm	47.4 %	8935 mm	(characteristic)

SUPPORT REACTIONS:

Support:	ULSmax:	ULSmin:	SLSmax:	SLSmin:	Rd/A:	Bearing:
1:	2.17 kN	0.37 kN	1.51 kN	0.38 kN	--	--
2:	7.13 kN	1.11 kN	5.04 kN	1.20 kN	2.09 N/mm2	61.8 %
3:	2.65 kN	-1.57 kN	1.69 kN	-0.85 kN	0.77 N/mm2	22.9 %
4:	5.34 kN	1.22 kN	3.68 kN	1.24 kN	1.56 N/mm2	46.3 %
5:	2.27 kN	0.53 kN	1.57 kN	0.55 kN	1.33 N/mm2	39.4 %

- Upplift occurs, make sure of the anchoring

HANGERS:

Support:	Hanger:	Bearing:	Hanger name:
1	41.2 %	38.5 %	IUT217/40, to rectangle header

J20B Left Hall + Cupboards + Bed3

Profile: FJI 38/240 (B=38 mm, H=240 mm)

Cantilever/Span:	Horizontal [mm]:
Span 1	3280.0
Span 2	1380.0
Span 3	2280.0
Span 4	3890.0
Total:	10830.0

Support:	Position x [mm]:	Width [mm]:	Type:
1:	0	0	Pinned support (X,Y)
2:	3280	90	Pinned support (Y)
3:	4660	90	Pinned support (Y)
4:	6940	90	Pinned support (Y)
5:	10830	45	Pinned support (Y)

LOADING INFORMATION:

Dead load (Dead load, Permanent, ULS-movability = 25.9 %):

Point load: 1:	FY = 0.53 kN	x = 3700.0 mm	(Partition)
Point load: 2:	FY = 0.16 kN	x = 9600.0 mm	(Partition)
Surface load: 1:	QY = 0.750 kN/m2	x = 0 - 10830 mm	

Live load (Imposed load A-residential areas, Medium-term, ULS/SLS-movability = 100.0 %):

Surface load: 1:	QY = 1.500 kN/m2	x = 0 - 10830 mm

Maximum utility rate: 48.8 %

GOVERNING DESIGN RESULTS:

Check:	Actual:	Allowable:	% allowable:	Location x:	
Shear (y):	2.70 kN	7.25 kN	37.2 %	7225 mm	Medium-term
Bending (Mz):	1.88 kNm	5.83 kNm	32.3 %	9206 mm	Medium-term
(without kcrit):	1.88 kNm	5.83 kNm	32.3 %	9206 mm	Medium-term
Utot,fin:	7.59 mm	15.56 mm	48.8 %	8935 mm	(characteristic)
Utot,inst:	5.27 mm	11.11 mm	47.4 %	8935 mm	(characteristic)

SUPPORT REACTIONS:

Support:	ULSmax:	ULSmin:	SLSmax:	SLSmin:	Rd/A:	Bearing:
1:	1.80 kN	0.38 kN	1.23 kN	0.39 kN	--	--
2:	4.91 kN	1.11 kN	3.39 kN	1.20 kN	1.44 N/mm2	42.6 %
3:	2.58 kN	-1.12 kN	1.65 kN	-0.52 kN	0.76 N/mm2	22.4 %
4:	5.33 kN	1.22 kN	3.67 kN	1.24 kN	1.56 N/mm2	46.2 %
5:	2.27 kN	0.53 kN	1.57 kN	0.55 kN	1.33 N/mm2	39.4 %

- Upplift occurs, make sure of the anchoring

HANGERS:

Support:	Hanger:	Bearing:	Hanger name:
1	34.0 %	31.8 %	IUT217/40, to rectangle header

J20C Left Hall + Cupboards + Bed3

Profile: FJI 38/240 (B=38 mm, H=240 mm)

Cantilever/Span:	Horizontal [mm]:
Span 1	3280.0
Span 2	1380.0
Span 3	2280.0
Span 4	3890.0
Total:	10830.0

Support:	Position x [mm]:	Width [mm]:	Type:
1:	0	0	Pinned support (X,Y)
2:	3280	90	Pinned support (Y)
3:	4660	90	Pinned support (Y)
4:	6940	90	Pinned support (Y)

5:	10830	45	Pinned support (Y)

LOADING INFORMATION:

Dead load (Dead load, Permanent, ULS-movability = 25.9 %):

Point load: 1:	FY = 0.53 kN	x = 3700.0 mm	(Partition)
Point load: 2:	FY = 0.16 kN	x = 9600.0 mm	(Partition)
Surface load: 1:	QY = 0.750 kN/m2	x = 0 - 10830 mm	

Partition load (Dead load, Permanent, ULS/SLS-movability = 100.0 %):

Line load: 1:	QY = 0.670 kN/m	x = 0 - 3700 mm	(High)

Live load (Imposed load A-residential areas, Medium-term, ULS/SLS-movability = 100.0 %):

Surface load: 1:	QY = 1.500 kN/m2	x = 0 - 10830 mm

Maximum utility rate: 65.3 %

GOVERNING DESIGN RESULTS:

Check:	Actual:	Allowable:	% allowable:	Location x:	
Shear (y):	2.13 kN	4.14 kN	51.5 %	2995 mm	Permanent
Bending (Mz):	2.10 kNm	5.83 kNm	36.1 %	1354 mm	Medium-term
(without kcrit):	2.10 kNm	5.83 kNm	36.1 %	1354 mm	Medium-term
Utot,fin:	7.55 mm	13.12 mm	57.6 %	1624 mm	(characteristic)
Utot,inst:	4.62 mm	9.37 mm	49.3 %	1624 mm	(characteristic)

SUPPORT REACTIONS:

Support:	ULSmax:	ULSmin:	SLSmax:	SLSmin:	Rd/A:	Bearing:
1:	3.05 kN	0.37 kN	2.16 kN	0.38 kN	--	--
2:	7.54 kN	1.11 kN	5.34 kN	1.20 kN	2.20 N/mm2	65.3 %
3:	2.65 kN	-1.72 kN	1.69 kN	-0.97 kN	0.77 N/mm2	22.9 %
4:	5.35 kN	1.22 kN	3.69 kN	1.24 kN	1.56 N/mm2	46.4 %
5:	2.27 kN	0.53 kN	1.57 kN	0.55 kN	1.33 N/mm2	39.4 %

- Upplift occurs, make sure of the anchoring

HANGERS:

Support:	Hanger:	Bearing:	Hanger name:
1	59.6 %	54.0 %	IUT217/40, to rectangle header

J22A Back Hall + Bath + Es3 + Bed3

Profile: FJI 38/240 (B=38 mm, H=240 mm)

Cantilever/Span:	Horizontal [mm]:
Span 1	1470.0
Span 2	1780.0
Span 3	1870.0
Span 4	3890.0
Total:	9010.0

Support:	Position x [mm]:	Width [mm]:	Type:
1:	0	63	Pinned support (X,Y)
2:	1470	130	Pinned support (Y)
3:	3250	130	Pinned support (Y)
4:	5120	130	Pinned support (Y)
5:	9010	63	Pinned support (Y)

LOADING INFORMATION:

Dead load (Dead load, Permanent, ULS-movability = 25.9 %):
Surface load: 1: QY = 0.750 kN/m2 x = 0 - 9010 mm

Partition load (Dead load, Permanent, ULS/SLS-movability = 100.0 %):

Point load: 1:	FY = 0.53 kN	x = 2000.0 mm	(High)
Point load: 2:	FY = 0.16 kN	x = 7700.0 mm	(Low)
Line load: 1:	QY = 0.670 kN/m	x = 0 - 1850 mm	(Partition High)

Live load (Imposed load A-residential areas, Medium-term, ULS/SLS-movability = 100.0 %):
Surface load: 1: QY = 1.500 kN/m2 x = 0 - 9010 mm

Maximum utility rate: 48.9 %

GOVERNING DESIGN RESULTS:

Check:	Actual:	Allowable:	% allowable:	Location x:	
Shear (y):	2.67 kN	7.25 kN	36.9 %	5425 mm	Medium-term
Bending (Mz):	1.88 kNm	5.83 kNm	32.2 %	7433 mm	Medium-term
(without kcrit):	1.88 kNm	5.83 kNm	32.2 %	7433 mm	Medium-term
Utot,fin:	7.60 mm	15.56 mm	48.9 %	7208 mm	(characteristic)
Utot,inst:	5.26 mm	11.11 mm	47.3 %	7208 mm	(characteristic)

SUPPORT REACTIONS:

Support:	ULSmax:	ULSmin:	SLSmax:	SLSmin:	Rd/A:	Bearing:
1:	1.46 kN	-0.03 kN	1.02 kN	0.04 kN	0.61 N/mm2	19.8 %
2:	4.27 kN	0.43 kN	3.02 kN	0.48 kN	0.86 N/mm2	28.4 %
3:	2.67 kN	-0.70 kN	1.77 kN	-0.30 kN	0.54 N/mm2	17.8 %
4:	5.24 kN	1.02 kN	3.62 kN	1.06 kN	1.06 N/mm2	34.9 %
5:	2.26 kN	0.45 kN	1.56 kN	0.47 kN	0.94 N/mm2	30.7 %

- Upplift occurs, make sure of the anchoring

J22B Back Hall + Bath + Es3 + Bed3

Profile: FJI 38/240 (B=38 mm, H=240 mm)

Cantilever/Span:	Horizontal [mm]:
Span 1	1470.0
Span 2	1780.0
Span 3	1870.0
Span 4	3890.0
Total:	9010.0

Support:	Position x [mm]:	Width [mm]:	Type:
1:	0	63	Pinned support (X,Y)
2:	1470	130	Pinned support (Y)
3:	3250	130	Pinned support (Y)
4:	5120	130	Pinned support (Y)
5:	9010	63	Pinned support (Y)

LOADING INFORMATION:
Dead load (Dead load, Permanent, ULS-movability = 25.9 %):
Surface load: 1: QY = 0.750 kN/m2 x = 0 - 9010 mm

Partition load (Dead load, Permanent, ULS/SLS-movability = 100.0 %):

Point load: 1:	FY = 0.53 kN	x = 2000.0 mm	(High)
Point load: 2:	FY = 0.16 kN	x = 7700.0 mm	(Low)

Live load (Imposed load A-residential areas, Medium-term, ULS/SLS-movability = 100.0 %):
Surface load: 1: QY = 1.500 kN/m2 x = 0 - 9010 mm

Maximum utility rate: 48.8 %

GOVERNING DESIGN RESULTS:

Check:	Actual:	Allowable:	% allowable:	Location x:	
Shear (y):	2.67 kN	7.25 kN	36.9 %	5425 mm	Medium-term
Bending (Mz):	1.88 kNm	5.83 kNm	32.2 %	7433 mm	Medium-term
(without kcrit):	1.88 kNm	5.83 kNm	32.2 %	7433 mm	Medium-term
Utot,fin:	7.60 mm	15.56 mm	48.8 %	7208 mm	(characteristic)
Utot,inst:	5.26 mm	11.11 mm	47.3 %	7208 mm	(characteristic)

SUPPORT REACTIONS:

Support:	ULSmax:	ULSmin:	SLSmax:	SLSmin:	Rd/A:	Bearing:
1:	0.85 kN	-0.02 kN	0.58 kN	0.05 kN	0.36 N/mm2	11.6 %
2:	3.16 kN	0.43 kN	2.20 kN	0.48 kN	0.64 N/mm2	21.0 %
3:	2.63 kN	-0.64 kN	1.74 kN	-0.25 kN	0.53 N/mm2	17.5 %
4:	5.24 kN	1.02 kN	3.61 kN	1.07 kN	1.06 N/mm2	34.8 %
5:	2.26 kN	0.45 kN	1.56 kN	0.47 kN	0.94 N/mm2	30.7 %

- Upplift occurs, make sure of the anchoring

J22C Back Hall + Bath + Es3 + Bed3

Profile: FJI 38/240 (B=38 mm, H=240 mm)

Cantilever/Span:	Horizontal [mm]:
Span 1	1470.0
Span 2	1780.0
Span 3	1870.0
Span 4	3890.0
Total:	9010.0

Support:	Position x [mm]:	Width [mm]:	Type:
1:	0	63	Pinned support (X,Y)
2:	1470	130	Pinned support (Y)
3:	3250	130	Pinned support (Y)
4:	5120	130	Pinned support (Y)
5:	9010	63	Pinned support (Y)

LOADING INFORMATION:

Dead load (Dead load, Permanent, ULS-movability = 25.9 %):
Surface load: 1: QY = 0.750 kN/m2 x = 0 - 9010 mm

Partition load (Dead load, Permanent, ULS/SLS-movability = 100.0 %):

Point load: 1:	FY = 0.26 kN	x = 2000.0 mm	(High)
Point load: 2:	FY = 0.08 kN	x = 7700.0 mm	(Low)
Line load: 1:	QY = 0.670 kN/m	x = 0 - 5500 mm	(High)
Line load: 2:	QY = 0.200 kN/m	x = 5500 - 9010 mm	(Low)

Live load (Imposed load A-residential areas, Medium-term, ULS/SLS-movability = 100.0 %):
Surface load: 1: QY = 1.500 kN/m2 x = 0 - 9010 mm

Maximum utility rate: 60.2 %

GOVERNING DESIGN RESULTS:

Check:	Actual:	Allowable:	% allowable:	Location x:	
Shear (y):	3.22 kN	7.25 kN	44.4 %	5425 mm	Medium-term
Bending (Mz):	2.20 kNm	5.83 kNm	37.8 %	5120 mm	Medium-term
(without kcrit):	2.20 kNm	5.83 kNm	37.8 %	5120 mm	Medium-term
Utot,fin:	9.37 mm	15.56 mm	60.2 %	7208 mm	(characteristic)
Utot,inst:	6.24 mm	11.11 mm	56.2 %	7208 mm	(characteristic)

SUPPORT REACTIONS:

Support:	ULSmax:	ULSmin:	SLSmax:	SLSmin:	Rd/A:	Bearing:
1:	1.47 kN	-0.09 kN	1.04 kN	-0.00 kN	0.61 N/mm2	20.0 %
2:	4.59 kN	0.31 kN	3.25 kN	0.40 kN	0.93 N/mm2	30.5 %
3:	4.37 kN	-0.92 kN	3.03 kN	-0.46 kN	0.88 N/mm2	29.1 %
4:	7.07 kN	0.96 kN	4.97 kN	1.02 kN	1.43 N/mm2	47.0 %
5:	2.65 kN	0.43 kN	1.85 kN	0.45 kN	1.10 N/mm2	36.0 %

- Upplift occurs, make sure of the anchoring

J24 Bed2 + Bed3

Profile: FJI 45/240 (B=45 mm, H=240 mm)

Cantilever/Span:	Horizontal [mm]:
Span 1	5120.0
Span 2	3890.0
Total:	9010.0

Support:	Position x [mm]:	Width [mm]:	Type:
1:	0	45	Pinned support (X,Y)
2:	5120	90	Pinned support (Y)
3:	9010	45	Pinned support (Y)

LOADING INFORMATION:

Dead load (Dead load, Permanent, ULS-movability = 25.9 %):

Surface load: 1: QY = 0.700 kN/m2 x = 0 - 9010 mm

Live load (Imposed load A-residential areas, Medium-term, ULS/SLS-movability = 100.0 %):

Surface load: 1: QY = 1.000 kN/m2 x = 0 - 9010 mm

Maximum utility rate: 73.5 %

GOVERNING DESIGN RESULTS:

Check:	Actual:	Allowable:	% allowable:	Location x:	
Shear (y):	2.72 kN	7.51 kN	36.2 %	4835 mm	Medium-term
Bending (Mz):	2.53 kNm	7.02 kNm	36.0 %	5120 mm	Medium-term
(without kcrit):	2.53 kNm	7.02 kNm	36.0 %	5120 mm	Medium-term
Utot,fin:	12.44 mm	20.48 mm	60.7 %	2478 mm	(characteristic)
Utot,inst:	8.82 mm	12.00 mm	73.5 %	2478 mm	(characteristic)

SUPPORT REACTIONS:

Support:	ULSmax:	ULSmin:	SLSmax:	SLSmin:	Rd/A:	Bearing:
1:	2.12 kN	0.47 kN	1.46 kN	0.51 kN	1.05 N/mm2	32.3 %
2:	5.55 kN	1.59 kN	3.86 kN	1.59 kN	1.37 N/mm2	42.4 %
3:	1.57 kN	0.04 kN	1.06 kN	0.17 kN	0.78 N/mm2	24.0 %

J25 Es2 + Bed2

Profile: FJI 45/240 (B=45 mm, H=240 mm)

Cantilever/Span:	Horizontal [mm]:
Span 1	1840.0
Span 2	5130.0
Total:	6970.0

Support:	Position x [mm]:	Width [mm]:	Type:
1:	0	63	Pinned support (X,Y)
2:	1840	126	Pinned support (Y)
3:	6970	63	Pinned support (Y)

LOADING INFORMATION:

Dead load (Dead load, Permanent, ULS-movability = 25.9 %):

Surface load: 1: QY = 0.700 kN/m2 x = 0 - 6970 mm

Live load (Imposed load A-residential areas, Medium-term, ULS/SLS-movability = 100.0 %):

Surface load: 1: QY = 1.000 kN/m2 x = 0 - 6970 mm

Maximum utility rate: 71.1 %

GOVERNING DESIGN RESULTS:

Check:	Actual:	Allowable:	% allowable:	Location x:	
Shear (y):	2.66 kN	7.51 kN	35.4 %	2143 mm	Medium-term
Bending (Mz):	2.30 kNm	7.02 kNm	32.8 %	1840 mm	Medium-term
(without kcrit):	2.30 kNm	7.02 kNm	32.8 %	1840 mm	Medium-term
Utot,fin:	12.31 mm	20.52 mm	60.0 %	4530 mm	(characteristic)
Utot,inst:	8.53 mm	12.00 mm	71.1 %	4530 mm	(characteristic)

SUPPORT REACTIONS:

Support:	ULSmax:	ULSmin:	SLSmax:	SLSmin:	Rd/A:	Bearing:
1:	0.50 kN	-0.95 kN	0.24 kN	-0.59 kN	0.18 N/mm2	6.0 %
2:	5.11 kN	1.46 kN	3.55 kN	1.46 kN	0.90 N/mm2	30.6 %
3:	2.07 kN	0.58 kN	1.44 kN	0.58 kN	0.73 N/mm2	24.9 %

- Upplift occurs, make sure of the anchoring

JS01 Stair Framing

Profile: KERTO-S 45x240 (B=45 mm, H=240 mm)

Cantilever/span lengths:

Cantilever/Span:	Horizontal [mm]:
Span 1	3260.0
Total:	3260.0

Support:	Position x [mm]:	Width [mm]:	Type:	
1:	0	57	Pinned support (X,Y)	LBV240/47
2:	3260	50	Pinned support (Y)	IUT217/47

LOADING INFORMATION:

Dead load (Dead load, Permanent, ULS-movability = 25.9 %):

Point load: 1:	FY = -0.26 kN	x = 191.0 mm (J15)
Point load: 2:	FY = -0.26 kN	x = 591.0 mm (J15)
Point load: 3:	FY = -0.26 kN	x = 991.0 mm (J15)
Point load: 4:	FY = -0.26 kN	x = 1391.0 mm (J15)

Point load: 5:	FY = -0.26 kN	x = 1791.0 mm	(J15)
Point load: 6:	FY = -0.26 kN	x = 2191.0 mm	(J15)
Point load: 7:	FY = -0.26 kN	x = 2591.0 mm	(J15)
Point load: 8:	FY = -0.26 kN	x = 2991.0 mm	(J15)

Partition load (Dead load, Permanent, ULS/SLS-movability = 100.0 %):

Line load: 1:	QY = 1.340 - 0.400 kN/m	x = 0 - 2110 mm	(PArtition high)
Line load: 2:	QY = 0.400 kN/m	x = 2110 - 3260 mm	(Partition Low)

Live load (Imposed load A-residential areas, Medium-term, ULS/SLS-movability = 100.0 %):

Point load: 1:	FY = 0.50 kN	x = 191.0 mm (J15)	
Point load: 2:	FY = 0.50 kN	x = 591.0 mm (J15)	
Point load: 3:	FY = 0.50 kN	x = 991.0 mm (J15)	
Point load: 4:	FY = 0.50 kN	x = 1391.0 mm	(J15)
Point load: 5:	FY = 0.50 kN	x = 1791.0 mm	(J15)
Point load: 6:	FY = 0.50 kN	x = 2191.0 mm	(J15)
Point load: 7:	FY = 0.50 kN	x = 2591.0 mm	(J15)
Point load: 8:	FY = 0.50 kN	x = 2991.0 mm	(J15)

Maximum utility rate: 77.9 %

GOVERNING DESIGN RESULTS:

Check:	Actual:	Allowable:	% allowable:	Location x:	
Shear (y):	3.94 kN	21.65 kN	18.2 %	0 mm	Medium-term
Bending (Mz):	2.82 kNm	14.32 kNm	19.7 %	1467 mm	Medium-term
(without kcrit):	2.82 kNm	14.32 kNm	19.7 %	1467 mm	Medium-term
Utot,fin:	3.44 mm	13.04 mm	26.4 %	1548 mm	(characteristic)
Utot,inst:	2.91 mm	9.31 mm	31.2 %	1630 mm	(characteristic)

SUPPORT REACTIONS:

Support:	ULSmax:	ULSmin:	SLSmax:	SLSmin:	Rd/A:	Bearing:
1:	3.94 kN	-1.44 kN	2.41 kN	-1.06 kN	--	--
2:	3.08 kN	-1.37 kN	1.80 kN	-1.02 kN	--	--

- Upplift occurs, make sure of the anchoring
- "--" indicates that hangers are used

HANGERS:

Support:	Hanger:	Bearing:	Hanger name:
1	77.9 %	22.5 %	LBV240/47, to rectangle header all nail holes filled with web stiffeners
2	74.2 %	19.0 %	IUT217/47, to I-joist header all nail holes filled with backer block and web stiffeners

See construction details and refer to manufacturers literature for further information

JS02 Left/Right Hall Junction

Profile: KERTO-S 39x240 (B=39 mm, H=240 mm)

Cantilever/Span:	Horizontal [mm]:
Span 1	1460.0
Total:	1460.0

Support:	Position x [mm]:	Width [mm]:	Type:
1:	0	126	Pinned support (X,Y)
2:	1460	63	Pinned support (Y)

LOADING INFORMATION:

Dead load (Dead load, Permanent, ULS-movability = 25.9 %):

Point load: 1:	FY = 1.09 kN	x = 380.0 mm (Right Hall)	
Point load: 2:	FY = 1.09 kN	x = 780.0 mm (Right Hall)	
Point load: 3:	FY = 1.09 kN	x = 1180.0 mm	(Right Hall)
Point load: 4:	FY = 0.48 kN	x = 390.0 mm (Left Hall)	
Point load: 5:	FY = 0.48 kN	x = 790.0 mm (Left Hall)	
Point load: 6:	FY = 0.66 kN	x = 1020.0 mm	(Left Hall)
Point load: 7:	FY = 0.24 kN	x = 1280.0 mm	(Left Hall)

Live load (Imposed load A-residential areas, Medium-term, ULS/SLS-movability = 100.0 %):

Point load: 1:	FY = 1.27 kN	x = 380.0 mm (Right Hall)	
Point load: 2:	FY = 1.27 kN	x = 780.0 mm (Right Hall)	
Point load: 3:	FY = 1.27 kN	x = 1180.0 mm	(Right Hall)
Point load: 4:	FY = 0.95 kN	x = 390.0 mm (Left Hall)	
Point load: 5:	FY = 0.95 kN	x = 790.0 mm (Left Hall)	
Point load: 6:	FY = 1.31 kN	x = 1020.0 mm	(Left Hall)
Point load: 7:	FY = 0.48 kN	x = 1280.0 mm	(Front Hall)

Maximum utility rate: 56.6 %

GOVERNING DESIGN RESULTS:

Check:	Actual:	Allowable:	% allowable:	Location x:	
Shear (y):	10.00 kN	18.76 kN	53.3 %	1387 mm	Medium-term
Bending (Mz):	4.22 kNm	12.41 kNm	34.0 %	780 mm	Medium-term
(without kcrit):	4.22 kNm	12.41 kNm	34.0 %	780 mm	Medium-term
Utot,fin:	2.20 mm	5.84 mm	37.6 %	780 mm	(characteristic)
Utot,inst:	1.63 mm	4.17 mm	39.0 %	780 mm	(characteristic)

SUPPORT REACTIONS:

Support:	ULSmax:	ULSmin:	SLSmax:	SLSmin:	Rd/A:	Bearing:
1:	8.18 kN	2.32 kN	5.68 kN	2.32 kN	1.66 N/mm2	28.7 %
2:	10.00 kN	2.81 kN	6.95 kN	2.81 kN	4.07 N/mm2	56.6 %

JS03 Right Hall

Profile: KERTO-S 39x240 (B=39 mm, H=240 mm)

Cantilever/Span:	Horizontal [mm]:
Span 1	1440.0
Total:	1440.0

Support:	Position x [mm]:	Width [mm]:	Type:
1:	0	63	Pinned support (X,Y)
2:	1440	63	Pinned support (Y)

LOADING INFORMATION:

Dead load (Dead load, Permanent, ULS-movability = 25.9 %):

Point load: 1:	FY = 0.73 kN	x = 80.0 mm (Right Side)	
Point load: 2:	FY = 0.73 kN	x = 480.0 mm (Right Side)	
Point load: 3:	FY = 0.73 kN	x = 880.0 mm (Right Side)	
Point load: 4:	FY = 0.73 kN	x = 1280.0 mm	(Right Side)
Point load: 5:	FY = 0.48 kN	x = 400.0 mm (Left Side)	
Point load: 6:	FY = 0.48 kN	x = 740.0 mm (Left Side)	
Point load: 7:	FY = 0.48 kN	x = 1200.0 mm	(Left Side)

Live load (Imposed load A-residential areas, Medium-term, ULS/SLS-movability = 100.0 %):

Point load: 1:	FY = 1.46 kN	x = 80.0 mm (Right Side)	
Point load: 2:	FY = 1.46 kN	x = 480.0 mm (Right Side)	
Point load: 3:	FY = 1.46 kN	x = 880.0 mm (Right Side)	
Point load: 4:	FY = 1.46 kN	x = 1280.0 mm	(Right Side)
Point load: 5:	FY = 0.95 kN	x = 400.0 mm (Left Side)	
Point load: 6:	FY = 0.95 kN	x = 740.0 mm (Left Side)	
Point load: 7:	FY = 0.95 kN	x = 1200.0 mm	(Left Side)

Maximum utility rate: 59.5 %

GOVERNING DESIGN RESULTS:

Check:	Actual:	Allowable:	% allowable:	Location x:	
Shear (y):	9.55 kN	17.06 kN	56.0 %	36 mm	Medium-term
Bending (Mz):	3.44 kNm	11.28 kNm	30.5 %	740 mm	Medium-term
(without kcrit):	3.44 kNm	11.28 kNm	30.5 %	740 mm	Medium-term
Utot,fin:	1.75 mm	5.76 mm	30.4 %	740 mm	(characteristic)
Utot,inst:	1.33 mm	4.11 mm	32.2 %	740 mm	(characteristic)

SUPPORT REACTIONS:

Support:	ULSmax:	ULSmin:	SLSmax:	SLSmin:	Rd/A:	Bearing:
1:	9.55 kN	2.20 kN	6.59 kN	2.20 kN	3.89 N/mm2	59.5 %
2:	9.37 kN	2.16 kN	6.46 kN	2.16 kN	3.81 N/mm2	58.3 %

Ground floor walls

Design conditions

Service class $\text{Class} = 1$

Design category $\text{DesignCat} = \text{"A"}$

$\psi_0 = \psi\text{val}(\text{DesignCat}, 0) = 0.7$

$\psi_1 = \psi\text{val}(\text{DesignCat}, 1) = 0.5$

$\psi_2 = \psi\text{val}(\text{DesignCat}, 2) = 0.3$

Principle load duration $\text{LoadDuration} = \textit{Medium}$

Walls are timber framed from 63mm x 38mm CLS with top & bottom plates constructed from 2 pieces laid flat, glued and nailed together. Support posts are 38mm wide @ 0.400m centers 2.66m high and aligned or midway between joists. First floor joists are mostly 45mm wide @ 0.4m centers. Room facing sides of studs are restrained by 38 x 63mm counter battens @ 600mm centers.

Drawings **4 - Ground floor plan**
14 - FF loads on GF

Spreadsheet **Joists**

Top Plate width $b = 63 \cdot mm$

Top Plate height $h = 2 \times 38 \cdot mm$

Span between loads $span = 0.4m$

Stud width $b_s = 38 \cdot mm$

Stud depth $h_s = b = 63 \cdot mm$

Stud length $l_s = 2.66 \cdot m$

Spacing between couner battens $Cbs = 0.6 \cdot m$

Width of joist $b_j = 45 \cdot mm$

$\text{Material} = \text{"Softwood C16"}$

Material safety factor $\gamma_M = \text{get\_k}(\text{Material}, \text{Class}, \gamma_M) = 1.3$

Duration safty factor $k_{mod} = \text{get\_k}(\text{Material}, \text{Class}, k_{mod} + \text{LoadDuration}) = 0.8$

Final deformation factor $k_{def} = \text{get\_k}(\text{Material}, \text{Class}, k_{def}) = 0.6$

Characteristic material properties $f_{m.k} = \text{Tc}(\text{Material}, f_{m.0.k})\ MPa$

$f_{c.0.k} = \text{Tc}(\text{Material}, f_{c.0.k})\ MPa$

$f_{c.90.k} = \text{Tc}(\text{Material}, f_{c.90.k})\ MPa$

$f_{t.0.k} = \text{Tc}(\text{Material}, f_{t.0.k})\ MPa$

$E_{0.mean} = \text{Tc}(\text{Material}, E_{0.mean})\ MPa$

$E_{0.05} = \text{Tc}(\text{Material}, E_{0.05})\ MPa$

$G_{0.mean} = \text{Tc}(\text{Material}, G_{mean})\ MPa$

$k_{sys} = 1.0$

Height modification

$$k_h = \max\left[1, \min\left[\left(\frac{Tc(Material, k_{h.d})}{h \div mm}\right)^{Tc(Material, k_{h.s})}, Tc(Material, k_{h.max})\right]\right] = 1.15$$

Design material properties

$$f_{m.d} = \frac{f_{m.k} \cdot k_{mod} \cdot k_h \cdot k_{sys}}{\gamma_M}$$

$$f_{t.0.d} = \frac{f_{t.0.k} \cdot k_{mod} \cdot k_h \cdot k_{sys}}{\gamma_M}$$

$$f_{c.0.d} = \frac{f_{c.0.k} \cdot k_{mod} \cdot k_{sys}}{\gamma_M}$$

$$f_{c.90.d} = \frac{f_{c.90.k} \cdot k_{mod} \cdot k_{sys}}{\gamma_M}$$

Studs

Area of a stud $A_s = b_s \cdot h_s = 2.39 \times 10^3 \cdot mm^2$

Compression strength of a stud. $F_{c.s} = A_s \times f_{c.0.d} = 25.04 \cdot kN$

Buckling strength of a stud about the y-y axis, $F_{b.y.s} = A_s \cdot f_{c.0.d} \cdot calc\_k_c\left(h_s, l_s, f_{c.0.k}, E_{0.05}\right) = 3.4 \cdot kN$

Buckling strength of a stud about the z-z axis, $F_{b.z.s} = A_s \cdot f_{c.0.d} \cdot calc\_k_c\left(b_s, Cbs, f_{c.0.k}, E_{0.05}\right) = 17.69 \cdot kN$

Maximum compressive load in a stud $F_{c.63C16} = \min\left(F_{c.s}, F_{b.y.s}, F_{b.z.s}\right) = 3.4 \cdot kN$

Top plates

$$I_y = \frac{h^3 \cdot b}{12} = 230.46 \cdot cm^4$$

$$W_{el.y} = \frac{I_y}{(h \div 2)}$$

$$M_{max} = f_{m.d} \cdot W_{el.y} = 0.68 \cdot kN \cdot m$$

$$V_{max} = f_{v.d} \cdot h \cdot b \cdot \left(\frac{2}{3}\right) = 12.75 \cdot kN$$

Maximum bending occurs when the point load is central.

So maximum load $L_{bend.max} = \frac{M_{max}}{(span \div 2)} \times 2 = 6.84 \cdot kN$

Maximum shear load will be the same as V_{max} $L_{shear.max} = V_{max} = 12.75 \cdot kN$

Bearing strength for each joist. As this will be aligned with a stud no enhancment of bearing area is allowed and area is limited by the size of the stud itself.

$$k_{c.90} = 1.0$$

$$A_{ef} = b \cdot b_s = 2394 \cdot mm^2$$

$$L_{bear.max} = k_{c.90} \cdot f_{c.90.d} \cdot A_{ef} = 3.24 \cdot kN$$

$L_{max} = \min(L_{bend.max}, L_{shear.max}, L_{bear.max}, F_{b.y.s}, F_{b.z.s}) = 3.24 \cdot kN$

Instantanious deflection of the header @ L_{max} $\quad \dfrac{(L_{max} \cdot span^3)}{48 \cdot E_{0.mean} \cdot I_y} \times \left[1 + 1.2 \cdot \dfrac{E_{0.mean}}{G_{0.mean}} \cdot \left(\dfrac{h}{span}\right)^2\right] = 0.4 \cdot mm$

This is the ULS support reaction of the Joist which can be supported by the wall. On internal walls made up of two paralell walls the total load can be doubled.

This is sufficient for nearly all the first floor joist loads in the building. The problems occur each side of the entertainment room as this only has single walls supporting the load, the wide part of the gallery and with the bearings of JS02 and JS03.
JS03 with relativly large bearing area >200mm long should not be a problem.

$38 \cdot mm \times 200 \cdot mm \times f_{c.90.d} = 10.29 \cdot kN$

JS02 will require the wall to have a post brought right up to support the beam ULS load of 10 kN.

The maximum compressive force on a stud is $\quad F_{c.s} = 25.04 \cdot kN \quad$ so this O.K.

These posts are braced by aditional posts either side so the buckling resistance will be trebled.

$3 \cdot F_{b.y.s} = 10.21 \cdot kN$

Now for the entertainment room walls we will upgrade the timber used to 89mm C24 and position all loads centrally between studs spaced @ 400mm. This will give the following values.

$h = 2 \times 38 \cdot mm \qquad b = 89mm \qquad h_s = 89mm \qquad b_s = 38mm$

Material = "Softwood C24"

Material safety factor $\quad \gamma_M = get\_k(Material, Class, \gamma_M) = 1.3$

Duration safty factor $\quad k_{mod} = get\_k(Material, Class, k_{mod} + LoadDuration) = 0.8$

Final deformation factor $\quad k_{def} = get\_k(Material, Class, k_{def}) = 0.6$

Characteristic material properties $\quad f_{m.k} = Tc(Material, f_{m.0.k})\ MPa$

$f_{c.0.k} = Tc(Material, f_{c.0.k})\ MPa$

$f_{c.90.k} = Tc(Material, f_{c.90.k})\ MPa$

$f_{t.0.k} = Tc(Material, f_{t.0.k})\ MPa$

$E_{0.mean} = Tc(Material, E_{0.mean})\ MPa$

$E_{0.05} = Tc(Material, E_{0.05})\ MPa$

$G_{0.mean} = Tc(Material, G_{mean})\ MPa$

$k_{sys} = 1.0$

Height modification $\quad k_h = \max\left[1, \min\left[\left(\dfrac{Tc(Material, k_{h.d})}{h \div mm}\right)^{Tc(Material, k_{h.s})}, Tc(Material, k_{h.max})\right]\right] = 1.15$

Design material properties $\quad f_{m.d} = \dfrac{f_{m.k} \cdot k_{mod} \cdot k_h \cdot k_{sys}}{\gamma_M}$

$$f_{t.0.d} = \frac{f_{t.0.k} \cdot k_{mod} \cdot k_h \cdot k_{sys}}{\gamma_M}$$

$$f_{c.0.d} = \frac{f_{c.0.k} \cdot k_{mod} \cdot k_{sys}}{\gamma_M}$$

$$f_{c.90.d} = \frac{f_{c.90.k} \cdot k_{mod} \cdot k_{sys}}{\gamma_M}$$

$$L_{shear.max} = f_{v.d} \cdot h \cdot b \cdot \left(\frac{2}{3}\right) \times 2 = 36 \cdot kN$$

$$I_y = \frac{h^3 \cdot b}{12} = 325.57 \cdot cm^4$$

$$W_{el.y} = \frac{I_y}{(h \div 2)}$$

$$M_{max} = f_{m.d} \cdot W_{el.y} = 1.45 \cdot kN \cdot m$$

$$L_{bend.max} = \frac{M_{max}}{(span \div 2)} \times 2 = 14.5 \cdot kN$$

Bearing value can now allow paralell grain factors. [EC5 6.1.5(1)]

$$A_{ef} = b \cdot \left(b_j + 30 \cdot mm \times 2\right) \qquad A_{ef} = 9345 \cdot mm^2$$

$$L_{bear.max} = f_{c.90.d} \cdot A_{ef} = 14.4 \cdot kN$$

$$A_s = h_s \cdot b_s$$

Compression strength of a stud. $F_{c.s} = A_s \times f_{c.0.d} = 43.71 \cdot kN$

Buckling strength of a stud about the y-y axis, $F_{b.y.s} = A_s \cdot f_{c.0.d} \cdot calc\_k_c\left(h_s, I_s, f_{c.0.k}, E_{0.05}\right) = 12.52 \cdot kN$

Buckling strength of a stud about the z-z axis, $F_{b.z.s} = A_s \cdot f_{c.0.d} \cdot calc\_k_c\left(b_s, Cbs, f_{c.0.k}, E_{0.05}\right) = 32.44 \cdot kN$

$$L_{max} = \min\left(L_{bend.max}, L_{shear.max}, L_{bear.max}, F_{b.y.s}\right) = 12.5 \cdot kN$$

Instantanious deflection of the header @ L_{max}

$$\frac{\left(L_{max} \cdot span^3\right)}{48 \cdot E_{0.mean} \cdot I_y} \times \left[1 + 1.2 \cdot \frac{E_{0.mean}}{G_{0.mean}} \cdot \left(\frac{h}{span}\right)^2\right] = 0.79 \cdot mm$$

This is O.K. for all the joists in this area.

Ground Floor Door Lintels

There are 7 doors in walls taking loads (see diagrams A-E on drawing, 3 off door B).

Door A

The lintel for this door is made from Kerto-S LVL. Here are the design results from FINNWOOD 2.1

Type of structure:	Lintel
Material:	KERTO-S
Profile:	2x90x195 (B=180 mm, H=195 mm)
Service class:	1

Span lengths:

Span:	Horizontal [mm]:
Span 1	658.0
Span 2	1336.0
Span 3	658.0
Total:	2652.0

Support:	Position x [mm]:		Width [mm]: Type:
1:	0	38	Pinned support (X,Y)
2:	658	76	Pinned support (Y)
3:	1994	76	Pinned support (Y)
4:	2652	38	Pinned support (Y)

$f_{m,k}$ (Mz):	46.33 N/mm2
$f_{m,k}$ (My):	50.00 N/mm2
$f_{c,0,k}$:	35.00 N/mm2
$f_{c,90,k}$:	6.00 N/mm2
$f_{t,0,k}$:	35.26 N/mm2
$f_{v,k}$ (Vy):	4.10 N/mm2
$f_{v,k}$ (Vz):	2.30 N/mm2
$E_{,mean}$:	13800 N/mm2
$G_{,mean}$:	600 N/mm2
$E_{0.05}$:	11600 N/mm2

Safety factor:	1.20	
Load duration:	k_{mod}:	k_{def}:
Medium-term:	0.800	0.600

LOADING INFORMATION:

Dead load (Dead load, Permanent, ULS-movability = 25.9 %):

Point load: 1:	FY = 1.70 kN	x = 240.0 mm
Point load: 2:	FY = 2.10 kN	x = 640.0 mm
Point load: 3:	FY = 2.10 kN	x = 1040.0 mm
Point load: 4:	FY = 2.10 kN	x = 1440.0 mm
Point load: 5:	FY = 1.90 kN	x = 1840.0 mm
Point load: 6:	FY = 1.95 kN	x = 2240.0 mm
Point load: 7:	FY = 1.95 kN	x = 2640.0 mm

Live load (Imposed load A-residential areas, Medium-term, ULS/SLS-movability = 100.0 %):

Point load: 1:	FY = 2.20 kN	x = 240.0 mm
Point load: 2:	FY = 2.60 kN	x = 640.0 mm
Point load: 3:	FY = 2.60 kN	x = 1040.0 mm
Point load: 4:	FY = 2.60 kN	x = 1440.0 mm
Point load: 5:	FY = 2.90 kN	x = 1840.0 mm
Point load: 6:	FY = 3.15 kN	x = 2240.0 mm
Point load: 7:	FY = 3.15 kN	x = 2640.0 mm

DESIGN RESULTS:

Norm/Standard: EN 1995-1-1

Maximum utility rate: 36.3 %

DESIGN PARAMETERS:

Allowed Utot,fin: L/250 (characteristic)

Allowed Utot,inst: L/350 and 12.00 mm (characteristic)

Factor for left cantilever: 2.00

Factor for right cantilever: 2.00

Buckling is prevented on both directions (y and z)

Lateral torsional buckling (Lk1 is used when Mz>0 and Lk2 when Mz<0):

Distance between supports above of the beam: Lk1 = 300.00 mm

Distance between supports below of the beam: Lk2 = 300.00 mm

GOVERNING DESIGN RESULTS:

Check:	Actual:	Allowable:	% allowable:	Location x:	
Shear (y):	12.23 kN	63.96 kN	19.1 %	1923 mm	Comb. 3/6, Medium-term
Bending (Mz):	2.19 kNm	35.24 kNm	6.2 %	1440 mm	Comb. 3/3, Medium-term
(without kcrit):	2.19 kNm	35.24 kNm	6.2 %	1440 mm	Comb. 3/3, Medium-term
Span 1, Utot,fin:	0.05 mm	2.63 mm	2.1 %	240 mm	Comb. 6/2 (characteristic)
Span 1, Utot,inst:	0.04 mm	1.88 mm	2.2 %	240 mm	Comb. 6/2 (characteristic)
Span 2, Utot,fin:	0.40 mm	5.34 mm	7.4 %	1392 mm	Comb. 6/3 (characteristic)
Span 2, Utot,inst:	0.29 mm	3.82 mm	7.7 %	1392 mm	Comb. 6/3 (characteristic)
Span 3, Utot,fin:	0.07 mm	2.63 mm	2.6 %	2240 mm	Comb. 6/2 (characteristic)
Span 3, Utot,inst:	0.05 mm	1.88 mm	2.8 %	2240 mm	Comb. 6/2 (characteristic)

EXTREME FORCES:

Result:	Maximum val:	Location x:
Vy,max	12.23 kN	1923 mm
Mz,max	2.19 kNm	1440 mm

SUPPORT REACTIONS:

Support:	ULSmax:	ULSmin:	SLSmax:	SLSmin:	Rd/A:	Bearing:
Rs_1:	2.79 kN	-1.32 kN	1.72 kN	-0.60 kN	0.41 N/mm2	10.2 %
Rs_2:	19.68 kN	5.81 kN	13.70 kN	5.90 kN	1.44 N/mm2	36.0 %
Rs_3:	19.89 kN	5.50 kN	13.81 kN	5.57 kN	1.45 N/mm2	36.3 %
Rs_4:	8.87 kN	0.01 kN	5.91 kN	0.78 kN	1.30 N/mm2	32.4 %

- Upplift occurs, make sure of the anchoring

Support stud

The support studs are made from two rows (one for each piece of the lintel) of 38 x 89mm CLS C24 with a length of 2.544m.
@ supports 2 & 3 they are doubled. Base plate is doubled pieces laid flat.

$h_s = 89 \cdot mm$

$b_s = 38 \cdot mm$

$l_s = 2.544 \cdot m$

Material = "Softwood C24"

Material safety factor $\gamma_M = \text{get\_k}(\text{Material}, \text{Class}, \gamma_M) = 1.3$

Duration safty factor $k_{mod} = \text{get\_k}(\text{Material}, \text{Class}, k_{mod} + \text{LoadDuration}) = 0.8$

Final deformation factor $k_{def} = \text{get\_k}(\text{Material}, \text{Class}, k_{def}) = 0.6$

Characteristic material properties $f_{m.k} = \text{Tc}(\text{Material}, f_{m.0.k})\ MPa$

$f_{c.0.k} = \text{Tc}(\text{Material}, f_{c.0.k})\ MPa$

$f_{c.90.k} = \text{Tc}(\text{Material}, f_{c.90.k})\ MPa$

$f_{t.0.k} = \text{Tc}(\text{Material}, f_{t.0.k})\ MPa$

$E_{0.mean} = \text{Tc}(\text{Material}, E_{0.mean})\ MPa$

$E_{0.05} = \text{Tc}(\text{Material}, E_{0.05})\ MPa$

$G_{0.mean} = \text{Tc}(\text{Material}, G_{mean})\ MPa$

$k_{sys} = 1.0$

Height modification $$k_h = \max\left[1, \min\left[\left(\frac{\text{Tc}(\text{Material}, k_{h.d})}{h \div mm}\right)^{\text{Tc}(\text{Material}, k_{h.s})}, \text{Tc}(\text{Material}, k_{h.max})\right]\right] = 1.15$$

Design material properties $$f_{m.d} = \frac{f_{m.k} \cdot k_{mod} \cdot k_h \cdot k_{sys}}{\gamma_M}$$

$$f_{t.0.d} = \frac{f_{t.0.k} \cdot k_{mod} \cdot k_h \cdot k_{sys}}{\gamma_M}$$

$$f_{c.0.d} = \frac{f_{c.0.k} \cdot k_{mod} \cdot k_{sys}}{\gamma_M}$$

$$f_{c.90.d} = \frac{f_{c.90.k} \cdot k_{mod} \cdot k_{sys}}{\gamma_M}$$

Test the strength of the studs.

$A_s = b_s \cdot h_s = 3.38 \times 10^3 \cdot mm^2$

Compression strength of a stud. $F_{c.s} = A_s \times f_{c.0.d} = 43.71 \cdot kN$

Buckling strength of a stud about the y-y axis, $F_{b.y.s} = A_s \cdot f_{c.0.d} \cdot \text{calc\_k}_c\left(h_s, l_s, f_{c.0.k}, E_{0.05}\right) = 13.58 \cdot kN$

Buckling strength of a stud about the z-z axis, $F_{b.z.s} = A_s \cdot f_{c.0.d} \cdot \text{calc\_k}_c\left(b_s, \text{Cbs}, f_{c.0.k}, E_{0.05}\right) = 32.44 \cdot kN$

Maximum compressive load in a stud $F_{c.89C24} = \min\left(F_{c.s}, F_{b.y.s}, F_{b.z.s}\right) = 13.58 \cdot kN$

Each support position only need to carry half of the support reaction (max 10kN) so the strength of the studs are O.K.

Base plate compresive strength

Maximum compression load will be from the cental supports.

$h = 89 \cdot mm$

$b = 38 \cdot mm$

Bearing value allowing for paralell grain & continuous support factors. [EC5 6.1.5(1-3)]

$k_{c.90} = 1.25$

$f_{c.90.d} \cdot k_{c.90} \cdot [h \cdot (b + 30 \cdot mm \times 2)] = 16.8 \cdot kN$ for a single stud

$f_{c.90.d} \cdot k_{c.90} \cdot [h \cdot (2 \cdot b + 30 \cdot mm \times 2)] = 23.3 \cdot kN$ for a double stud

Compression on the base plates is O.K.

Door B

This door also has a Kert-S lintel but it is smaller. Results from FINNWOOD 2.1

Type of structure: Beam

Material:	KERTO-S
Profile:	2x45x195 (B=90 mm, H=195 mm)
Service class:	1

Span:	Horizontal [mm]:
Span 1	1870.0
Total:	1870.0

Support:	Position x [mm]:	Width [mm]:	Type:
1:	0	47	Pinned support (X,Y)
2:	1870	47	Pinned support (Y)

$f_{m,k}$ (Mz):	46.33 N/mm2
$f_{m,k}$ (My):	50.00 N/mm2
$f_{c,0,k}$:	35.00 N/mm2
$f_{c,90,k}$:	6.00 N/mm2
$f_{t,0,k}$:	35.26 N/mm2
$f_{v,k}$ (Vy):	4.10 N/mm2
$f_{v,k}$ (Vz):	2.30 N/mm2
$E_{,mean}$:	13800 N/mm2
$G_{,mean}$:	600 N/mm2
$E_{0.05}$:	11600 N/mm2

Safety factor: 1.20

Load duration class:		kmod:	kdef:
Medium-term:	0.800	0.600	

LOADING INFORMATION:

Dead load (Dead load, Permanent, ULS-movability = 25.9 %):

Point load: 1:	FY = 1.25 kN	x = 0.0 mm
Point load: 2:	FY = 1.25 kN	x = 400.0 mm

Point load: 3:	FY = 1.25 kN	x = 800.0 mm
Point load: 4:	FY = 1.25 kN	x = 1200.0 mm
Point load: 5:	FY = 2.50 kN	x = 1600.0 mm

Live load (Imposed load A-residential areas, Medium-term, ULS/SLS-movability = 100.0 %):

Point load: 1:	FY = 2.30 kN	x = 0.0 mm
Point load: 2:	FY = 2.30 kN	x = 400.0 mm
Point load: 3:	FY = 2.30 kN	x = 800.0 mm
Point load: 4:	FY = 2.30 kN	x = 1200.0 mm
Point load: 5:	FY = 2.30 kN	x = 1600.0 mm

DESIGN RESULTS:

Norm/Standard: EN 1995-1-1

Maximum utility rate: 52.2 %

DESIGN PARAMETERS:

Allowed Utot,fin: L/250 (characteristic)

Allowed Utot,inst: L/350 and 12.00 mm (characteristic)

GOVERNING DESIGN RESULTS:

Check:	Actual:	Allowable:	% allowable:	Location x:	
Shear (y):	12.43 kN	31.98 kN	38.9 %	1636 mm	Comb. 3/1, Medium-term
Bending (Mz):	5.79 kNm	17.62 kNm	32.9 %	800 mm	Comb. 3/1, Medium-term
(without kcrit):	5.79 kNm	17.62 kNm	32.9 %	800 mm	Comb. 3/1, Medium-term
Span 1, Utot,fin:	3.17 mm	7.48 mm	42.4 %	935 mm	Comb. 6/1 (characteristic)
Span 1, Utot,inst:		2.36 mm	5.34 mm	44.2 %	935 mm Comb. 6/1 (characteristic)

GOVERNING DESIGN RESULT COMBINATIONS:

Combination 3/1 (Medium-term): 1.35*Dead load + 1.50*Live load

Combination 6/1 (characteristic: 1.00*Dead load + 1.00*Live load

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EXTREME FORCES:

Result:	Maximum value:	Location x:
Vy,max	12.43 kN	1636 mm
Mz,max	5.79 kNm	800 mm

SUPPORT REACTIONS:

Support:	ULSmax:	ULSmin:	SLSmax:	SLSmin:	Rd/A:	Bearing:
1:	14.94 kN	3.76 kN	10.34 kN	3.76 kN	3.53 N/mm2	52.2 %
2:	12.43 kN	3.74 kN	8.66 kN	3.74 kN	2.94 N/mm2	43.4 %

Support stud

The support studs are a single piece at each end, resting on standard wall base plates.

$h_S = 144 \cdot mm$

$b_S = 47 \cdot mm$

$l_S = 2.544 \cdot m$

Material = "Softwood C16"

Material safety factor $\gamma_M = \text{get\_k}(\text{Material}, \text{Class}, \gamma_M) = 1.3$

Duration safty factor $k_{mod} = \text{get\_k}(\text{Material}, \text{Class}, k_{mod} + \text{LoadDuration}) = 0.8$

Final deformation factor $k_{def} = \text{get\_k}(\text{Material}, \text{Class}, k_{def}) = 0.6$

Characteristic material properties

$$f_{m.k} = \text{Tc}(\text{Material}, f_{m.0.k})\ \text{MPa}$$

$$f_{c.0.k} = \text{Tc}(\text{Material}, f_{c.0.k})\ \text{MPa}$$

$$f_{c.90.k} = \text{Tc}(\text{Material}, f_{c.90.k})\ \text{MPa}$$

$$f_{t.0.k} = \text{Tc}(\text{Material}, f_{t.0.k})\ \text{MPa}$$

$$E_{0.mean} = \text{Tc}(\text{Material}, E_{0.mean})\ \text{MPa}$$

$$E_{0.05} = \text{Tc}(\text{Material}, E_{0.05})\ \text{MPa}$$

$$G_{0.mean} = \text{Tc}(\text{Material}, G_{mean})\ \text{MPa}$$

$$k_{sys} = 1.0$$

Height modification

$$k_h = \max\left[1, \min\left[\left(\frac{\text{Tc}(\text{Material}, k_{h.d})}{h \div mm}\right)^{\text{Tc}(\text{Material}, k_{h.s})}, \text{Tc}(\text{Material}, k_{h.max})\right]\right] = 1.11$$

Design material properties

$$f_{m.d} = \frac{f_{m.k} \cdot k_{mod} \cdot k_h \cdot k_{sys}}{\gamma_M}$$

$$f_{t.0.d} = \frac{f_{t.0.k} \cdot k_{mod} \cdot k_h \cdot k_{sys}}{\gamma_M}$$

$$f_{c.0.d} = \frac{f_{c.0.k} \cdot k_{mod} \cdot k_{sys}}{\gamma_M}$$

$$f_{c.90.d} = \frac{f_{c.90.k} \cdot k_{mod} \cdot k_{sys}}{\gamma_M}$$

Test the strength of the studs.

$$A_s = b_s \cdot h_s = 6.77 \times 10^3 \cdot mm^2$$

Compression strength of a stud. Lintels have only 90mm width in contact with the stud.

$$F_{c.s} = 90mm \cdot b_s \times f_{c.0.d} = 44.25 \cdot kN$$

Buckling strength of a stud about the y-y axis, $F_{b.y.s} = A_s \cdot f_{c.0.d} \cdot \text{calc\_k}_c(h_s, l_s, f_{c.0.k}, E_{0.05}) = 43.91 \cdot kN$

Buckling strength of a stud about the z-z axis, $F_{b.z.s} = A_s \cdot f_{c.0.d} \cdot \text{calc\_k}_c(b_s, \text{Cbs}, f_{c.0.k}, E_{0.05}) = 58.84 \cdot kN$

Each support position has a max load of 15kN so the strength of the studs are O.K.

Base plate compresive strength

Bearing value allowing for paralell grain & continious support factors. [EC5 6.1.5(1-3)]

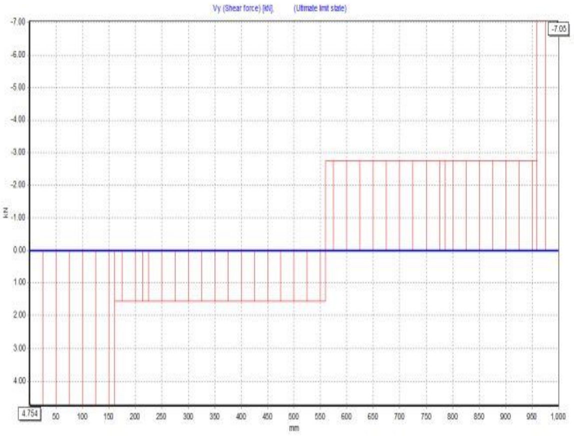
$$k_{c.90} = 1.25$$

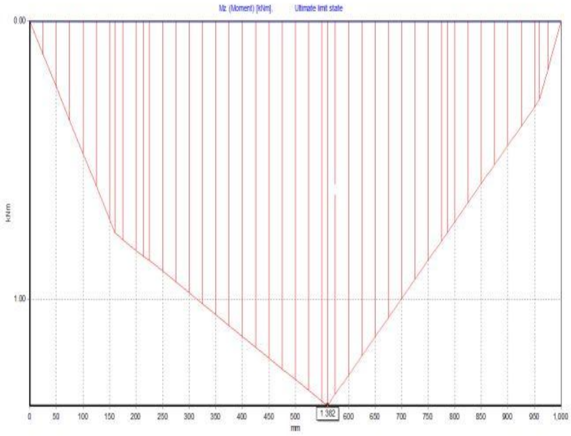
$$f_{c.90.d} \cdot k_{c.90} \cdot \left[126mm \cdot \left(b_s + 30 \cdot mm \times 2\right)\right] = 22.8 \cdot kN$$

Compression on the base plates is O.K.

Door C

Door C is much simpler and only spans 1m, with lower loads imposed.
Shear force and Bending diagrams for this span. (no allowance for header being continuous)





This gives us values of :- $V = 7.05 \cdot kN$ $M = 1.382 \cdot kN \cdot m$ @0.56m $Sr_1 = 4.75 \cdot kN$ $Sr_2 = 7.05 \cdot kN$

All if these values can be halved as it is a double wall in this loacation.

$V \div 2 = 3.53 \cdot kN$ $M \div 2 = 0.69 \cdot kN \cdot m$ $Sr_1 \div 2 = 2.38 \cdot kN$ $Sr_2 \div 2 = 3.53 \cdot kN$

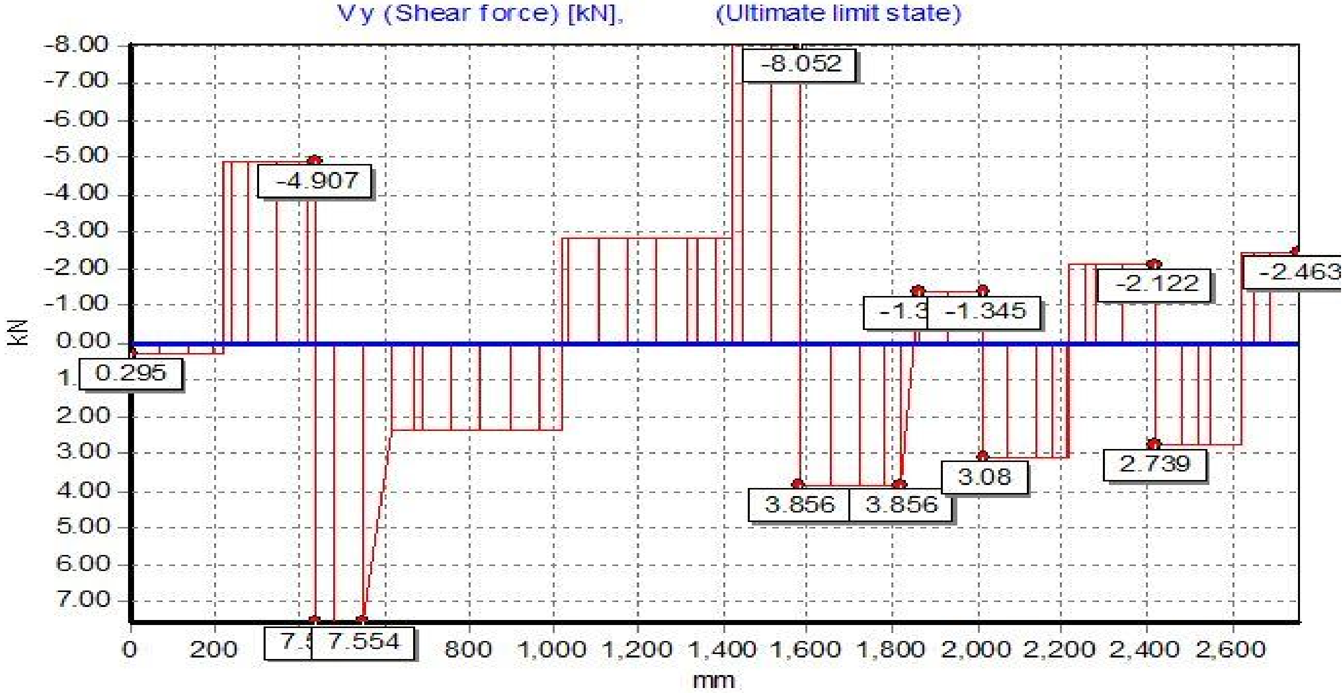
Values for maxium stress in standard wall header from earlier :-

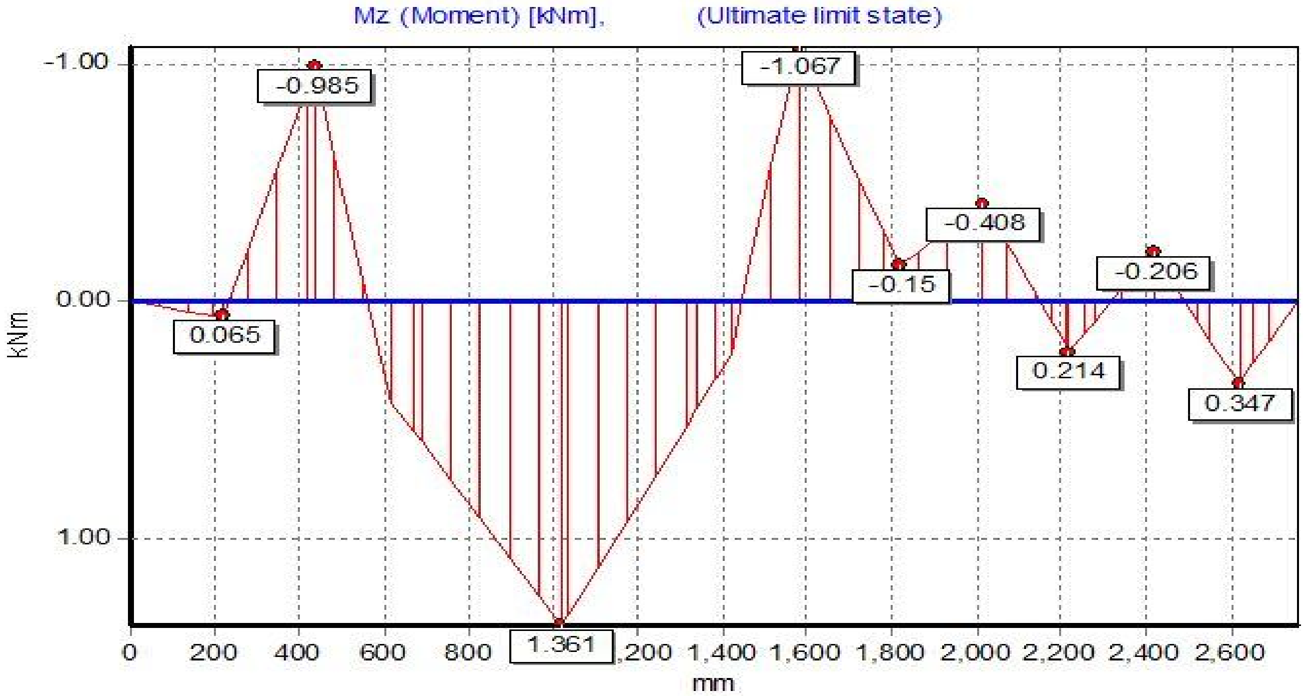
$V_{max} = 12.75 \cdot kN$ $M_{max} = 1.45 \cdot kN \cdot m$ $F_{c.63C16} = 3.4 \cdot kN$

These values are very tight but acceptable as the header is actually continuous with other studs nearby sharing the loads, studs will also be braced by the door framing.

Door D

This door is more complicated as it is a single thickness wall of the entertainment room and has loads simular to Door B. Modelling the first 2.7m of the wall gives these Bending and Shear force diagrams :-





This gives us values of :- $V = 8.05 \cdot kN$ $M = 1.361 \cdot kN \cdot m$ @0.56m $Sr_1 = 12.46 \cdot kN$ $Sr_2 = 11.91 \cdot kN$

This lintel is in the entertainment room wall so $b = 89mm$

$Material = "Softwood C24"$

Material safety factor $\gamma_M = get\_k(Material, Class, \gamma_M) = 1.3$

Duration safty factor $k_{mod} = get\_k(Material, Class, k_{mod} + LoadDuration) = 0.8$

Final deformation factor $k_{def} = get\_k(Material, Class, k_{def}) = 0.6$

Characteristic material properties

$f_{m.k} = Tc(Material, f_{m.0.k})\ MPa$

$f_{c.0.k} = Tc(Material, f_{c.0.k})\ MPa$

$f_{c.90.k} = Tc(Material, f_{c.90.k})\ MPa$

$k_{sys} = 1.0$

Height modification

$$k_h = max\left[1, min\left[\left(\frac{Tc(Material, k_{h.d})}{h \div mm}\right)^{Tc(Material, k_{h.s})}, Tc(Material, k_{h.max})\right]\right] = 1.11$$

Design material properties

$$f_{m.d} = \frac{f_{m.k} \cdot k_{mod} \cdot k_h \cdot k_{sys}}{\gamma_M}$$

$$f_{c.0.d} = \frac{f_{c.0.k} \cdot k_{mod} \cdot k_{sys}}{\gamma_M}$$

$$f_{c.90.d} = \frac{f_{c.90.k} \cdot k_{mod} \cdot k_{sys}}{\gamma_M}$$

minimum depth for bending

$$h_m = \sqrt{\frac{M \times 6}{f_{m.d} \times b}} = 74.8102 \cdot mm$$

minimum depth for shear

$$h_v = \frac{3 \cdot V}{2 \cdot b \cdot f_{v.d}} = 33.97 \cdot mm$$

minimum number of 38mm thick pieces will be

$$n = ceil\left(\frac{max(h_m, h_v)}{38mm}\right) = 2 \qquad h = n \cdot 38mm = 76 \cdot mm$$

maximum compressive load in a 89mm stud $F_{c.89C24} = 13.58 \cdot kN$

Bearing value allowing for paralell grain factors. [EC5 6.1.5(1-3)]

$k_{c.90} = 1.0$

$f_{c.90.d} \cdot k_{c.90} \cdot [89mm \cdot (38mm + 30 \cdot mm \times 2)] = 13.4 \cdot kN$

So one extra header piece (extended by >30mm each side) of single studs is O.K.

Door E

This door has very small loads and is 0.8m wide and is O.K. without any additional lintel.

Cold water tank

The cold water tank is situated over the door to Bedroom 2. The tank which is 1.5 x 0.4 x 1.5m (volume 900l) sits on the first floor deck and is supported on a framework below this to the top of the wall and to the ground floor. All members are 38 x 63mm C16 CLS.

$b = 38mm$

$h = 63mm$

$LoadDuration = \textbf{\textit{Permanent}}$

$Material = \text{"Softwood C16"}$

Material safety factor $\gamma_M = get\_k(Material, Class, \gamma_M) = 1.3$

Duration safty factor $k_{mod} = get\_k(Material, Class, k_{mod} + LoadDuration) = 0.6$

Final deformation factor $k_{def} = get\_k(Material, Class, k_{def}) = 0.6$

Characteristic material properties $f_{m.k} = Tc(Material, f_{m.0.k})\ MPa$

$f_{c.0.k} = Tc(Material, f_{c.0.k})\ MPa$

$f_{c.90.k} = Tc(Material, f_{c.90.k})\ MPa$

$f_{t.0.k} = Tc(Material, f_{t.0.k})\ MPa$

$E_{0.mean} = Tc(Material, E_{0.mean})\ MPa$

$E_{0.05} = Tc(Material, E_{0.05})\ MPa$

$G_{0.mean} = Tc(Material, G_{mean})\ MPa$

$k_{sys} = 1.1$ All parts made of 4 load sharing elements

Height modification $$k_h = max\left[1, min\left[\left(\frac{Tc(Material, k_{h.d})}{h \div mm}\right)^{Tc(Material, k_{h.s})}, Tc(Material, k_{h.max})\right]\right] = 1.19$$

Design material properties $$f_{m.d} = \frac{f_{m.k} \cdot k_{mod} \cdot k_h \cdot k_{sys}}{\gamma_M}$$

$$f_{t.0.d} = \frac{f_{t.0.k} \cdot k_{mod} \cdot k_h \cdot k_{sys}}{\gamma_M}$$

$$f_{c.0.d} = \frac{f_{c.0.k} \cdot k_{mod} \cdot k_{sys}}{\gamma_M}$$

$$f_{c.90.d} = \frac{f_{c.90.k} \cdot k_{mod} \cdot k_{sys}}{\gamma_M} = 1.12 \cdot MPa$$

$$I_y = \frac{b \cdot h^3}{12}$$

$$I_z = \frac{b^3 \cdot h}{12}$$

$$A_s = b \cdot h = 2.39 \times 10^{-3} m^2$$

First layer is 4 members laid flat at 375mm spacings each taking equal loads.

Design tank load $q_t = 900 \cdot l \cdot \gamma_{water} \cdot \gamma_G = 11.92 \cdot kN$

Member load $q_m = \frac{q_t}{4} = 2.98 \cdot kN$

These then load into four lintels 1.85m long to span over the door way.

So point loads on these lintels are $q_{pl} = \frac{q_m}{4} = 0.74 \cdot kN$

Bearing pressure $\sigma_{c.pl} = \frac{q_{pl}}{38mm \cdot 63mm} = 0.31 \cdot N \cdot mm^{-2}$ $OKifLT\left(\sigma_{c.pl}, f_{c.90.d}\right) = \text{"O.K."}$

Using Finnwood to perform the analysis of the lintels gives

Finnwood 2.1

Water tank lintel.s01
Cantilever/span lengths:

Span 1	1000.0
Span 2	550.0
Total:	1550.0

Support:	Position x [mm]:	Width [mm]:	Type:
1:	0	63	Pinned support (X,Y)
2:	1000	63	Pinned support (Y)
3:	1550	63	Pinned support (Y)

LOADING INFORMATION:
Dead load (Dead load, Permanent, ULS-movability = 25.9 %):

Point load: 1:	FY = 0.55 kN	x = 182.0 mm
Point load: 2:	FY = 0.55 kN	x = 557.0 mm
Point load: 3:	FY = 0.55 kN	x = 932.0 mm
Point load: 4:	FY = 0.55 kN	x = 1307.0 mm

GOVERNING DESIGN RESULTS:

Check:	Actual:	Location x:
Shear (y):	1.37 kN	969 mm
Bending (Mz):	0.20 kNm	557 mm

SUPPORT REACTIONS:

Support:	ULSmax:	ULSmin:	SLSmax:	SLSmin:	Rd/A:
1:	0.86 kN	0.63 kN	0.64 kN	0.64 kN	0.35 N/mm2
2:	1.93 kN	1.43 kN	1.43 kN	1.43 kN	0.79 N/mm2
3:	0.23 kN	0.08 kN	0.13 kN	0.13 kN	0.10 N/mm2

Design bending moment $M_{y.d} = 0.2kN \cdot m$

$$W_{el.y} = \frac{I_y}{(h \div 2)}$$

bending stress

$$\sigma_{m.d} = \frac{M_{y.d}}{W_{el.y}} = 7.96 \cdot MPa$$

$$OKifLT\left(\sigma_{m.d}, f_{m.d}\right) = \text{"O.K."}$$

Design shear force

$$V_d = 1.37kN$$

shear stress

$$\tau_{v.d} = \frac{3}{2}\frac{V_d}{A_s} = 0.86 \cdot MPa$$

$$OKifLT\left(\tau_{v.d}, f_{v.d}\right) = \text{"O.K."}$$

2 Lintels are supported by post made from a piece joined edgewise to the flat side of another member the resulting second moment of areas are

Design compresive load

$$F_d = 2 \cdot 1.93kN = 3.86 \cdot kN$$

$l_y = 2.93m$ $l_z = 0.6m$ Counter batten spacing

$$I_{yy} = I_y + A_s \cdot (25.25mm)^2 + I_z + A_s \cdot (25.25mm)^2 = 413.3 \cdot cm^4$$

$$I_{zz} = I_y + I_z = 107.99 \cdot cm^4$$

Radius of gyration

$$i_y = \sqrt{\frac{I_{yy}}{2A_s}} = 0.03\,m \qquad i_z = \sqrt{\frac{I_{zz}}{2A_s}} = 0.02\,m$$

Slenderness ratio

$$\lambda_y = \frac{l_y}{i_y} = 99.73 \qquad \lambda_z = \frac{l_z}{i_z} = 39.95$$

Relative slenderness

$$\lambda_{rel.y} = \frac{\lambda_y}{\pi} \cdot \sqrt{\frac{f_{c.0.k}}{E_{0.05}}} = 1.78$$

$$\lambda_{rel.z} = \frac{\lambda_z}{\pi} \cdot \sqrt{\frac{f_{c.0.k}}{E_{0.05}}} = 0.71$$

$$\lambda_{rel} = \max\left(\lambda_{rel.y}, \lambda_{rel.z}\right)$$

$$k = 0.5 \cdot \left[1 + 0.2 \cdot \left(\lambda_{rel} - 0.3\right) + \lambda_{rel}^2\right] = 2.23$$

$$k_c = \frac{1}{k + \sqrt{k^2 - \lambda_{rel}^2}} = 0.28$$

$$\sigma_{c.d} = \frac{F_d}{2A_s \cdot k_c} = 2.89 \cdot MPa$$

$$OKifLT\left(\sigma_{c.d}, f_{c.0.d}\right) = \text{"O.K."}$$

Entertainment Room

The entertainment room is a soundproofed concrete box inside and isolated from the main structure, constructed from dense concrete blocks for the walls and a cast concrete roof/ceiling.

Roof/Ceiling design

The roof is a two way reinforced concrete slab with unrestrained simply supported edges.

Slab dimensions

Shorter span of the slab	$l_x = 3.97m$	
Longer span of the slab	$l_y = 4.9m$	
Depth of the slab	$h = 150mm$	

Loading

Characteristic permanent action	$G_k = h \cdot \gamma_{conc} = 3.68 \cdot kN \cdot m^{-2}$	Only load is self weight
Characteristic variable action	$Q_k = 0$	
Design Ultimate load	$q = \gamma_G \cdot G_k = 4.96 \cdot kN \cdot m^{-2}$	
Quasi permanent load	$q_{SLS} = G_k$	

Reinforcement

Reinforcement provided	A193 Mesh	
Wire size	$\phi_s = 7mm$	
Wire spacing	$s_{x.p} = 200mm$	
	$s_{y.p} = 200mm$	
Area of reinforcement	$A_{sx.p} = 193mm^2 \div m$	
	$A_{sy.p} = 193mm^2 \div m$	
Characteristic yield strength	$f_{y.k} = 500MPa$	
Partial factor (Table 2.1N)	$\gamma_s = 1.15$	
Design yield strength (fig. 3.8)	$f_{y.d} = f_{y.k} \div \gamma_s = 434.78 \cdot MPa$	

Concrete properties

Concrete strength class	C25/30	
Characteristic cylinder strength	$f_{c.k} = 25MPa$	
Compressive strength factor (cl. 3.1.6)	$\alpha_{cc} = 0.85$	
Design compressive strength (cl. 3.1.6)	$f_{c.d} = \frac{f_{c.k} \cdot \alpha_{cc}}{\gamma_C} = 14.17 \cdot MPa$	
Mean axial tensile strength (Table 3.1)	$f_{c.t.m} = 0.3MPa \cdot \left(\frac{f_{c.k}}{1MPa}\right)^{\frac{2}{3}} = 2.56 \cdot MPa$	
Maximum aggregate size	$d_g = 30mm$	

Concrete cover to reinforcement

Nominal cover to outer bottom reinforcement $c_{nom} = 20mm$

Fire resistance period to bottom of slab $R_{btm} = 30min$

Axia distance to bottom reinft (Table 5.8) $a_{fi} = 10mm$

Min. btm cover requirement with regard to bond $c_{min.b} = \phi_s = 7 \cdot mm$

Reinforcement fabrication No QA system

Cover allowance for deviation $\Delta C_{dev} = 10mm$

Min. required nominal cover to bottom reinft $c_{nom.min} = \Delta C_{dev} + c_{min.b} = 17 \cdot mm$

Check if sufficient cover $OKifLT\left(c_{nom.min}, c_{nom}\right) = \text{"O.K."}$

Reinforcement design at midspan in short span direction (cl. 6.1)

Bending moment coefficient $\alpha_{sx.p} = 0.0871$

Design bending moment $M_{x.p} = \alpha_{sx.p} \cdot q \cdot l_x^2 = 6.82 \cdot kN$

Effective depth to tension reinforcement $d_{x.p} = h - c_{nom} - 1.5\phi_s = 119.5 \cdot mm$

K factor $K = \dfrac{M_{x.p}}{d_{x.p}^2 \cdot f_{c.k}} = 0.019$

Redistribution ratio $\delta = 1.0$

K' factor $K' = 0.598\delta - 0.18\delta^2 - 0.21 = 0.208$

K < K' - Compression reinforcement is not required $A_{scx.p.req} = 0mm^2 m^{-1}$

Lever arm $z = \min\left[0.95 \cdot d_{x.p}, \dfrac{d_{x.p}}{2}\left(1 + \sqrt{1 - 3.53K}\right)\right] = 113.52 \cdot mm$

Area of reinforcement required for bending $A_{sx.p.m} = \dfrac{M_{x.p}}{f_{y.d} \cdot z} = 138.08 \cdot mm^2 m^{-1}$

Minimum area of reinforcement required $A_{sx.p.min} = \max\left(0.26 \dfrac{f_{c.t.m}}{f_{y.k}} d_{x.p}, 0.0013d_{x.p}\right) = 159.39 \cdot mm^2 m^{-1}$

Area of reinforcement required $A_{sx.p.req} = \max\left(A_{sx.p.m}, A_{sx.p.min}\right) = 159.39 \cdot mm^2 m^{-1}$

Check against provided area $OKifLT\left(A_{sx.p.req}, A_{sx.p}\right) = \text{"O.K."}$

Check reinforcement spacing

Reinforcement service stress $\sigma_{sx.p} = f_{y.d} \cdot \min\left(\dfrac{A_{sx.p.m}}{A_{sx.p}}, 1.0\right) \dfrac{q_{SLS}}{q} = 230 \cdot MPa$

Maximum allowable spacing (Table 7.3N w_k=0.3mm) $s_{max.x.p} = 1.25\left(400 - \dfrac{\sigma_{sx.p}}{MPa}\right) mm = 212 \cdot mm$

Check against spacing provided $OKifLT\left(s_{x.p}, s_{max.x.p}\right) = \text{"O.K."}$

Reinforcement design at midspan in long span direction (cl. 6.1)

Bending moment coefficient $\alpha_{sy.p} = 0.0576$

Design bending moment $M_{y.p} = \alpha_{sy.p} \cdot q \cdot l_y^{2} = 6.87 \cdot kN$

Effective depth to tension reinforcement $d_{y.p} = h - c_{nom} - 0.5\phi_s = 126.5 \cdot mm$

$$K = \frac{M_{y.p}}{d_{y.p}^{2} \cdot f_{c.k}} = 0.017$$

Redistribution ratio $\delta = 1.0$

$$K' = 0.598\delta - 0.18\delta^{2} - 0.21 = 0.208$$

K < K' - Compression reinforcement is not required $A_{scy.p.req} = 0mm^{2}m^{-1}$

Lever arm

$$z = \min\left[0.95 \cdot d_{y.p}, \frac{d_{y.p}}{2}\left(1 + \sqrt{1 - 3.53K}\right)\right] = 120.17 \cdot mm$$

Area of reinforcement required for bending $A_{sy.p.m} = \frac{M_{y.p}}{f_{y.d} \cdot z} = 131 \cdot mm^{2}m^{-1}$

Minimum area of reinforcement required $A_{sy.p.min} = \max\left(0.26\frac{f_{c.t.m}}{f_{y.k}}d_{y.p}, 0.0013d_{y.p}\right) = 168.72 \cdot mm^{2}m^{-1}$

Area of reinforcement required $A_{sy.p.req} = \max\left(A_{sy.p.m}, A_{sy.p.min}\right) = 168.72 \cdot mm^{2}m^{-1}$

Check against provided area $\mathrm{OKifLT}\left(A_{sy.p.req}, A_{sy.p}\right) = \text{"O.K."}$

Check reinforcement spacing

Reinforcement service stress $\sigma_{sy.p} = f_{y.d} \cdot \min\left(\frac{A_{sy.p.m}}{A_{sy.p}}, 1.0\right)\frac{q_{SLS}}{q} = 219 \cdot MPa$

Maximum allowable spacing (Table 7.3N w_k=0.3mm) $s_{max.y.p} = 1.25\left(400 - \frac{\sigma_{sy.p}}{MPa}\right)mm = 226 \cdot mm$

Check against spacing provided $\mathrm{OKifLT}\left(s_{y.p}, s_{max.y.p}\right) = \text{"O.K."}$

Shear capacity check at short span discontinuous support

Shear force $V_{x.d} = q \cdot \frac{l_x}{2} = 9.85 \cdot \frac{kN}{m}$

Effective depth $d_{x.d} = d_{x.p} = 119.5 \cdot mm$

Effective depth factor $k = \min\left(2.0, 1 + \sqrt{\frac{200mm}{d_{x.d}}}\right) = 2$

Reinforcement ratio $\rho_l = \min\left(0.02, \frac{A_{sx.p}}{d_{x.d}}\right) = 0.0016$

Minimum shear resistance

$$V_{Rd.c.min} = 0.035MPa \cdot k^{1.5} \cdot \sqrt{\frac{f_{c.k}}{MPa}} \cdot d_{x.d} = 59.15 \cdot \frac{kN}{m}$$

Shear resistance

$$V_{Rd.c.x.d} = \max\left[V_{Rd.c.min}, \frac{0.18MPa}{\gamma_C} \cdot k \cdot \left(\sqrt[3]{\frac{f_{c.k}}{MPa} \cdot 100 \cdot \rho_l}\right) d_{x.d}\right] = 59.15 \cdot \frac{kN}{m}$$

Check against design shear

$$OKifLT\left(V_{x.d}, V_{Rd.c.x.d}\right) = \text{"O.K."}$$

Shear capacity check at long span discontinuous support

Shear force

$$V_{y.d} = q \cdot \frac{l_y}{2} = 12.16 \cdot \frac{kN}{m}$$

Effective depth

$$d_{y.d} = d_{y.p} = 126.5 \cdot mm$$

Effective depth factor

$$k = \min\left(2.0, 1 + \sqrt{\frac{200mm}{d_{y.d}}}\right) = 2$$

Reinforcement ratio

$$\rho_l = \min\left(0.02, \frac{A_{sy.p}}{d_{x.d}}\right) = 0.0016$$

Minimum shear resistance

$$V_{Rd.c.min} = 0.035MPa \cdot k^{1.5} \cdot \sqrt{\frac{f_{c.k}}{MPa}} \cdot d_{y.d} = 62.61 \cdot \frac{kN}{m}$$

Shear resistance

$$V_{Rd.c.y.d} = \max\left[V_{Rd.c.min}, \frac{0.18MPa}{\gamma_C} \cdot k \cdot \left(\sqrt[3]{\frac{f_{c.k}}{MPa} \cdot 100 \cdot \rho_l}\right) d_{y.d}\right] = 62.61 \cdot \frac{kN}{m}$$

Check against design shear

$$OKifLT\left(V_{y.d}, V_{Rd.c.y.d}\right) = \text{"O.K."}$$

Basic span-to-depth deflection ratio check (cl. 7.4.2)

Reference reinforcement ratio

$$\rho_0 = 10^{-3} \sqrt{\frac{f_{c.k}}{MPa}} = 0.005$$

Required tension reinforcement ratio

$$\rho = \max\left(0.0035, \frac{A_{sx.p.req}}{d_{x.p}}\right) = 0.0035$$

Stuctural system factor (Table 7.4N)

$$K_\delta = 1.0$$

Basic limit span-to-depth ratio (Exp. 7.16)

$$r_{lim.x.bas} = K_\delta \cdot \left[11 + 1.5 \sqrt{\frac{f_{c.k}}{MPa}} \cdot \frac{\rho_0}{\rho} + 3.2 \sqrt{\frac{f_{c.k}}{MPa}} \left(\frac{\rho_0}{\rho} - 1\right)^{1.5}\right] = 26.2$$

Modified limit span-to-eff. depth ratio

$$r_{lim.x} = \min\left(1.5, \frac{500MPa}{f_{c.k}} \frac{A_{sx.p}}{A_{sx.p.m}}\right) \cdot r_{lim.x.bas} = 39.31$$

Actual span-to-eff. depth ratio

$$r_{act.x} = \frac{l_x}{d_{x.p}} = 33.22$$

Check against limit

$$OKifLT\left(r_{act.x}, r_{lim.x}\right) = \text{"O.K."}$$

Wall design

The walls are constructed from single leaf of dense concrete blocks and are restrained on all four edges.

Wall length	$L = 4.9m$	Longest wall
Wall height	$h = 2.63m$	
Wall thickness	$t = 100mm$	

Partial factors for material strength

Category of manufacturing control	Category 1
Class of execution control	Class 1
Partial factor for masonry in compressive flexure	$\gamma_{Mc} = 2.3$

Effective height of masonry walls - Section 5.5.1.2

Reduction factors $p_2 = 0.75$ constrained by roof and floor

$$p_4 = \frac{1}{1+\left(\frac{p_2 h}{3L}\right)^2} p_2 = 0.74$$

Effective height of wall - eq 5.2 $h_{ef} = p_4 \cdot h = 1.94\,m$

Effective thickness of masonry walls - Section 5.5.1.3

Effective thickness $t_{ef} = t = 0.1\,m$

Masonry details

Mean compressive strength of masonry unit	$f_b = 7.3MPa$
Density of masonary	$\gamma = 18.8kN \cdot m^{-3}$
Mortar type	M4 - General purpose
Compressive strength of mortar	$f_m = 4MPa$
Compressive strength factor - Table NA.4	$K = 0.55$
Characteristic compressive strength of masonry - eq 3.2	$f_k = K \cdot f_b^{0.7} \cdot f_m^{0.3} = 3.35 \cdot MPa$
Design compressive strength	$f_d = f_k \div \gamma_{Mc} = 1.46 \cdot MPa$

Vertical loading details

Dead load on top of wall $G_k = 150mm \cdot \gamma_{conc} \cdot \frac{L}{2} = 9.01 \cdot kN \cdot m^{-1}$ Maxiumum load from roof

No live load and load is not eccentric $e_G = 0$

Slenderness ratio of masonry walls - Section 5.5.1.4

Allowable slenderness ratio $SR_{all} = 27$

Actuall slenderness ratio $SR = \frac{h_{ef}}{t_{ef}} = 19.38$

$OKifLT\left(SR, SR_{all}\right) = $ "O.K."

Check vertical loads
Reduction factor for slenderness and eccentricity - Section 6.1.2.2

Design bending moment at top or bottom of wall $M_{i.d} = \gamma_G \cdot G_k \cdot e_G = 0$

Design vertical load at top or bottom of wall $N_{i.d} = \gamma_G \cdot G_k = 12.16 \cdot kN \cdot m^{-1}$

Initial eccentricity - cl.5.5.1.1 $e_{init} = \frac{h_{ef}}{450} = 4.31 \cdot mm$

Eccentricity due to horizontal load $e_h = 0$

Eccentricity at top or bottom of wall - eq.6.5 $e_i = \max\left(\frac{M_{i.d}}{N_{i.d}} + e_h + e_{init}, 0.05t\right) = 5 \cdot mm$

Reduction factor at top or bottom of wall - eq.6.4 $\Phi_i = \max\left(1 - 2\frac{e_i}{t}, 0\right) = 0.9$

Design bending moment at middle of wall $M_{m.d} = \gamma_G \cdot G_k \cdot e_G = 0$

Design vertical load at middle of wall $N_{m.d} = \gamma_G \cdot G_k + \gamma \cdot t \cdot \frac{h}{2} = 14.64 \cdot kN \cdot m^{-1}$

Eccentricity due to horizontal load $e_{hm} = 0$

Eccentricity at middle of wall due to loads- eq.6.7 $e_m = \frac{M_{m.d}}{N_{m.d}} + e_{hm} + e_{init} = 4.31 \cdot mm$

Eccentricity at middle of wall due to creep $e_k = 0mm$

Eccentricity at middle of wall - eq.6.6 $e_{m.k} = \max\left(e_m + e_k, 0.05t\right) = 5 \cdot mm$

From eq.G.2 $A_1 = 1 - 2\frac{e_{m.k}}{t} = 0.9$

Short term secant modulus of elasticity factor $K_E = 1000$

Modulus of elasticity - cl.3.7.2 $E = K_E \cdot f_k = 3352 \cdot MPa$

Slenderness - eq.G.4 $\lambda = \frac{h_{ef}}{t_{ef}} \cdot \sqrt{\frac{f_k}{E}} = 0.61$

From eq.G.3 $u = \frac{\lambda - 0.063}{0.73 - 1.17\frac{e_{m.k}}{t}} = 0.82$

Reduction factor at middle of wall - eq.G.1 $\Phi_m = \max\left(A_1 \cdot e^{\frac{-u^2}{2}}, 0\right) = 0.64$

Reduction factor for slenderness and eccentricity $\Phi = \min\left(\Phi_i, \Phi_m\right) = 0.64$

Verification of unreinforced masonry walls subjected to mainly vertical loading - Section 6.1.2

Design value of the vertical load $N_{Ed} = \max\left(N_{i.d}, N_{m.d}\right) = 14.64 m \cdot kN \cdot m^{-2}$

Vertical resistance of wall - eq.6.2 $N_{Rd} = \Phi \cdot t \cdot f_d = 93.82 \cdot kN \cdot m^{-1}$

$OKifLT\left(N_{Ed}, N_{Rd}\right) = \text{"O.K."}$

External wall design

Wall Design

The external wall is formed from two stud walls separated by spacers to form a single wall 370mm thick. All the walls have doubled header & footers. The outer frame is formed from 89x38mm C24 CLS timber, sheathed on the outside by 11mm OSB3 sheets glued and nailed to the studs. This frame carries the roof loads and is supported by a 680mm high block wall. The inner frame is formed from 63x38mm C16 CLS timber with horizontal CLS rails at 600mm intervals forming a service void, which in turn are sheathed 18mm OSB anc 10mm Femacell. The frame carries the first floor joists on walls B,D,H,J & L. The spacers are formed from 11mm OSB plates 300x370mm glued at approx. 900mm intervals up the studs starting at the base of the outer stud. Outer studs are all the same height but the inner studs vary depending on location.

Drawings	**15 - Inner Frame**
	16 - Outer wall framing
	17 - Outer wall rafter alignment
	18 - External wall framing details
Spreadsheet	**Studs**

The wall will be analysed as two separate frames but allowing for the connection between them.

Stud spacing $s_{stud} = 0.612 \cdot m$

As the wall is made of multiple studs $k_{sys} = 1.1$

Outer Frame

The outer frame carries the roof loading as well as the weight of the rain screen.

The outer frames loading is detailed in the Rafters spreadsheet on the Studs Sheet, this spreadsheat also make an aproximation of the combined stress ratios for compression and bending allowing the worst case values to be more fully analysed.
Stud O5 has the highest value but this actually coincides with a window so will be delt with later, K7 is the next worst case and this will be analysed.

Design class $Class = 2$

Stud material $Material = \text{"Softwood C24"}$

Stud width $b = 38 \cdot mm$

Stud depth $h = 89 \cdot mm$

Stud height $l_s = 2.58 \cdot m$

Stud area $A_s = b \cdot h = 3382 \cdot mm^2$

Section modulus of stud about y-y axis $W_y = \frac{b \cdot h^2}{6}$

Actions

Compressive actions

Frame self weight (inc rain screen) $G_{self.k} = 1.53 \frac{kN}{m} \cdot s_{stud} = 0.94 \cdot kN$

Rafter dead load $G_{raf.k} = 9.50kN$

Snow on roof $Q_{s.k} = 2.5kN$

Wind on roof $Q_{w.r.k} = 0.36kN$

Bending action from wind on wall $\quad Q_{w.wall.k} = w_{w_{back,D}} = 0.45 \cdot \frac{kN}{m^2}$ $\quad$ From wind forces calculations

Find critical load combination

psi values relating to loads

$$\psi s = \begin{pmatrix} 1 & \textit{Permanent} & 1 \\ \psi val("S",0) & \textit{Short} & \psi val("S",2) \\ \psi val("W",0) & \textit{Instant} & \psi val("W",2) \end{pmatrix}$$

Create a list of the load combinations

$$Loads = Load\_Combos\left[\psi s, \begin{pmatrix} G_{self.k} + G_{raf.k} \\ Q_{s.k} \\ Q_{w.r.k} \end{pmatrix} \div N, \begin{pmatrix} \gamma_G \\ \gamma_Q \\ \gamma_Q \end{pmatrix}\right]$$

Then iterate the calculations for all combinations

$$c = 0..\ rows(Loads) - 1$$

$$LoadDuration_c = Loads_{c,d_l}$$

Compressive loads

$$N_{d_c} = Loads_{c,ULS} N$$

Design lateral load

$$W_{d_c} = \gamma_Q \cdot Q_{w.wall.k} \cdot Loads_{c,mQ_2}$$

($Loads_{c,mQ2}$ is combination multiplier for winds)

Design moment per stud

$$M_d = W_d \cdot s_{stud} \cdot \frac{l_s^2}{8}$$

Material properties

Material safety factor $\quad \gamma_M = get\_k(Material, Class, \gamma_M) = 1.3$

Duration safty factor $\quad k_{mod_c} = get\_k\left(Material, Class, k_{mod} + LoadDuration_c\right)$

Final deformation factor $\quad k_{def} = get\_k(Material, Class, k_{def}) = 0.8$

Characteristic material properties

$$f_{m.k} = Tc(Material, f_{m.0.k})\ MPa$$

$$f_{c.0.k} = Tc(Material, f_{c.0.k})\ MPa$$

$$f_{c.90.k} = Tc(Material, f_{c.90.k})\ MPa$$

$$f_{t.0.k} = Tc(Material, f_{t.0.k})\ MPa$$

$$E_{0.mean} = Tc(Material, E_{0.mean})\ MPa$$

$$E_{0.05} = Tc(Material, E_{0.05})\ MPa$$

$$G_{0.mean} = Tc(Material, G_{mean})\ MPa$$

$$k_{sys} = 1.1$$

Height modification

$$k_h = max\left[1, min\left[\left(\frac{Tc(Material, k_{h.d})}{h \div mm}\right)^{Tc(Material, k_{h.s})}, Tc(Material, k_{h.max})\right]\right] = 1.11$$

Design material properties $f_{m.d} = \frac{f_{m.k} \cdot k_{mod} \cdot k_h \cdot k_{sys}}{\gamma_M}$

$f_{t.0.d} = \frac{f_{t.0.k} \cdot k_{mod} \cdot k_h \cdot k_{sys}}{\gamma_M}$

$f_{c.0.d} = \frac{f_{c.0.k} \cdot k_{mod} \cdot k_{sys}}{\gamma_M}$

$f_{c.90.d} = \frac{f_{c.90.k} \cdot k_{mod} \cdot k_{sys}}{\gamma_M}$

Axial compression of stud

Design compression stress $\sigma_{c.0.d} = \frac{N_d}{A_s}$

The z-z axis is fully restrained by the sheathing, the y-y axis is restrained by the spacer plates.

Effecive length in y-y $l_{y.eff} = 0.9m$

Instability about y-y $k_{c.y} = calc\_k_c\left(h, l_{y.eff}, f_{c.0.k}, E_{0.05}\right) = 0.92$

ratio of stress/strength $r_c = \frac{\sigma_{c.0.d}}{k_{c.y} \cdot f_{c.0.d}}$

Moment on stud

Bending stress due to wind $\sigma_{m.y.d} = \frac{M_d}{W_y}$

ratio of stress/strength $r_m = \frac{\sigma_{m.y.d}}{f_{m.d}}$

Combined stress

$r = \max\left(r_c + r_m\right) = 1$

Bearing strength of the sole plates

Sole plate is continuously supported $k_{c.90} = 1.25$ [EC5 6.1.5(3)]

Effective area of bearing $A_b = (b + 60mm) \cdot h = 8722 \cdot mm^2$

Design compressive stress $\sigma_{c.90.d} = \frac{N_d}{A_b}$

ratio of stress/strength $r_{cs} = \max\left(\frac{\sigma_{c.90.d}}{k_{c.90} \cdot f_{c.90.d}}\right) = 1.02$ $Check\left(r_{cs}\right) =$ "******* THIS CHECK HAS FAILE

I think this value is close enough to be safe..

Corner studs

Most of the exterior and interior building corners support a hip or valley rafter. This stud is formed from three pieces of timber, 2 at right angles and the third at the rafter angle. All three pieces are glued together and are effectivly restrained in both y-y and z-z axises by the sheathing. The larger cross section and stresses no higher than the above mean no further calculations are needed.

Window and Door lintels

The maximum loading on a window lintel is from rafter O5, this rafter along with O4 bear on a lintel over a 1.8m window. The actual stud is only 470mm long. The lintel is formed from a thin webbed beam made up of the top plates and the lintel joined by the outer sheathing and a 11mm OSB web on the inside which are glued in place. Wind effects on such a small area will be negligable and can be ignored.

Spreadsheet Lintels

Main span length $L_s = 1.8m$

Number of flanges $f_n = 1$

Number of webs $w_n = 2$

Beam depth $h_b = 612mm$

Beam flanges are fully restrained $k_c = 1$

Load sharing is not active $k_{sys} = 1.0$

Flange

Flange material $Material_f =$ "Softwood C24"

Height of flange $h_f = 2 \times 38mm$

Width of flange $b_f = 89mm$

Web

Web material $Material_w =$ "OSB3 11mm"

Actions

Loads from rafter O5 and O4 (from Rafters sheet)

Dead load $G_k = \begin{pmatrix} 9.63 \\ 1.01 \end{pmatrix} kN$

Snow Load $Q_{s.k} = \begin{pmatrix} 2.7 \\ 0.26 \end{pmatrix} kN$ @ $pos = \begin{pmatrix} 0.6 \\ 1.2 \end{pmatrix} m$ from end

Wind load $Q_{w.k} = \begin{pmatrix} 0.67 \\ 0.31 \end{pmatrix} kN$

Find critical load combination

psi values relating to loads

$$\psi s = \begin{pmatrix} 1 & \textit{Permanent} & 1 \\ \psi val("S", 0) & \textit{Short} & \psi val("S", 2) \\ \psi val("W", 0) & \textit{Instant} & \psi val("W", 2) \end{pmatrix}$$

Create a list of the load combinations

$$Loads = Load\_Combos\left[\psi s, \begin{pmatrix} G_k \\ Q_{s.k} \\ Q_{w.k} \end{pmatrix} \div N, \begin{pmatrix} \gamma_G \\ \gamma_Q \\ \gamma_Q \end{pmatrix}\right]$$

Then iterate the calculations for all combinations — $c = 0 .. \mathrm{rows}(Loads) - 1$

Load duration for this load combo — $LoadDuration_c = Loads_{c,\boldsymbol{d_l}}$

ULS Actions

Calculate bending and shear for the given point loads — $MV_c = MV\_point\_loads\left(Loads_{c,\boldsymbol{ULS}}, \frac{pos}{m}, \frac{L_s}{m}\right)$

Design moment — $M_{d_c} = \left(MV_c\right)_{\boldsymbol{M_d}} \cdot N \cdot m$

Design shear force — $V_{d_c} = \max\left[\left(MV_c\right)_{\boldsymbol{SR_1}}, \left(MV_c\right)_{\boldsymbol{SR_2}}\right] N$

SLS Actions

$F_{SLS.i_c} = Loads_{c,\boldsymbol{SLS_i}} N$

$F_{SLS.f_c} = Loads_{c,\boldsymbol{SLS_f}} N$

Material characteristics

Flange material

Material safety factor — $\gamma_{M.f} = get\_k\left(Material_f, Class, \boldsymbol{\gamma_M}\right) = 1.3$

Duration modification factors for each load combo — $k_{mod.f_c} = get\_k\left(Material_f, Class, \boldsymbol{k_{mod}} + LoadDuration_c\right)$

Final deformation factor — $k_{def.f} = get\_k\left(Material_f, Class, \boldsymbol{k_{def}}\right) = 0.8$

Material design characteristics

$f_{m.f.k} = Tc\left(Material_f, \boldsymbol{f_{m.0.k}}\right) MPa$

$f_{c.0.f.k} = Tc\left(Material_f, \boldsymbol{f_{c.0.k}}\right) MPa$

$f_{t.0.f.k} = Tc\left(Material_f, \boldsymbol{f_{t.0.k}}\right) MPa$

$E_{0.mean.f} = Tc\left(Material_f, \boldsymbol{E_{0.mean}}\right) MPa$

Effective flange width — $b_{f.ef} = b_f \cdot f_n = 89 \cdot mm$

Web material

Material safety factor — $\gamma_{M.w} = get\_k\left(Material_w, Class, \boldsymbol{\gamma_M}\right) = 1.2$

Duration modification factors for each load combo — $k_{mod.w_c} = get\_k\left(Material_w, Class, \boldsymbol{k_{mod}} + LoadDuration_c\right)$

Final deformation factor — $k_{def.w} = get\_k\left(Material_w, Class, \boldsymbol{k_{def}}\right) = 2.25$

Material design characteristics

$f_{v.w.k} = Tc\left(Material_w, \boldsymbol{f_{v.k}}\right) MPa$

$f_{r.w.k} = Tc\left(Material_w, \boldsymbol{f_{r.90.k}}\right) MPa$

$f_{c.90.w.k} = Tc\left(Material_w, \boldsymbol{f_{c.90.k}}\right) MPa$

$f_{t.90.w.k} = Tc\left(Material_w, \boldsymbol{f_{t.90.k}}\right) MPa$

$$E_{c.90.mean.w} = Tc\left(Material_w, \boldsymbol{E_{tc.90.mean}}\right) MPa$$

$$G_{mean.w} = Tc\left(Material_w, \boldsymbol{G_{mean}}\right) MPa$$

Web thickness

$$b_w = Tc\left(Material_w, \boldsymbol{thick}\right) mm = 11 \cdot mm$$

Effective web thickness

$$b_{w.ef} = \begin{cases} \dfrac{b_w}{2} & \text{if } w_n = 1 \\ b_w & \text{otherwise} \end{cases} = 11 \cdot mm$$

Clear height of the web

$$h_w = h_b - 2h_f = 0.46\,m$$

Area of the web

$$A_w = h_b \cdot b_w \cdot w_n = 0.01\,m^2$$

Material characteristics - design

Flange

Height modification

$$k_{h.f} = \max\left[1, \min\left[\left(\frac{Tc\left(Material_f, \boldsymbol{k_{h.d}}\right)}{h_f \div mm}\right)^{Tc\left(Material_f, \boldsymbol{k_{h.s}}\right)}, Tc\left(Material_f, \boldsymbol{k_{h.max}}\right)\right]\right] = 1$$

Design characteristics

$$f_{m.f.d} = \frac{f_{m.f.k} \cdot k_{mod.f} \cdot k_{h.f} \cdot k_{sys}}{\gamma_{M.f}}$$

$$f_{t.0.f.d} = \frac{f_{t.0.f.k} \cdot k_{mod.f} \cdot k_{h.f} \cdot k_{sys}}{\gamma_{M.f}}$$

$$f_{c.0.f.d} = \frac{f_{c.0.f.k} \cdot k_{mod.f} \cdot k_{sys}}{\gamma_{M.f}}$$

$$f_{c.90.f.d} = \frac{f_{c.90.f.k} \cdot k_{mod.f} \cdot k_{sys}}{\gamma_{M.f}}$$

Web

$$f_{t.90.w.d} = \frac{f_{t.90.w.k} \cdot k_{mod.w} \cdot k_{sys}}{\gamma_{M.w}}$$

$$f_{c.90.w.d} = \frac{f_{c.90.w.k} \cdot k_{mod.w} \cdot k_{sys}}{\gamma_{M.w}}$$

$$f_{v.w.d} = \frac{f_{v.w.k} \cdot k_{mod.w} \cdot k_{sys}}{\gamma_{M.w}}$$

$$f_{r.w.d} = \frac{f_{r.w.k} \cdot k_{mod.w} \cdot k_{sys}}{\gamma_{M.w}}$$

Geometric properties – transformed sections

Instantaneous – transformed section properties:

Second moment of area of flanges

$$I_{f.ef} = \frac{b_{f.ef}}{12} \cdot \left(h_b^{\,3} - h_w^{\,3}\right) = 9.78 \times 10^8 \cdot mm^4$$

Transformed web thickness (into flange) $b_{w.tfd.i} = b_w \cdot \frac{E_{c.90.mean.w}}{E_{0.mean.f}} = 3 \cdot mm$

Second moment of area of web $I_{ef.w.i} = \frac{b_{w.tfd.i}}{12} \cdot h_b^3 = 5.73 \times 10^{-5} m^4$

Instantaneous second moment of area of the transformed section $I_{ef.i} = I_{ef.w.i} + I_{f.ef} = 1.04 \times 10^{-3} m^4$

Final – transformed section properties:

of web thickness $b_{w.tfd.f_c} = b_{w.tfd.i} \cdot \frac{1 + Loads_{c, \psi_2} \cdot k_{def.f}}{1 + Loads_{c, \psi_2} \cdot k_{def.w}}$

Second moment of area of web $I_{ef.w.f} = \frac{b_{w.tfd.f}}{12} \cdot h_b^3$

Final second moment of area of the transformed section $I_{ef.f} = I_{ef.w.f} + I_{f.ef}$

Bending stress check in the flanges and the web

Because the mean modulus of elasticity of the flange material is greater than that of the web,only check the stresses in the flanges at the final deformation condition and those in the web at the instantaneous condition.

Stress in flange due to bending – final condition:

Bending stress in top and bottom flange $\sigma_{m.max.f.d} = \frac{M_d}{I_{ef.f}} \cdot \frac{h_b}{2}$

Test against bending strength $r_{b.f} = \max\left(\frac{\sigma_{m.max.f.d}}{f_{m.f.d}}\right) = 0.13$ $Check(r_{b.f}) = $ "O.K."

Stress in web due to bending – instananeous condition:

Bending stress in the web $\sigma_{m.w.d} = \frac{M_d}{I_{ef.i}} \cdot \frac{h_b}{2} \cdot \left(\frac{E_{c.90.mean.w}}{E_{0.mean.f}}\right)$

Test against bending strength in compression $r_{b.w.c} = \max\left(\frac{\sigma_{m.w.d}}{f_{c.90.w.d}}\right) = 0.14$ $Check(r_{b.w.c}) = $ "O.K."

Test against bending strength in tension $r_{b.w.t} = \max\left(\frac{\sigma_{m.w.d}}{f_{t.90.w.d}}\right) = 0.25$ $Check(r_{b.w.t}) = $ "O.K."

Stress in the flange due to axial stress – final condition:

Axial stress in top and bottom flange $\sigma_{ax.f.d} = \frac{M_d}{I_{ef.f}} \cdot \left(\frac{h_b}{2} - \frac{h_f}{2}\right)$

Test against axial strength in compression $r_{ax.f.c} = \max\left(\frac{\sigma_{ax.f.d}}{f_{c.0.f.d} \cdot k_c}\right) = 0.15$ $Check(r_{ax.f.c}) = $ "O.K."

Test against axial strength in tension

$$r_{ax.f.t} = \max\left(\frac{\sigma_{ax.f.d}}{f_{t.0.f.d}}\right) = 0.19 \qquad \text{Check}\left(r_{ax.f.t}\right) = \text{"O.K."}$$

Buckling and shear stress check in the web

Buckling condition in the web in EC5

$$\text{ratio} = \frac{h_w}{b_w} = 41.82$$

Maxiumum value of the ratio is 70

$$r_{b.w} = \frac{\text{ratio}}{70} = 0.6 \qquad \text{Check}\left(r_{b.w}\right) = \text{"O.K."}$$

Shear strength of the web

Design shear force able to be taken by each web; EC5, equation (9.9))

$$F_{v.w.Ed} = \begin{cases} b_w \cdot h_w \cdot \left(1 + \frac{h_f}{h_w}\right) \cdot f_{v.w.d} & \text{if ratio} \le 35 \\ 35 b_w^{\,2} \left(1 + \frac{h_f}{h_w}\right) \cdot f_{v.w.d} & \text{otherwise} \end{cases}$$

Design shear force able to be taken by the beam

$$F_{v.Ed} = F_{v.w.Ed} \cdot w_n$$

Test shear force in web

$$r_{v.w} = \max\left(\frac{V_d}{F_{v.Ed}}\right) = 0.54 \qquad \text{Check}\left(r_{v.w}\right) = \text{"O.K."}$$

Shear strength of the glued joint between the web and the flanges

First moment of area of a flange about the NA,

$$S_f = b_{f.ef} \cdot h_f \cdot \left(\frac{h_b}{2} - \frac{h_f}{2}\right) = 1.81 \times 10^3 \cdot \text{cm}^3$$

Total length of the glue line in the flange

$$l_g = 2h_f = 0.15\,\text{m}$$

Shear stress in the glue line (instant.)

$$\tau_{mean.d.i} = \frac{V_d \cdot S_f}{I_{ef.i} \cdot l_g}$$

Shear stress in the glue line (final)

$$\tau_{mean.d.f} = \frac{V_d \cdot S_f}{I_{ef.f} \cdot l_g}$$

EC5 takes into account the effect of stress concentrations at the web/flange interface in the vicinity of position of the join to web when the height of the flange is greater than $4b_{w.ef}$

$$f_{v.90.d} = \begin{cases} f_{r.w.d} & \text{if } h_f \le 4b_{w.ef} \\ f_{r.w.d} \cdot \left(\frac{4b_{w.ef}}{h_f}\right)^{0.8} & \text{otherwise} \end{cases}$$

$$r_{v.g} = \max\left(\frac{\tau_{mean.d.i}}{f_{v.90.d}}, \frac{\tau_{mean.d.f}}{f_{v.90.d}}\right) = 0.65 \qquad \text{Check}\left(r_{v.g}\right) = \text{"O.K."}$$

Deflection of the beam at the SLS

At the instantaneous condition:

Calculate delection constant for instantaneous SLS
$$EIy_c = EIy\_point\_loads\left(F_{SLS.i_c}, pos, 1.8m\right)$$

Instantaneous deflection at mid-span
$$\mu_{inst} = \max\left(\frac{EIy}{E_{0.mean.f} \cdot I_{ef.i}}\right) = 0.13 \cdot mm$$

Allowable Instantaneous deflection at mid-span
$$\mu_{inst.allow} = \frac{L_s}{300} = 6 \cdot mm$$

$$r_{d.i} = \frac{\mu_{inst}}{\mu_{inst.allow}} = 0.02$$

$Check\left(r_{d.i}\right) = "O.K."$

At the final deformation condition:

transform of web thickness
$$b_{w.tfd.f} = b_{w.tfd.i} \cdot \frac{1 + k_{def.f}}{1 + k_{def.w}} = 1.66 \cdot mm$$

Second moment of area of web
$$I_{ef.w.f} = \frac{b_{w.tfd.f}}{12} \cdot h_b^{\,3} = 3.17 \times 10^7 \cdot mm^4$$

Second moment of area of beam
$$I_{ef.f} = I_{ef.w.f} + I_{f.ef} = 1.01 \times 10^{-3} m^4$$

Calculate delection constant for final SLS
$$EIy_c = EIy\_point\_loads\left(F_{SLS.f_c}, pos, 1.8m\right)$$

Final deflection at mid-span
$$\mu_{final} = \max\left[\frac{EIy \cdot \left(1 + k_{def.f}\right)}{E_{0.mean.f} \cdot I_{ef.i}}\right] = 0.17 \cdot mm$$

Final Instantaneous deflection at mid-span
$$\mu_{final.allow} = \frac{L_s}{250} = 7.2 \cdot mm$$

$$r_{d.f} = \frac{\mu_{final}}{\mu_{final.allow}} = 0.02$$

$Check\left(r_{d.f}\right) = "O.K."$

Support reaction

Area of support
$$A_s = (30mm + 38mm) b_f = 6052 \cdot mm^2$$

Compressive Stress
$$\sigma_{c.90.d} = \frac{V_d}{A_s}$$

Discrete supports with $L_s > 2h_b$
$$k_{c.90} = 1.5$$

Ratio of stress/strength
$$r_{c.s} = \max\left(\frac{\sigma_{c.90.d}}{k_{c.90} \cdot f_{c.90.f.d}}\right) = 0.36$$

Results of calculation

Maximum utility rate $\max\left(r_{b.f}, r_{b.w.c}, r_{b.w.t}, r_{ax.f.c}, r_{ax.f.t}, r_{b.w}, r_{v.w}, r_{v.g}, r_{d.i}, r_{d.f}, r_{c.s}\right) = 65\cdot\%$

Support stud for lintel

The lintel support is a standard stud along side a cripple stud supporting the lintel. The two studs are glued together resulting in a composite stud supporting rafter load and a lintel support reaction.

Support reaction 1 $sr_{1_c} = (MV_c)_1 N$ $\quad sr_1^T = (9.12\ \ 10.54\ \ 11.95\ \ 9.53\ \ 10.95\ \ 12.36\ \ 9.95\ \ 11.36\ \ 12.36)\cdot kN$

Support reaction 2 $sr_{2_c} = (MV_c)_2 N$ $\quad sr_2^T = (5.24\ \ 6.05\ \ 6.85\ \ 5.57\ \ 6.37\ \ 7.17\ \ 5.89\ \ 6.69\ \ 7.17)\cdot kN$

Support reaction 1 is the higher value so Sr_1 and load from O3 will be used

Stud material Material = "Softwood C24"

Stud width $b = 2\times 38\cdot mm$

Stud depth $h = 89\cdot mm$

Stud height $l_s = 2.58\cdot m$

Stud area $A_s = b\cdot h = 6764\cdot mm^2$

Section modulus of stud about y-y axis $W_y = \frac{b\cdot h^2}{6}$

Material safety factor $\gamma_M = get\_k(Material, Class, \gamma_M) = 1.3$

Height modification $k_h = \max\left[1, \min\left[\left(\frac{Tc(Material, k_{h.d})}{h \div mm}\right)^{Tc(Material, k_{h.s})}, Tc(Material, k_{h.max})\right]\right] = 1.11$

Actions from rafter

Compressive actions for the rafter load

Frame self weight (inc rain screen) $G_{self.k} = 1.53\frac{kN}{m}\cdot s_{stud} = 0.94\cdot kN$

Rafter dead load $G_{raf.k} = 1.01kN$

Snow on roof $Q_{s.k} = 0.26kN$ — Loads from rafter O3

Wind on roof $Q_{w.roof.k} = 0.31kN$

Bending action from wind on wall $Q_{w.wall.k} = w_{w_{back, D}} = 0.45\cdot\frac{kN}{m^2}$ — From wind forces calculations

Find critical load combination for rafter loads

psi values relating to loads $\psi s = \begin{pmatrix} 1 & \textit{Permanent} & 1 \\ \psi val("S", 0) & \textit{Short} & \psi val("S", 2) \\ \psi val("W", 0) & \textit{Instant} & \psi val("W", 2) \end{pmatrix}$

Create a list of the load combinations

$$Loads = Load\_Combos\left[\psi s, \begin{pmatrix} G_{self.k} + G_{raf.k} \\ Q_{s.k} \\ Q_{w.roof.k} \end{pmatrix} \div N, \begin{pmatrix} \gamma_G \\ \gamma_Q \\ \gamma_Q \end{pmatrix}\right]$$

Then iterate the calculations for all combinations

$$c = 0..\ rows(Loads) - 1$$

Combine rafter load and support reaction from lintel

Design Compressive loads

$$N_{d_c} = Loads_{c,0} N + sr_{1_c}$$ (Both lists are in the same load combination order)

$$N_d^T = (11749\ \ 13359\ \ 14969\ \ 12394\ \ 14004\ \ 15614\ \ 13039\ \ 14649\ \ 15614)\ N$$

Design lateral load

$$W_{d_c} = \gamma_Q \cdot Q_{w.wall.k} \cdot Loads_{c,mQ_2}$$ ($Loads_{c,mQ2}$ is combination multiplier for winds)

$$W_d^T = (0\ \ 0\ \ 0\ \ 340\ \ 340\ \ 340\ \ 680\ \ 680\ \ 340)\ Pa$$

Design moment per stud

$$M_d = W_d \cdot s_{stud} \cdot \frac{l_s^2}{8}$$

Material properties

Material safety factor

$$\gamma_M = get\_k(Material, Class, \gamma_M) = 1.3$$

Duration safty factor

$$k_{mod_c} = get\_k(Material, Class, k_{mod} + LoadDuration_c)$$

Final deformation factor

$$k_{def} = get\_k(Material, Class, k_{def}) = 0.8$$

Characteristic material properties

$$f_{m.k} = Tc(Material, f_{m.0.k})\ MPa$$

$$f_{c.0.k} = Tc(Material, f_{c.0.k})\ MPa$$

$$f_{c.90.k} = Tc(Material, f_{c.90.k})\ MPa$$

$$f_{t.0.k} = Tc(Material, f_{t.0.k})\ MPa$$

$$E_{0.mean} = Tc(Material, E_{0.mean})\ MPa$$

$$E_{0.05} = Tc(Material, E_{0.05})\ MPa$$

$$G_{0.mean} = Tc(Material, G_{mean})\ MPa$$

$$k_{sys} = 1.0$$

Height modification

$$k_h = max\left[1, min\left[\left(\frac{Tc(Material, k_{h.d})}{h \div mm}\right)^{Tc(Material, k_{h.s})}, Tc(Material, k_{h.max})\right]\right] = 1.11$$

Design material properties

$$f_{m.d} = \frac{f_{m.k} \cdot k_{mod} \cdot k_h \cdot k_{sys}}{\gamma_M}$$

$$f_{t.0.d} = \frac{f_{t.0.k} \cdot k_{mod} \cdot k_h \cdot k_{sys}}{\gamma_M}$$

$$f_{c.0.d} = \frac{f_{c.0.k} \cdot k_{mod} \cdot k_{sys}}{\gamma_M}$$

$$f_{c.90.d} = \frac{f_{c.90.k} \cdot k_{mod} \cdot k_{sys}}{\gamma_M}$$

Axial compression of stud

Design compression stress $\sigma_{c.0.d} = \frac{N_d}{A_s}$

The z-z axis is fully restrained by the sheathing being glued to the stud, the y-y axis is restrained by the spacer plates.

Effecive length in y-y $l_{y.eff} = 0.9m$

Instability about y-y $k_{c.y} = calc\_k_c\left(h, l_{y.eff}, f_{c.0.k}, E_{0.05}\right) = 0.92$

ratio of stress/strength $r_c = \frac{\sigma_{c.0.d}}{k_{c.y} \cdot f_{c.0.d}}$

Moment on stud

Bending stress due to wind $\sigma_{m.y.d} = \frac{M_d}{W_y}$

ratio of stress/strength $r_m = \frac{\sigma_{m.y.d}}{f_{m.d}}$

Combined stress

$r = \max\left(r_c + r_m\right) = 0.29$ Check(r) = "O.K."

Bearing strength of the sole plates

Sole plate is continuously supported $k_{c.90} = 1.25$ [EC5 6.1.5(3)]

Effective area of bearing $A_b = (b + 60mm) \cdot h = 12104 \cdot mm^2$

Design compressive stress $\sigma_{c.90.d} = \frac{N_d}{A_b}$

ratio of stress/strength $r_{cs} = \max\left(\frac{\sigma_{c.90.d}}{k_{c.90} \cdot f_{c.90.d}}\right) = 0.67$ Check$\left(r_{cs}\right)$ = "O.K."

Racking Resistance

The racking resistance of the exterior walls is provided by the outside frame and its sheathing. Although the sheathing will be glued in place this gluing will not be taken into account in the following calculations. Racking resistance is calculated to EC5 9.2.4.3 Simplified analysis of wall diaphragms – Method B as specified in the National annex.
Nails will be 2.8 x 50mm Galvanised Ring shank nails from Paslode.

Spreadsheet External wall panels

Common properties of the walls

Width of each stud $b_s = 38mm$

Depth of each stud $h_s = 89mm$

Wall height $h_w = 2.73m$

Stud spacing $s_s = 612mm$

Fastener spacing, $s_f = 150mm$

Sheathing material $Material_h = \text{"OSB3 11mm"}$

Density of sheathing $\rho_{h.k} = Tc\left(Material_h, \boldsymbol{\rho_k}\right) = 550$

Stud material $Material_s = \text{"Softwood C24"}$

Densisty of stud $\rho_{s.k} = Tc\left(Material_s, \boldsymbol{\rho_k}\right) = 350$

Minimum panel width $b_{p.min} = \dfrac{h_w}{4} = 0.683\,m$

Calculate nail shear strength for the sheathing fixings

Nail data from datasheet

Nail length $l_{nail} = 50mm$

Nail diameter $d_n = 2.8mm$

Nail head diameter $d_h = 7.25mm$

Characteristic yield moment of a nail, $M_{y.Rk} = 2860N\cdot mm$

Test density for nail data $\rho_{n.t} = 350$

Density limit for sheet material $\rho_{n.max.sheet} = 380$

Density limit for solid timber $\rho_{n.max.timber} = 550$

Allowable density for nail values $\rho_{h.a} = \begin{vmatrix} \rho_{h.k} \text{ if } \rho_{h.k} < \rho_{n.max.sheet} \\ \rho_{n.max.sheet} \text{ otherwise} \end{vmatrix} = 380$

Allowable density for nail values $\rho_{s.a} = \begin{vmatrix} \rho_{s.k} \text{ if } \rho_{s.k} < \rho_{n.max.timber} \\ \rho_{n.max.timber} \text{ otherwise} \end{vmatrix} = 350$

Pointside withdrawal resistance $f_{p.ax.k} = 7.79\cdot N\cdot mm^{-2}\cdot\left(\dfrac{\rho_{s.a}}{\rho_{n.t}}\right)^2 = 7.79\cdot N\cdot mm^{-2}$

Head pull through strength $f_{head.k} = 20.29 \cdot N \cdot mm^{-2} \cdot \left(\frac{\rho_{h.a}}{\rho_{n.t}}\right)^2 = 23.92 \cdot N \cdot mm^{-2}$

Calculations from EC5 8.2.2

Modification factor for joint $k_{mod} = \sqrt{get\_k\left(Material_h, Class, \boldsymbol{k_{mod}} + \boldsymbol{Instant}\right) \cdot get\_k\left(Material_s, Class, \boldsymbol{k_{mod}} + \boldsymbol{Instant}\right)}$

Thickness of sheathing material $t_h = 11mm$

Nail pointside penetration $t_p = l_{nail} - t_h = 39 \cdot mm$

$\frac{t_p}{d_n} = 13.9$ more than 8 so full values for pointside allowed

Material factor for connections $\gamma_{M.conn} = 1.3$

Characteristic embedment strength of timber parallel to the grain EC5, equation (8.15))

Pointside $f_{h.p.k} = 0.082 \cdot \rho_{s.k} \cdot \left(d_n \cdot mm^{-1}\right)^{-0.3} \cdot N \cdot mm^{-2} = 21.07 \cdot N \cdot mm^{-2}$

Headside $f_{h.h.k} = 65 \cdot \left(d_n \cdot mm^{-1}\right)^{-0.7} \cdot \left(\frac{t_p}{mm}\right)^{0.1} \cdot N \cdot mm^{-2} = 45.6 \cdot N \cdot mm^{-2}$

Characteristic withdrawal capacity of nail, Fax.Rk, is lesser of equations (EC5, equations (8.24))

Pointside $F_{ax.Rk.1} = f_{p.ax.k} \cdot d_n \cdot t_p = 850.67\,N$

Headside $F_{ax.Rk.2} = f_{head.k} \cdot d_h^{\,2} = 1.26 \times 10^3\,N$

$F_{ax.Rk} = \min\left(F_{ax.Rk.1}, F_{ax.Rk.2}\right) = 850.67\,N$

Load-carrying capacity of the connection

For a panel-to-timber joint with nails in single shear, the characteristic lateral resistance per shear plane is the smallest value of equations a-f (EC5, equations (8.6)) where:

$$\beta = \frac{f_{h.p.k}}{f_{h.h.k}} = 0.46$$

$$Ax = \frac{F_{ax.Rk}}{4}$$

$Ax_{percent} = 1 + 50\%$ 'other nails' [EC5 8.2.2(2)]

Mode a $F_{v.Rk.a} = f_{h.h.k} \cdot t_h \cdot d_n = 1405\,N$

Mode b $F_{v.Rk.b} = f_{h.p.k} \cdot t_p \cdot d_n = 2301\,N$

Mode c $F_{v.Rk.c.j} = \frac{f_{h.h.k} \cdot t_h \cdot d_n}{1+\beta} \cdot \left[\sqrt{\beta + 2 \cdot \beta^2 \cdot \left[1 + \frac{t_p}{t_h} + \left(\frac{t_p}{t_h}\right)^2\right] + \beta^3 \cdot \left(\frac{t_p}{t_h}\right)^2} - \beta \cdot \left(1 + \frac{t_p}{t_h}\right)\right]$

$F_{v.Rk.c} = F_{v.Rk.c.j} + Ax = 1079\,N$

$F_{v.Rk.cc} = F_{v.Rk.c.j} \cdot Ax_{percent} = 1299\,N$

Mode d

$$F_{v.Rk.d.j} = 1.05 \cdot \frac{f_{h.h.k} \cdot t_h \cdot d_n}{2+\beta} \cdot \left[\sqrt{2 \cdot \beta \cdot (1+\beta) + \frac{4 \cdot \beta \cdot (2+\beta) \cdot M_{y.Rk}}{f_{h.h.k} \cdot t_h^{\,2} \cdot d_n}} - \beta\right]$$

$$F_{v.Rk.d} = F_{v.Rk.d.j} + Ax = 823\,N$$

$$F_{v.Rk.dd} = F_{v.Rk.d.j} \cdot Ax_{percent} = 916\,N$$

Mode e

$$F_{v.Rk.e.j} = 1.05 \cdot \frac{f_{h.h.k} \cdot t_p \cdot d_n}{1+2\beta} \cdot \left[\sqrt{2 \cdot \beta^2 \cdot (1+\beta) + \frac{4 \cdot \beta \cdot (1+2 \cdot \beta) \cdot M_{y.Rk}}{f_{h.h.k} \cdot t_p^{\,2} \cdot d_n}} - \beta\right]$$

$$F_{v.Rk.e} = F_{v.Rk.e.j} + Ax = 1193\,N$$

$$F_{v.Rk.ee} = F_{v.Rk.e.j} \cdot Ax_{percent} = 1470\,N$$

Mode f

$$F_{v.Rk.f.j} = 1.15 \cdot \sqrt{\frac{2 \cdot \beta}{1+\beta} \cdot \left(2 \cdot M_{y.Rk} \cdot f_{h.h.k} \cdot d_n\right)}$$

$$F_{v.Rk.f} = F_{v.Rk.f.j} + Ax = 994\,N$$

$$F_{v.Rk.ff} = F_{v.Rk.f.j} \cdot Ax_{percent} = 1172\,N$$

The characteristic lateral resistance per shear plane per nail will be

$$F_{v.Rk} = \min\left(F_{v.Rk.a}, F_{v.Rk.b}, F_{v.Rk.c}, F_{v.Rk.cc}, F_{v.Rk.d}, F_{v.Rk.dd}, F_{v.Rk.e}, F_{v.Rk.ee}, F_{v.Rk.f}, F_{v.Rk.ff}\right) = 823\,N$$

Design shear resistance of the nail

$$F_{v.Rd} = \frac{k_{mod} \cdot F_{v.Rk}}{\gamma_{M.conn}} = 630\,N$$

Racking calculations

Buckling of the sheathing

Test ratio of clear span between studs and sheating thickness

$$br = \frac{\left(s_s - b_s\right)}{t_h}$$ max value is 100 OKifLT(br, 100) = "O.K."

Modification factors

Basic fastener spacing
EC5 equ. 9.26

$$s_0 = \frac{9700 \cdot 3 \cdot d_n}{\rho_{h.k}} = 148.15 \cdot mm$$

Fastener spacing factor
EC5 equ. 9.29

$$k_s = \frac{1}{0.86 \cdot \frac{s_f}{s_0} + 0.57} = 0.69$$

Sheathing material factor
only on one side EC5 equ 9.23

$$k_n = 1.0$$

Functions for panel dependent factors

Panel dimension factor
EC5 eq. 9.27

$$k_d(b_i) = \left| \begin{array}{l} r \leftarrow \dfrac{b_i}{h_w} \\ r \ \text{ if } \ r \le 1 \\ \text{otherwise} \\ \quad \left| \begin{array}{l} r^{0.4} \ \text{ if } \ b_i \le 4.8m \\ \left(\dfrac{4.8m}{h_w}\right)^{0.4} \ \text{ otherwise} \end{array} \right. \end{array} \right.$$

Uniformly distributed load factor
EC5 equ. 9.28

$$k_q(q_i, b_i) = 1 + \left[0.083 \cdot q_i \cdot \frac{m}{kN} - 0.0008 \cdot \left(q_i \cdot \frac{m}{kN}\right)^2\right] \cdot \left(\frac{2.4m}{b_i}\right)^{0.4}$$

Evaluation for each wall

Matrix of Panel widths for each wall
Rows are wall letters
Columns are panel numbers

$$\begin{array}{c} \begin{pmatrix} \text{"1"} & \text{"2"} & \text{"3"} \end{pmatrix} \\ b = \begin{pmatrix} 1.82 & 3.1 & 1.94 \\ 0.40 & 0.40 & 0.01 \\ 1.19 & 1.87 & 1.09 \\ 1.39 & 0.01 & 0.01 \\ 3.99 & 0.01 & 0.01 \\ 1.41 & 1.62 & 3.49 \\ 1.17 & 1.02 & 0.01 \\ 0.66 & 0.75 & 0.01 \\ 4.43 & 1.37 & 0.01 \\ 2.59 & 0.01 & 0.01 \\ 1.35 & 1.31 & 0.01 \\ 0.75 & 0.66 & 0.01 \\ 1.74 & 2.35 & 0.01 \\ 4.16 & 2.97 & 1.02 \end{pmatrix} \cdot m \quad \begin{pmatrix} A \\ B \\ C \\ D \\ E \\ H \\ I \\ J \\ K \\ L \\ M \\ N \\ O \\ P \end{pmatrix} \end{array}$$

Entries of 0.01 are actually zero but are given this value to prevent a divide by zero fault.
There are no walls F&G

Take the roof load from each rafter and convert it to a design UDL by using EC5 equation 9.31 factoring in wind direction in each of the principle directions. Minimum dead load and any wind uplift are factored using γ_G=1.35 and γ_Q=1.5 See Panels spreadsheet for calculations

$$q_{front} = \begin{pmatrix} 0 & 0 & 0 \\ 0.86 & 3.15 & 0 \\ 0 & 0 & 0 \\ 1.62 & 0 & 0 \\ 0 & 0 & 0 \\ 1.84 & 0.87 & 0.77 \\ 0 & 0 & 0 \\ 10.72 & 0.51 & 0 \\ 0 & 0 & 0 \\ 3.04 & 0 & 0 \\ 0 & 0 & 0 \\ 7.8 & 0.9 & 0 \\ 0 & 0 & 0 \\ 0.6 & 0.87 & 0.96 \end{pmatrix} \cdot \frac{kN}{m} \quad q_{left} = \begin{pmatrix} 1.55 & 1.28 & 1.39 \\ 0 & 0 & 0 \\ 0 & 2.29 & 0 \\ 0 & 0 & 0 \\ 2.3 & 0 & 0 \\ 0 & 0 & 0 \\ 0.5 & 0.6 & 0 \\ 0 & 0 & 0 \\ 0.33 & 1.13 & 0 \\ 0 & 0 & 0 \\ 1.57 & 4.66 & 0 \\ 0 & 0 & 0 \\ 0.48 & 0.79 & 0 \\ 0 & 0 & 0 \end{pmatrix} \cdot \frac{kN}{m} \quad q_{back} = \begin{pmatrix} 0 & 0 & 0 \\ 13.37 & 0.45 & 0 \\ 0 & 0 & 0 \\ 4.28 & 0 & 0 \\ 0 & 0 & 0 \\ 0.11 & 1.03 & 0.73 \\ 0 & 0 & 0 \\ 0.24 & 1.84 & 0 \\ 0 & 0 & 0 \\ 1.25 & 0 & 0 \\ 0 & 0 & 0 \\ 0.99 & 2.15 & 0 \\ 0 & 0 & 0 \\ 2.24 & 0.87 & 0.6 \end{pmatrix} \cdot \frac{kN}{m} \quad q_{right} = \begin{pmatrix} -0.06 & 0.18 & 1.44 \\ 0 & 0 & 0 \\ 0 & 0.77 & 0 \\ 0 & 0 & 0 \\ 0.2 & 0 & 0 \\ 0 & 0 & 0 \\ 0.73 & 0.32 & 0 \\ 0 & 0 & 0 \\ 1.48 & 3.03 & 0 \\ 0 & 0 & 0 \\ 1.8 & 0.33 & 0 \\ 0 & 0 & 0 \\ 1.1 & 0.03 & 0 \\ 0 & 0 & 0 \end{pmatrix} \cdot \frac{kN}{m}$$

Which walls are used to calculate racking strength for each wind direction

$$rw = \begin{array}{c} \begin{array}{cccc} f & l & b & r \end{array} \\ \begin{pmatrix} 0 & 1 & 0 & 1 \\ 1 & 0 & 1 & 0 \\ 0 & 1 & 0 & 1 \\ 1 & 0 & 1 & 0 \\ 0 & 1 & 0 & 1 \\ 1 & 0 & 1 & 0 \\ 0 & 1 & 0 & 1 \\ 1 & 0 & 1 & 0 \\ 0 & 1 & 0 & 1 \\ 1 & 0 & 1 & 0 \\ 0 & 1 & 0 & 1 \\ 1 & 0 & 1 & 0 \\ 0 & 1 & 0 & 1 \\ 1 & 0 & 1 & 0 \end{pmatrix} \end{array} \quad \begin{pmatrix} A \\ B \\ C \\ D \\ E \\ H \\ I \\ J \\ K \\ L \\ M \\ N \\ O \\ P \end{pmatrix}$$

Apply EC5 equation 9.25 to each panel and for each wind force direction (while testing for minimum panel width)

$w = 0..13 \qquad p = 0..2$

$$F_{v.front.Rd.p_{w,p}} = \frac{F_{v.Rd} \cdot b_{w,p}}{s_0} \cdot k_s \cdot k_n \cdot k_d(b_{w,p}) \cdot k_q\left[(q_{front})_{w,p}, b_{w,p}\right] \cdot (b_{w,p} > b_{p.min}) \cdot (rw_{w,front} \neq 0)$$

$$F_{v.left.Rd.p_{w,p}} = \frac{F_{v.Rd} \cdot b_{w,p}}{s_0} \cdot k_s \cdot k_n \cdot k_d(b_{w,p}) \cdot k_q\left[(q_{left})_{w,p}, b_{w,p}\right] \cdot (b_{w,p} > b_{p.min}) \cdot (rw_{w,left} \neq 0)$$

$$F_{v.back.Rd.p_{w,p}} = \frac{F_{v.Rd} \cdot b_{w,p}}{s_0} \cdot k_s \cdot k_n \cdot k_d(b_{w,p}) \cdot k_q\left[(q_{back})_{w,p}, b_{w,p}\right] \cdot (b_{w,p} > b_{p.min}) \cdot (rw_{w,back} \neq 0)$$

$$F_{v.right.Rd.p_{w,p}} = \frac{F_{v.Rd} \cdot b_{w,p}}{s_0} \cdot k_s \cdot k_n \cdot k_d(b_{w,p}) \cdot k_q\left[(q_{right})_{w,p}, b_{w,p}\right] \cdot (b_{w,p} > b_{p.min}) \cdot (rw_{w,right} \neq 0)$$

Sum the values for each panel to get a value for the wall

$$F_{v.front.Rd} = \sum_{p=0}^{2} F_{v.front.Rd.p}^{\langle p \rangle} \qquad F_{v.back.Rd} = \sum_{p=0}^{2} F_{v.back.Rd.p}^{\langle p \rangle}$$

$$F_{v.left.Rd} = \sum_{p=0}^{2} F_{v.left.Rd.p}^{\langle p \rangle} \qquad F_{v.right.Rd} = \sum_{p=0}^{2} F_{v.right.Rd.p}^{\langle p \rangle}$$

$$F_{v.front.Rd}^{T} = (0.00\ \ 0.00\ \ 0.00\ \ 2.43\ \ 0.00\ \ 17.61\ \ 0.00\ \ 0.65\ \ 0.00\ \ 8.98\ \ 0.00\ \ 1.19\ \ 0.00\ \ 26.02) \cdot kN$$

$$F_{v.left.Rd}^{T} = (19.20\ \ 0.00\ \ 7.37\ \ 0.00\ \ 15.79\ \ 0.00\ \ 2.76\ \ 0.00\ \ 18.47\ \ 0.00\ \ 5.02\ \ 0.00\ \ 9.78\ \ 0.00) \cdot k$$

$$F_{v.back.Rd}^{T} = (0.00\ \ 0.00\ \ 0.00\ \ 2.97\ \ 0.00\ \ 17.24\ \ 0.00\ \ 0.75\ \ 0.00\ \ 7.97\ \ 0.00\ \ 0.69\ \ 0.00\ \ 27.52) \cdot kN$$

$$F_{v.right.Rd}^{T} = (17.91\ \ 0.00\ \ 6.86\ \ 0.00\ \ 13.89\ \ 0.00\ \ 2.77\ \ 0.00\ \ 20.02\ \ 0.00\ \ 4.25\ \ 0.00\ \ 9.59\ \ 0.00) \cdot k$$

Sum the walls to give a total racking resistance for the building in each wind direction.

$$F_{v.Rd_{front}} = \sum F_{v.front.Rd} \qquad F_{v.Rd_{left}} = \sum F_{v.left.Rd} \qquad F_{v.Rd_{back}} = \sum F_{v.back.Rd} \qquad F_{v.Rd_{right}} = \sum F_{v.right.Rd}$$

Racking strength of the walls by direction

$$F_{v.Rd} = \begin{pmatrix} 56.87 \\ 78.4 \\ 57.15 \\ 75.3 \end{pmatrix} \cdot kN$$

Racking forces on the walls by direction

$$F_{rack} = \begin{pmatrix} 13.69 \\ 10.53 \\ 21.39 \\ 21.38 \end{pmatrix} \cdot kN$$ From Wind calculation sheets

Ratio of strength/strain $r_{rack} = \max\left(\frac{F_{rack}}{F_{v.Rd}}\right) = 0.37$ $OKifLT\left(r_{rack}, 1\right) = "O.K."$

Inner Frame

The inner frame only carries significant loads where it supports the first floor. When transfering the support reactions from the first floor we will use the ULSmax values from the joist calculations as these represent the critical load combination (Live +dead/kmod$_{med}$ > dead /kmod$_{perm}$). The joists are not aligned with the studs in general. Calculations on the worse case loads for bending ,shear and compression are shown below.

Inner frame is fully interior so $\quad Class = 1$

Header sizing $\quad b = 63 \cdot mm$

$h = 2 \times 38 \cdot mm$

$span = 0.6m$

Stud Sizes $\quad b_s = 38 \cdot mm$

$h_s = b = 63 \cdot mm$

$l_{s.y} = 0.9 \cdot m$

$l_{s.z} = 0.6 \cdot m$

$b_j = 45 \cdot mm$

Material = "Softwood C16"

Material safety factor $\quad \gamma_M = get\_k(Material, Class, \gamma_M) = 1.3$

Duration safty factor $\quad k_{mod} = get\_k(Material, Class, k_{mod} + Medium) = 0.8$

Final deformation factor $\quad k_{def} = get\_k(Material, Class, k_{def}) = 0.6$

Characteristic material properties $\quad f_{m.k} = Tc(Material, f_{m.0.k})\,MPa$

$f_{c.0.k} = Tc(Material, f_{c.0.k})\,MPa$

$f_{c.90.k} = Tc(Material, f_{c.90.k})\,MPa$

$f_{t.0.k} = Tc(Material, f_{t.0.k})\,MPa$

$E_{0.mean} = Tc(Material, E_{0.mean})\,MPa$

$E_{0.05} = Tc(Material, E_{0.05})\,MPa$

$G_{0.mean} = Tc(Material, G_{mean})\,MPa$

$k_{sys} = 1.1$

Height modification
$$k_h = max\left[1, min\left[\left(\frac{Tc(Material, k_{h.d})}{h \div mm}\right)^{Tc(Material, k_{h.s})}, Tc(Material, k_{h.max})\right]\right] = 1.15$$

Design material properties
$$f_{m.d} = \frac{f_{m.k} \cdot k_{mod} \cdot k_h \cdot k_{sys}}{\gamma_M}$$

$$f_{t.0.d} = \frac{f_{t.0.k} \cdot k_{mod} \cdot k_h \cdot k_{sys}}{\gamma_M}$$

$$f_{c.0.d} = \frac{f_{c.0.k} \cdot k_{mod} \cdot k_{sys}}{\gamma_M}$$

$$f_{c.90.d} = \frac{f_{c.90.k} \cdot k_{mod} \cdot k_{sys}}{\gamma_M}$$

Studs

$$A_s = b_s \cdot h_s = 2.39 \times 10^3 \cdot mm^2$$

Compression strength of a stud. $F_{c.s} = A_s \times f_{c.0.d} = 27.55 \cdot kN$

Buckling strength of a stud about the y-y axis, $F_{b.y.s} = A_s \cdot f_{c.0.d} \cdot calc\_k_c\left(h_s, I_{s.y}, f_{c.0.k}, E_{0.05}\right) = 21.28 \cdot kN$

Buckling strength of a stud about the z-z axis, $F_{b.z.s} = A_s \cdot f_{c.0.d} \cdot calc\_k_c\left(b_s, I_{s.z}, f_{c.0.k}, E_{0.05}\right) = 19.46 \cdot kN$

Maximum compressive load in a stud $F_{c.63C16} = \min\left(F_{c.s}, F_{b.y.s}, F_{b.z.s}\right) = 19.46 \cdot kN$

Top plates

$$W_y = \frac{h^2 \cdot b}{6} = 0.06 L$$

$$M_{max} = f_{m.d} \cdot W_y = 0.75 \cdot kN \cdot m$$

$$V_{max} = f_{v.d} \cdot h \cdot b \cdot \left(\frac{2}{3}\right) = 12.75 \cdot kN$$

Maximum bending occurs when the point load is central.

So maximum load $L_{bend.max} = \frac{M_{max}}{(span \div 2)} \times 2 = 5.02 \cdot kN$

Maximum shear load will be when 76mm from support (shear for position < h can be ignored [EC5 6.1.7(3)])

$$MV\_point\_loads[(1), (0.076), 0.6] = \begin{pmatrix} 0.07 \\ 0.87 \\ 0.13 \end{pmatrix}$$

$$L_{shear.max} = \frac{V_{max}}{0.87} = 14.65 \cdot kN$$

Bearing strength for each joist. As this could be aligned with a stud no enhancment of bearing area is allowed and area is limited by the size of the stud itself.

$$k_{c.90} = 1.0$$

$$A_{ef} = b \cdot b_s = 2394 \cdot mm^2$$

$$L_{bear.max} = k_{c.90} \cdot f_{c.90.d} \cdot A_{ef} = 3.57 \cdot kN$$

$$L_{max} = \min\left(L_{bend.max}, L_{shear.max}, L_{bear.max}, F_{b.y.s}, F_{b.z.s}\right) = 3.57 \cdot kN$$

Instantanious deflection of the header @ L_{max}

$$\frac{\left(L_{max} \cdot span^3\right)}{48 \cdot E_{0.mean} \cdot \frac{h^3 b}{12}} \times \left[1 + 1.2 \cdot \frac{E_{0.mean}}{G_{0.mean}} \cdot \left(\frac{h}{span}\right)^2\right] = 1.14 \cdot mm$$

The maximum ULS load from the joists is 2.65kN from J22C so the strength of the inner frame is OK

Window lintels

The longest window lintel is on wall H and will be formed by the top plates with 2 additional headers support by cripple studs. The lintel is 1.8m wide and has 4 joists loading it.

Header sizing

$b = 63 \cdot mm$

$h = 4 \times 38 \cdot mm$

$span = 1.8m$

$k_{sys} = 1.1$

Height modification

$$k_h = \max\left[1, \min\left[\left(\frac{Tc(Material, \boldsymbol{k_{h.d}})}{h \div mm}\right)^{Tc(Material, \boldsymbol{k_{h.s}})}, Tc(Material, \boldsymbol{k_{h.max}})\right]\right] = 1$$

Design material properties

$$f_{m.d} = \frac{f_{m.k} \cdot k_{mod} \cdot k_h \cdot k_{sys}}{\gamma_M}$$

$$f_{t.0.d} = \frac{f_{t.0.k} \cdot k_{mod} \cdot k_h \cdot k_{sys}}{\gamma_M}$$

$$f_{c.0.d} = \frac{f_{c.0.k} \cdot k_{mod} \cdot k_{sys}}{\gamma_M}$$

$$f_{c.90.d} = \frac{f_{c.90.k} \cdot k_{mod} \cdot k_{sys}}{\gamma_M}$$

Bending and shear value for the loads

$$MV = MV\_point\_loads\left[\begin{pmatrix} 2270 \\ 2270 \\ 2270 \\ 2260 \end{pmatrix}, \begin{pmatrix} 0.27 \\ 0.61 \\ 1.01 \\ 1.41 \end{pmatrix}, 1.8\right]$$

$$M_d = MV_{\boldsymbol{M_d}} \cdot N \cdot m = 2.38 \cdot kN \cdot m$$

$$V_d = \max\left(MV_{\boldsymbol{SR_1}}, MV_{\boldsymbol{SR_2}}\right) \cdot N = 4.92 \cdot kN$$

Bending stress

$$\sigma_{m.d} = \frac{M_d}{W_y} = 39.2 \cdot N \cdot mm^{-2}$$

ratio of stress to strength

$$r_m = \frac{\sigma_{m.d}}{f_{m.d}} = 3.62$$

$OKifLT\left(r_m, 1\right) =$ "******* VALU

Shear stress

$$\sigma_{v.d} = \frac{V_d}{h \cdot b} \cdot \frac{3}{2} = 0.77 \cdot N \cdot mm^{-2}$$

ratio of stress to strength

$$r_v = \frac{\sigma_{v.d}}{f_{v.d}} = 0.19$$

$OKifLT\left(r_v, 1\right) =$ "O.K."

Compression strength at the support

$A_{eff} = (b_s + 30mm) \cdot b = 4284 \cdot mm^2$

compression stress $\sigma_{c.0.d} = \frac{V_d}{A_{eff}} = 1.15 \cdot N \cdot mm^{-2}$

ratio of stress to strength $r_c = \frac{\sigma_{c.0.d}}{f_{c.0.d}} = 0.1$ $OKifLT(r_c, 1) = "O.K."$

Lintel support

Check the strength of the combined cripple stud+standard stud under the Support reaction of the lintel and a J22A.

Stud width $b = 2 \times 38 \cdot mm$

Stud depth $h = 63 \cdot mm$

Stud height $l_s = 2.66 \cdot m$

Stud area $A_s = b \cdot h = 4788 \cdot mm^2$

Section modulus of stud about y-y axis $W_y = \frac{b \cdot h^2}{6}$

Combine joist load and support reaction from lintel

Design Compressive loads $N_d = 2.26kN + MV_{\boldsymbol{SR_2}} \cdot N = 6.41 \cdot kN$

$k_{sys1.0} = 1.0$

Design values $f_{c.0.d} = \frac{f_{c.0.k} \cdot k_{mod} \cdot k_{sys}}{\gamma_M}$

$f_{c.90.d} = \frac{f_{c.90.k} \cdot k_{mod} \cdot k_{sys}}{\gamma_M}$

Axial compression of stud

Design compression stress $\sigma_{c.0.d} = \frac{N_d}{A_s}$

The z-z axis is restrained by counter battens, the y-y axis is restrained by the spacer plates.

Effecive length in y-y $l_{y.eff} = 0.9m$

Instability about y-y $k_{c.y} = calc\_k_c(h, l_{y.eff}, f_{c.0.k}, E_{0.05}) = 0.77$

Effecive length in z-z $l_{z.eff} = 0.6m$

Instability about z-z $k_{c.y} = calc\_k_c(b, l_{z.eff}, f_{c.0.k}, E_{0.05}) = 0.95$

y-y instability is higher so

ratio of stress/strength $r_c = \frac{\sigma_{c.0.d}}{k_{c.y} \cdot f_{c.0.d}} = 0.12$ $Check(r_c) = "O.K."$

Bearing strength of the sole plates

Sole plate is continuously supported $k_{c.90} = 1.25$ [EC5 6.1.5(3)]

Effective area of bearing $A_b = (b + 60mm)\cdot h = 8568\cdot mm^2$

Design compressive stress $\sigma_{c.90.d} = \frac{N_d}{A_b} = 0.75\cdot N\cdot mm^{-2}$

ratio of stress/strength $r_{cs} = \max\left(\frac{\sigma_{c.90.d}}{k_{c.90}\cdot f_{c.90.d}}\right) = 0.4$ Check$\left(r_{cs}\right)$ = "O.K."

Foundation design

Foundation will be complete ground bearing slab, placed 500mm below ground level, with enhancements for higher load areas.
Slab will be divided with compressable seperators to allow for thermal exspansion due to heat storage tanks below.
Foundation will have no overburden. Soil is a Sandy Gravel with little fines. Water table is approx 10m below Ground level. Loads are calculated in the Foundation sheet and the following are the maximums for each load group.

1. Skylight columns - Base plate and foundation pad (Column A1 has the highest loads - see roof design)
2. Wall D1 - carries roof + first floor + entertainment room
3. Wall D2 - Entertainment room+ First floor
4. Wall N2 - carries first floor only
5. Wall O - roof loads only

Drawings **19 - Foundation marking**
20 - Foundations
Spreadsheet **Foundations**

Properties common to all designs

Depth of soil over foundation $h_{soil} = 0mm$

Depth of water over foundation $h_{water} = 0mm$

Partial factors for geotechnical design

Design combinations used $c = 0..1$

Actions - Table A.3

Permanent unfavourable action $\gamma_{Gg} = \begin{pmatrix} 1.35 \\ 1.0 \end{pmatrix}$

Variable unfavourable action $\gamma_{Qg} = \begin{pmatrix} 1.5 \\ 1.3 \end{pmatrix}$

Soil - Table A.4

Angle of shearing resistance $\gamma_{\phi'} = \begin{pmatrix} 1.0 \\ 1.25 \end{pmatrix}$

Effective cohesion $\gamma_{c'} = \begin{pmatrix} 1.0 \\ 1.25 \end{pmatrix}$

Density $\gamma_{\gamma} = \begin{pmatrix} 1.0 \\ 1.0 \end{pmatrix}$

Spread foundations - Table A.5

Bearing $\gamma_{R.v} = \begin{pmatrix} 1.0 \\ 1.4 \end{pmatrix}$

Sliding $\gamma_{R.h} = \begin{pmatrix} 1.0 \\ 1.1 \end{pmatrix}$

Soil properties

Density of soil $\gamma_{soil} = 18kN \cdot m^{-3}$

Characteristic cohesion $c'_k = 0kN \cdot m^{-2}$

Characteristic effective shear resistance angle $\phi'_k = 35deg$

Design angle of shearing resistance $\phi'_d = \mathrm{atan}\left(\frac{\tan(\phi'_k)}{\gamma_{\phi'}}\right)$ $\phi'_d = \begin{pmatrix} 35 \\ 29.26 \end{pmatrix} \cdot deg$

Design cohesion $c'_d = \frac{c'_k}{\gamma_{\phi'}}$ $c'_d = \begin{pmatrix} 0 \\ 0 \end{pmatrix}$

Bearing resistance factors

$N_{q_c} = e^{\pi \tan(\phi'_{d_c})} \cdot \left(\tan\left(45deg + \frac{\phi'_{d_c}}{2}\right)\right)^2$ $N_q = \begin{pmatrix} 33.3 \\ 16.92 \end{pmatrix}$

$N_{c_c} = \left(N_{q_c} - 1\right)\cot(\phi'_{d_c})$ $N_c = \begin{pmatrix} 46.12 \\ 28.42 \end{pmatrix}$

$N_{\gamma_c} = 2\left(N_{q_c} - 1\right)\tan(\phi'_{d_c})$ $N_\gamma = \begin{pmatrix} 45.23 \\ 17.84 \end{pmatrix}$

Concrete details (Table 3.1 - Strength and deformation characteristics for concrete)

Concrete strength class of foundation C25/30

Characteristic compressive cylinder strength $f_{c.k} = 25MPa$

Characteristic compressive cube strength $f_{c.k.cube} = 20MPa$

Compressive strength coefficient (cl.3.1.6(1)) $\alpha_{cc} = 0.85$

Design compressive concrete strength (exp3.15) $f_{c.d} = \alpha_{cc} \cdot \left(\frac{f_{c.k}}{\gamma_C}\right) = 14.17 \cdot MPa$

Mean value of compressive cylinder strength $f_{c.m} = f_{c.k} + 8MPa = 33 \cdot MPa$

Mean value of axial tensile strength $f_{c.t.m} = 0.3 \cdot MPa \left(\frac{f_{c.k}}{MPa}\right)^{\frac{2}{3}} = 2.56 \cdot MPa$

5% fractile of axial tensile strength $f_{c.t.k.0.05} = 0.7 f_{c.t.m} = 1.8 \cdot MPa$

Secant modulus of elasticity of concrete $E_{c.m} = 22 \cdot GPa \left(\frac{f_{c.m}}{10MPa}\right)^{0.3} = 3.15 \times 10^4 \cdot MPa$

Tens.strength coeff.for plain concrete (cl.12.3.1(1)) $\alpha_{ct.pl} = 0.8$

Design tens.strength for plain concrete (exp.12.1) $f_{ct.d.pl} = \alpha_{ct.pl} \cdot \frac{f_{c.t.k.0.05}}{\gamma_C} = 0.96 \cdot MPa$

$C_{Rd.c} = \frac{0.18MPa}{\gamma_C} = 0.12 \cdot MPa$

Maximum aggregate size $h_{agg} = 30mm$

Limiting crack width $w_{max} = 0.3mm$

Crack width coefficients $k_1 = 0.8$

$k_2 = 0.5$

$k_3 = 3.4$

$k_4 = 0.425$

Reinforcement details

Characteristic yield strength $f_{y.k} = 500MPa$

Design yield strength (fig. 3.8) $f_{y.d} = f_{y.k} \div \gamma_S = 434.78 \cdot MPa$

Modulus of elasticity of reinforcement $E_s = 210GPa$

Nominal cover to reinforcement $c_{nom} = 30mm$

1 - Skylight columns - Base plate and foundation pad

Characteristic loads

Characteristic permanent vertical load $G_k = 33.77kN$

Characteristic variable vertical load $Q_k = 14.94kN$

Base plate design

Design force

Compressive axial force $N_{c.Ed} = \gamma_G \cdot G_k + \gamma_Q \cdot Q_k = 68 \cdot kN$

Applied shear force $V_{Ed} = 0kN$

Column details

Column section RHS 100x50x3.0

Size in x $L_{c.x} = 100mm$

Size in y $L_{c.y} = 50mm$

Wall Thickness $t_c = 3mm$

Steel grade S275

Nominal yield strength $f_{yp.c} = 275MPa$

Nominal ultimate tensile strength $f_{u.c} = 410MPa$

Baseplate details

Size in x $L_{p.x} = 200mm$

Size in y $L_{p.y} = 150mm$

Thickness $t_p = 5mm$

Column eccentricity x-direction $e_{bpx} = 0mm$

Column eccentricity ydirection $e_{bpy} = 0mm$

Steel grade S275

Nominal yield strength $f_{yp.p} = 275MPa$

Nominal ultimate tensile strength $f_{u.p} = 410MPa$

Foundation details

Size in x $L_{f.x} = .9m$

Size in y $L_{f.y} = .7m$

Depth of concrete base $d_f = 250mm$

Dist CL baseplate to edge of concrete $x_{ce} = \frac{L_{f.x}}{2} = 0.45\,m$

Dist CL baseplate to edge of concrete (-ve) y-dir $y_{ce} = \frac{L_{f.y}}{2} = 0.35\,m$

Area of foundation $A_f = L_{f.x} \cdot L_{f.y} = 0.63\,m^2$

Limiting projection edge plate to edge conc x-dir $e_x = x_{ce} - \left(\frac{L_{p.x}}{2}\right) = 0.35\,m$

Limiting projection edge plate to edge conc y-dir $e_y = y_{ce} - \left(\frac{L_{p.y}}{2}\right) = 0.28\,m$

Maximum projection $e_{max} = \max\left(e_x, e_y\right) = 0.35\,m$

Foundation bearing strength

Foundation joint material coefficient $\beta_j = 0.667$

Projection beyond b'plate for fdn distribution area $h_{lim} = \min\left(2e_x, 2e_y, 2\min\left(L_{p.x}, L_{p.y}\right), d_f\right) = 0.25\,m$

Area of base plate $A_p = L_{p.x} \cdot L_{p.y} = 0.03\,m^2$

Area of distributed foundation $A_{pd} = \left(L_{p.x} + h_{lim}\right) \cdot \left(L_{p.y} + h_{lim}\right) = 0.18\,m^2$

Geometric enhancement coefficient $\alpha = \min\left[\left(\frac{A_{pd}}{A_p}\right)^{0.5}, 1 + \frac{d_f}{\max\left(L_{p.x}, L_{p.y}\right)}, 1 + 2\left(\frac{e_x}{L_{p.x}}\right), 1 + 2\left(\frac{e_y}{L_{p.y}}\right), 3\right] = 2.25$

Foundation bearing strength $f_{jd} = \beta_j \cdot \alpha \cdot f_{c.d} = 21.26 \cdot MPa$

Area of foundation required $A_{req} = \frac{N_{c.Ed}}{f_{jd}} = 3198 \cdot mm^2$

Effective area of base plate

Additional bearing width (6.2.5(4)) $c_0 = t_p \cdot \left(\frac{\min\left(f_{yp.c}, f_{yp.p}\right)}{3f_{jd} \cdot \gamma_{M0}}\right)^{0.5} = 10.38 \cdot mm$

Projection from outside face x-dir (–ve) $c_1 = \min\left[c_0, \frac{\left(L_{p.x} - L_{c.x}\right)}{2} + e_{bpx}\right] = 10.38 \cdot mm$

Projection from outside face x-dir (+ve) $c_2 = \min\left[c_0, \frac{\left(L_{p.x} - L_{c.x}\right)}{2} - e_{bpx}\right] = 10.38 \cdot mm$

Projection from inside face x-dir (+/-ve) $c_3 = \min\left[c_0, \frac{\left(L_{c.x} - 2t_c\right)}{2}\right] = 10.38 \cdot mm$

Projection from outside face y-dir (–ve) $c_4 = \min\left[c_0, \frac{\left(L_{p.y} - L_{c.y}\right)}{2} + e_{bpy}\right] = 10.38 \cdot mm$

Projection from outside face y-dir (+ve)
$$c_5 = \min\left[c_0, \frac{(L_{p.y} - L_{c.y})}{2} - e_{bpy}\right] = 10.38 \cdot mm$$

Projection from inside face y-dir (+/-ve)
$$c_6 = \min\left[c_0, \frac{(L_{c.y} - 2t_c)}{2}\right] = 10.38 \cdot mm$$

Effective area x-dir (-ve)
$$A_1 = (t_c + c_1 + c_3)(L_{c.y} + c_4 + c_5) = 1682 \cdot mm^2$$

Effective area x-dir (+ve)
$$A_2 = (t_c + c_2 + c_3)(L_{c.y} + c_4 + c_5) = 1682 \cdot mm^2$$

Effective area y-dir (-ve)
$$A_3 = (t_c + c_4 + c_6)\max\left[0mm, L_{c.x} - 2(t_c + c_3)\right] = 1740 \cdot mm^2$$

Effective area y-dir (+ve)
$$A_4 = (t_c + c_5 + c_6)\max\left[0mm, L_{c.x} - 2(t_c + c_3)\right] = 1740 \cdot mm^2$$

Total effective area
$$A_{eff} = A_1 + A_2 + A_3 + A_4 = 6844 \cdot mm^2$$

$$\text{OKifLT}(A_{req}, A_{eff}) = \text{"O.K."}$$

Weld strength

Weld length
$$l_{weld} = 2(L_{c.x} + L_{c.y}) = 300 \cdot mm$$

Weld leg length
$$s_{ww} = 4mm$$

Weld throat dimension
$$a_{ww} = \frac{1}{\sqrt{2}} \cdot s_{ww} = 2.83 \cdot mm$$

Correlation factor for fillet welds (Table 4.1)
$$\beta_w = 0.85$$

Design shear strength (4.5.3.3(3))
$$f_{vw.d} = \frac{\min(f_{u.c}, f_{u.p})}{\sqrt{3} \cdot \beta_w \cdot \gamma_{M2}} = 222.79 \cdot MPa$$

Design resistance per unit length (4.5.3.3(2))
$$F_{w.Rd.w} = f_{vw.d} \cdot a_{ww} = 630.14 \cdot \frac{N}{mm}$$

Design resistance
$$f_{vw.d.w} = F_{w.Rd.w} \cdot l_{weld} = 189.04 \cdot kN$$

$$\text{OKifLT}(N_{c.Ed}, f_{vw.d.w}) = \text{"O.K."}$$

Foundation Pad (both load combinations)

Foundation loads

Self weight
$$F_{swt} = d_f \cdot \gamma_{conc} = 6.13 \cdot kN \cdot m^{-2}$$

Soil weight
$$F_{soil} = h_{soil} \cdot \gamma_{soil} = 0$$

Bearing resistance (Section 6.5.2)

Forces on foundation

Design Force in z-axis
$$F_{z.d} = \gamma_{Gg} \cdot \left[A_f \cdot (F_{swt} + F_{soil}) + G_k\right] + \gamma_{Qg} \cdot Q_k \qquad F_{z.d} = \begin{pmatrix} 73.21 \\ 57.05 \end{pmatrix} \cdot kN$$

Design moment
$$M_{d.x} = F_{z.d} \cdot \frac{L_{f.x}}{2} \qquad M_{d.x} = \begin{pmatrix} 32.95 \\ 25.67 \end{pmatrix} \cdot kN \cdot m$$

$$M_{d.y} = F_{z.d} \cdot \frac{L_{f.y}}{2} \qquad M_{d.y} = \begin{pmatrix} 25.62 \\ 19.97 \end{pmatrix} \cdot kN \cdot m$$

Eccentricity of base reaction

in the x-axis $e_x = 0$ load is central

in the y-axis $e_y = 0$

Effective area of base

Effective length $L'_{f.x} = L_{f.x} - 2e_x = 0.9\,m$

Effective width $L'_{f.y} = L_{f.y} - 2e_x = 0.7\,m$

Effective area $A'_f = L'_{f.x} \cdot L'_{f.y} = 0.63\,m^2$

Pad base pressure

Design base pressure $f_{z.d} = \dfrac{F_{z.d}}{A'_f}$ $f_{z.d} = \begin{pmatrix} 116.21 \\ 90.56 \end{pmatrix} \cdot kN \cdot m^{-2}$

Net ultimate bearing capacity under drained conditions (Annex D.4)

Effective overburden pressure $q = (d_f + h_{soil})\gamma_{soil} - h_{water} \cdot \gamma_{water} = 4.5 \cdot kN \cdot m^{-2}$

Design effective overburden pressure $q' = \dfrac{q}{\gamma_\gamma}$ $q' = \begin{pmatrix} 4.5 \\ 4.5 \end{pmatrix} \cdot kN \cdot m^{-2}$

Load inclination factors

$i_q = 1.0$

$i_\gamma = 1.0$

$i_c = 1.0$

Foundation shape factors $s_q = 1 + \dfrac{L'_{f.y}}{L'_{f.x}} \sin(\phi'_d) = \begin{pmatrix} 1.45 \\ 1.38 \end{pmatrix}$

$s_\gamma = 1 - 0.3 \cdot \dfrac{L'_{f.y}}{L'_{f.x}} = 0.77$

$s_{c_c} = \dfrac{s_{q_c} \cdot N_{q_c} - 1}{N_{q_c} - 1}$ $s_c = \begin{pmatrix} 1.46 \\ 1.4 \end{pmatrix}$

Net ultimate bearing capacity $n_{f_c} = c'_{d_c} \cdot N_{c_c} \cdot s_{c_c} \cdot i_c + q'_c \cdot N_{q_c} \cdot s_{q_c} \cdot i_q + 0.5\gamma_{soil} \cdot L'_{f.y} \cdot N_{\gamma_c} \cdot s_\gamma \cdot i_\gamma$

$n_f = \begin{pmatrix} 435.13 \\ 191.24 \end{pmatrix} \cdot kN \cdot m^{-2}$

$r_{bc} = \dfrac{f_{z.d}}{n_f} = \begin{pmatrix} 0.27 \\ 0.47 \end{pmatrix}$ $Check(r_{bc}) = $ "O.K."

Check if plain concrete allowed [EC2 12.9.3]

Maximum projection $e_{max} = 0.35\,m$

Limit of projection $e_{lim} = \dfrac{0.85 \cdot d_f}{\sqrt{\dfrac{3 f_{z.d_0}}{f_{ct.d.pl}}}} = 0.352\,m$

$OKifLT\left(e_{max}, e_{lim}\right) = "O.K."$

2 - Wall D1 Outside corner of Entertainment room

This is the highest loaded foundation strip with 3 walls loading it.

Number of walls $w := 1..3$

Strip Foundation details

Size in x $L_{f.x} := 1m$

Size in y $L_{f.y} := .8m$

Depth of foundation $d_f := 100mm$

Self weight $F_{swt} := L_{f.y} \cdot d_f \cdot \gamma_{conc} = 1.96 \cdot kN \cdot m^{-1}$

Wall no.1 details

Width of wall $l_{y.1} := 100mm$

position in y-axis $y_1 := 0.1m$

Permanent load in z $F_{Gz_1} := 5.6kN \cdot m^{-1}$

Variable load in z $F_{Qz_1} := 2.1kN \cdot m^{-1}$

Wall no.2 details

Width of wall $l_{y.2} := 63mm$

position in y-axis $y_2 := 0.389m$

Permanent load in z $F_{Gz_2} := 2.2kN \cdot m^{-1}$

Variable load in z $F_{Qz_2} := 2.7kN \cdot m^{-1}$

Wall no.3 details

Width of wall $l_{y.3} := 100mm$

position in y-axis $y_3 := 0.520m$

Permanent load in z $F_{Gz_3} := 14.8kN \cdot m^{-1}$

Variable load in z $F_{Qz_3} := 0.0kN \cdot m^{-1}$

Bearing resistance (Section 6.5.2)

Forces on foundation

Design Force in z-axis

$$F_{z.d} := \left[\gamma_{Gg} \cdot \left(F_{swt} + \sum F_{Gz}\right) + \gamma_{Qg} \cdot \sum F_{Qz}\right] \cdot L_{f.x}$$

$$F_{z.d} = \begin{pmatrix} 40.36 \\ 30.8 \end{pmatrix} \cdot kN$$

Design moment

$$M_{d.y} := \left[\gamma_{Gg} \left[F_{swt} \cdot \frac{L_{f.y}}{2} + \sum_w \left(F_{Gz_w} \cdot y_w\right)\right] + \gamma_{Qg} \cdot \sum_w \left(F_{Qz_w} \cdot y_w\right)\right] \cdot L_{f.x}$$

$$M_{d.y} = \begin{pmatrix} 15.25 \\ 11.53 \end{pmatrix} \cdot kN \cdot m$$

Eccentricity of base reaction

in the y-axis

$$e_y = \frac{M_{d.y}}{F_{z.d}} - \frac{L_{f.y}}{2} \qquad e_y = \begin{pmatrix} -22.1 \\ -25.5 \end{pmatrix} \cdot mm$$

Effective area of base

Effective width

$$L'_{f.y} = L_{f.y} - 2e_y = \begin{pmatrix} 0.84 \\ 0.85 \end{pmatrix} m$$

Effecvtive length

$$L'_{f.x} = L_{f.x}$$

Effective area

$$A'_f = L'_{f.x} \cdot L'_{f.y} = \begin{pmatrix} 0.84 \\ 0.85 \end{pmatrix} m^2$$

Pad base pressure

Design base pressure

$$f_{z.d} = \frac{F_{z.d}}{A'_f} \qquad f_{z.d} = \begin{pmatrix} 47.8 \\ 36.19 \end{pmatrix} \cdot kN \cdot m^{-2}$$

Net ultimate bearing capacity under drained conditions (Annex D.4)

Effective overburden pressure

$$q = \left(d_f + h_{soil}\right)\gamma_{soil} - h_{water} \cdot \gamma_{water} = 1.8 \cdot kN \cdot m^{-2}$$

Design effective overburden pressure

$$q' = \frac{q}{\gamma_\gamma} \qquad q' = \begin{pmatrix} 1.8 \\ 1.8 \end{pmatrix} \cdot kN \cdot m^{-2}$$

Load inclination factors

$$i_q = 1.0$$

$$i_\gamma = 1.0$$

$$i_c = 1.0$$

Foundation shape factors

$$s_q = 1.000$$

$$s_\gamma = 1.000$$

$$s_c = 1.000$$

Net ultimate bearing capacity

$$n_{f_c} = c'_{d_c} \cdot N_{c_c} \cdot s_c \cdot i_c + q'_c \cdot N_{q_c} \cdot s_q \cdot i_q + 0.5\gamma_{soil} \cdot L'_{f.y_c} \cdot N_{\gamma_c} \cdot s_\gamma \cdot i_\gamma$$

$$n_f = \begin{pmatrix} 403.58 \\ 167.07 \end{pmatrix} \cdot kN \cdot m^{-2}$$

$$r_{bc} = \frac{f_{z.d}}{n_f} = \begin{pmatrix} 0.12 \\ 0.22 \end{pmatrix} \qquad \text{Check}\left(r_{bc}\right) = \text{"O.K."}$$

Concrete design

Reinforcement provided

Tension reinforcement provided A142 Mesh

Bar size $\phi_s = 6mm$

Bar spacing $s_s = 200mm$

Area of reinforcement $A_s = 142mm^2$

Rectangular section in flexure (Section 6.1)

Design bending moment $M_{Ed.x.max} = 1.4kN \cdot m$

Effective depth to tension reinforcement $d_x = d_f - c_{nom} - 0.5\phi_s = 67 \cdot mm$

K factor $K = \frac{M_{Ed.x.max}}{L_{f.y} \cdot d_x^2 \cdot f_{c.k}} = 0.016$

Redistribution ratio $\delta = 1.0$

K' factor $K' = 0.598\delta - 0.18\delta^2 - 0.21 = 0.208$

K < K' - Compression reinforcement is not required $A_{scx.p.req} = 0mm^2 m^{-1}$

Lever arm $z = \min\left[0.95 \cdot d_x, \frac{d_x}{2}\left(1 + \sqrt{1 - 3.53K}\right)\right] = 63.65 \cdot mm$

Depth of neutral axis $x = 2.5\left(d_x - z\right) = 8.38 \cdot mm$

Area of reinforcement required for bending $A_{sx.m} = \frac{M_{Ed.x.max}}{f_{y.d} \cdot z} = 51 \cdot mm^2$

Minimum area of reinforcement required $A_{sx.min} = \max\left(0.26\frac{f_{c.t.m}}{f_{y.k}}, 0.0013\right) \cdot L_{f.x} \cdot d_x = 89 \cdot mm^2$

Area of reinforcement required $A_{sx.req} = \max\left(A_{sx.m}, A_{sx.min}\right) = 89 \cdot mm^2$

$OKifLT\left(A_{sx.req}, A_s\right) = $ "O.K."

Crack control (Section 7.3)

Serviceability bending moment $M_{sls.x.max} = 1.1kN \cdot m$

Tensile stress in reinforcement $\sigma_s = \frac{M_{sls.x.max}}{A_s \cdot z} = 121.7 \cdot MPa$

Load duration factor $k_t = 0.4$

Effective depth of concrete in tension $h_{c.ef} = \min\left[2.5\left(d_f - d_x\right), \frac{d_f - x}{3}, \frac{d_f}{2}\right] = 30.54 \cdot mm$

Effective area of concrete in tension $A_{c.ef} = h_{c.ef} \cdot L_{f.x} = 0.03m^2$

Reinforcement ratio $\rho_{p.ef} = \frac{A_s}{A_{c.ef}} = 0.005$

Modular ratio $\alpha_e = \frac{E_s}{E_{c.m}} = 6.67$

Maximum crack spacing (exp.7.11) $s_{r.max} = k_3 \cdot c_{nom} + k_1 \cdot k_2 \cdot k_4 \cdot \frac{\phi_s}{\rho_{p.ef}} = 321.38 \cdot mm$

Maximum crack width (exp.7.8)
$$w_k = s_{r.max} \cdot \max\left[\frac{\sigma_s - \frac{k_t \cdot f_{c.t.m}}{\rho_{p.ef}} \cdot (1 + \alpha_e \cdot \rho_{p.ef})}{E_s}, \frac{0.6\sigma_s}{E_s}\right] = 0.112 \cdot mm$$

$$OKifLT(w_k, w_{max}) = \text{"O.K."}$$

Rectangular section in shear (Section 6.2)

Design shear force
$$V_{Ed.x.max} = 6.9kN$$

Effective depth factor
$$k = \min\left(2.0, 1 + \sqrt{\frac{200mm}{d_x}}\right) = 2$$

Reinforcement ratio
$$\rho_l = \min\left(0.02, \frac{A_s}{L_{f.x} d_x}\right) = 0.0021$$

Minimum shear strength
$$v_{min} = 0.035 \cdot MPa\, k^{1.5} \cdot \sqrt{\frac{f_{c.k}}{MPa}} = 0.49 \cdot MPa$$

Shear resistance
$$V_{Rd.c.x} = \max\left(v_{min}, C_{Rd.c}\, k \cdot \sqrt[3]{\frac{f_{c.k}}{MPa} \cdot 100 \cdot \rho_l}\right) \cdot (L_{f.x} \cdot d_x) = 33.16 \cdot kN$$

$$OKifLT(V_{Ed.x.max}, V_{Rd.c.x}) = \text{"O.K."}$$

3 - Wall D2 Entertainment room (Right side)

This is the highest loaded internal foundation strip with 2 walls loading it.

Number of walls $w = 1 .. 2$

Strip Foundation details

Size in x $L_{f.x} = 1m$

Size in y $L_{f.y} = .6m$

Depth of foundation $d_f = 100mm$

Self weight $F_{swt} = L_{f.y} \cdot d_f \cdot \gamma_{conc} = 1.47 \cdot kN \cdot m^{-1}$

Wall no.1 details

Width of wall $l_{y.1} = 100mm$

position in y-axis $y_1 = 0.225m$

Permanent load in z $F_{Gz_1} = 14.8kN \cdot m^{-1}$

Variable load in z $F_{Qz_1} = 0.0kN \cdot m^{-1}$

Wall no.2 details

Width of wall $l_{y.2} = 89mm$

position in y-axis $y_2 = 0.369m$

Permanent load in z

$$F_{Gz_2} = 6.3kN \cdot m^{-1}$$

Variable load in z

$$F_{Qz_2} = 7.0kN \cdot m^{-1}$$

Bearing resistance (Section 6.5.2)

Forces on foundation

Design Force in z-axis

$$F_{z.d} = \left[\gamma_{Gg} \cdot \left(F_{swt} + \sum F_{Gz}\right) + \gamma_{Qg} \cdot \sum F_{Qz}\right] \cdot L_{f.x}$$

$$F_{z.d} = \begin{pmatrix} 40.97 \\ 31.67 \end{pmatrix} \cdot kN$$

Design moment

$$M_{d.y} = \left[\gamma_{Gg}\left[F_{swt} \cdot \frac{L_{f.y}}{2} + \sum_{w}\left(F_{Gz_w} \cdot y_w\right)\right] + \gamma_{Qg} \cdot \sum_{w}\left(F_{Qz_w} \cdot y_w\right)\right] \cdot L_{f.x}$$

$$M_{d.y} = \begin{pmatrix} 12.1 \\ 9.45 \end{pmatrix} \cdot kN \cdot m$$

Eccentricity of base reaction

in the y-axis

$$e_y = \frac{M_{d.y}}{F_{z.d}} - \frac{L_{f.y}}{2} \qquad e_y = \begin{pmatrix} -4.6 \\ -1.5 \end{pmatrix} \cdot mm$$

Effective area of base

Effective width

$$L'_{f.y} = L_{f.y} - 2e_y = \begin{pmatrix} 0.61 \\ 0.6 \end{pmatrix} m$$

Effecvtive length

$$L'_{f.x} = L_{f.x}$$

Effective area

$$A'_f = L'_{f.x} \cdot L'_{f.y} = \begin{pmatrix} 0.61 \\ 0.6 \end{pmatrix} m^2$$

Pad base pressure

Design base pressure

$$f_{z.d} = \frac{F_{z.d}}{A'_f} \qquad f_{z.d} = \begin{pmatrix} 67.26 \\ 52.52 \end{pmatrix} \cdot kN \cdot m^{-2}$$

Net ultimate bearing capacity under drained conditions (Annex D.4)

Effective overburden pressure

$$q = \left(d_f + h_{soil}\right)\gamma_{soil} - h_{water} \cdot \gamma_{water} = 1.8 \cdot kN \cdot m^{-2}$$

Design effective overburden pressure

$$q' = \frac{q}{\gamma_\gamma} \qquad q' = \begin{pmatrix} 1.8 \\ 1.8 \end{pmatrix} \cdot kN \cdot m^{-2}$$

Load inclination factors

$$i_q = 1.0$$

$$i_\gamma = 1.0$$

$$i_c = 1.0$$

Foundation shape factors

$$s_q = 1.000$$

$$s_\gamma = 1.000$$

$$s_c = 1.000$$

Net ultimate bearing capacity

$$n_{f_c} = c'_{d_c} \cdot N_{c_c} \cdot s_c \cdot i_c + q'_c \cdot N_{q_c} \cdot s_q \cdot i_q + 0.5\gamma_{soil} \cdot L'_{f.y_c} \cdot N_{\gamma_c} \cdot s_\gamma \cdot i_\gamma$$

$$n_f = \begin{pmatrix} 307.88 \\ 127.26 \end{pmatrix} \cdot kN \cdot m^{-2}$$

$$r_{bc} = \frac{f_{z.d}}{n_f} = \begin{pmatrix} 0.22 \\ 0.41 \end{pmatrix}$$

$\text{Check}(r_{bc}) = \text{"O.K."}$

Concrete design

Reinforcement provided

Tension reinforcement provided A142 Mesh

Bar size $\phi_s = 6mm$

Bar spacing $s_s = 200mm$

Area of reinforcement $A_s = 142mm^2$

$$\frac{\pi}{s_s}\left(\frac{\phi_s}{2}\right)^2 = 141.37\frac{1}{m} \cdot mm^2$$

Rectangular section in flexure (Section 6.1)

Design bending moment $M_{Ed.x.max} = 1.7kN \cdot m$

Effective depth to tension reinforcement $d_x = d_f - c_{nom} - 0.5\phi_s = 67 \cdot mm$

K factor $K = \dfrac{M_{Ed.x.max}}{L_{f.y} \cdot d_x^2 \cdot f_{c.k}} = 0.025$

Redistribution ratio $\delta = 1.0$

K' factor $K' = 0.598\delta - 0.18\delta^2 - 0.21 = 0.208$

K < K' - Compression reinforcement is not required $A_{scx.p.req} = 0mm^2 m^{-1}$

Lever arm $z = \min\left[0.95 \cdot d_x, \dfrac{d_x}{2}\left(1 + \sqrt{1 - 3.53K}\right)\right] = 63.65 \cdot mm$

Depth of neutral axis $x = 2.5(d_x - z) = 8.38 \cdot mm$

Area of reinforcement required for bending $A_{sx.m} = \dfrac{M_{Ed.x.max}}{f_{y.d} \cdot z} = 61 \cdot mm^2$

Minimum area of reinforcement required $A_{sx.min} = \max\left(0.26\dfrac{f_{c.t.m}}{f_{y.k}}, 0.0013\right) \cdot L_{f.x} \cdot d_x = 89 \cdot mm^2$

Area of reinforcement required $A_{sx.req} = \max(A_{sx.m}, A_{sx.min}) = 89 \cdot mm^2$

$\text{OKifLT}(A_{sx.req}, A_s) = \text{"O.K."}$

Crack control (Section 7.3)

Serviceability bending moment $M_{sls.x.max} = 1.1kN \cdot m$

Tensile stress in reinforcement $\sigma_s = \dfrac{M_{sls.x.max}}{A_s \cdot z} = 121.7 \cdot MPa$

Load duration factor $k_t = 0.4$

Effective depth of concrete in tension $$h_{c.ef} = \min\left[2.5\left(d_f - d_x\right), \frac{d_f - x}{3}, \frac{d_f}{2}\right] = 30.54 \cdot mm$$

Effective area of concrete in tension $$A_{c.ef} = h_{c.ef} \cdot L_{f.x} = 0.03\,m^2$$

Reinforcement ratio $$\rho_{p.ef} = \frac{A_s}{A_{c.ef}} = 0.005$$

Modular ratio $$\alpha_e = \frac{E_s}{E_{c.m}} = 6.67$$

Maximum crack spacing (exp.7.11) $$s_{r.max} = k_3 \cdot c_{nom} + k_1 \cdot k_2 \cdot k_4 \cdot \frac{\phi_s}{\rho_{p.ef}} = 321.38 \cdot mm$$

Maximum crack width (exp.7.8) $$w_k = s_{r.max} \cdot \max\left[\frac{\sigma_s - \frac{k_t \cdot f_{c.t.m}}{\rho_{p.ef}} \cdot \left(1 + \alpha_e \cdot \rho_{p.ef}\right)}{E_s}, \frac{0.6\sigma_s}{E_s}\right] = 0.112 \cdot mm$$

$$\mathrm{OKifLT}\left(w_k, w_{max}\right) = \text{"O.K."}$$

Rectangular section in shear (Section 6.2)

Design shear force $$V_{Ed.x.max} = 8.7kN$$

Effective depth factor $$k = \min\left(2.0, 1 + \sqrt{\frac{200mm}{d_x}}\right) = 2$$

Reinforcement ratio $$\rho_l = \min\left(0.02, \frac{A_s}{L_{f.x} d_x}\right) = 0.0021$$

Minimum shear strength $$v_{min} = 0.035 \cdot MPa\,k^{1.5} \cdot \sqrt{\frac{f_{c.k}}{MPa}} = 0.49 \cdot MPa$$

Shear resistance $$V_{Rd.c.x} = \max\left(v_{min}, C_{Rd.c} k \cdot \sqrt[3]{\frac{f_{c.k}}{MPa} \cdot 100 \cdot \rho_l}\right) \cdot \left(L_{f.x} \cdot d_x\right) = 33.16 \cdot kN$$

$$\mathrm{OKifLT}\left(V_{Ed.x.max}, V_{Rd.c.x}\right) = \text{"O.K."}$$

4 - Wall N2 Great room Internal wall

This is the highest loaded internal foundation strip with 1 wall loading it.

Number of walls $w = 1..\,1$

Strip Foundation details

Size in x $L_{f.x} = 1m$

Size in y $L_{f.y} = .4m$

Depth of foundation $d_f = 100mm$

Self weight $F_{swt} = L_{f.y} \cdot d_f \cdot \gamma_{conc} = 0.98 \cdot kN \cdot m^{-1}$

Wall no.1 details

Width of wall $l_{y.1} = 144mm$

position in y-axis $y_1 = \frac{L_{f.y}}{2}$

Permanent load in z $F_{Gz_1} = 6.2kN \cdot m^{-1}$

Variable load in z $F_{Qz_1} = 8.0kN \cdot m^{-1}$

Bearing resistance (Section 6.5.2)

Forces on foundation

Design Force in z-axis $F_{z.d} = \left[\gamma_{Gg} \cdot \left(F_{swt} + \sum F_{Gz}\right) + \gamma_{Qg} \cdot \sum F_{Qz}\right] \cdot L_{f.x}$

$F_{z.d} = \begin{pmatrix} 21.69 \\ 17.58 \end{pmatrix} \cdot kN$

Design moment $M_{d.y} = \left[\gamma_{Gg}\left[F_{swt} \cdot \frac{L_{f.y}}{2} + \sum_w \left(F_{Gz_w} \cdot y_w\right)\right] + \gamma_{Qg} \cdot \sum_w \left(F_{Qz_w} \cdot y_w\right)\right] \cdot L_{f.x}$

$M_{d.y} = \begin{pmatrix} 4.34 \\ 3.52 \end{pmatrix} \cdot kN \cdot m$

Eccentricity of base reaction

in the y-axis $e_y = \frac{M_{d.y}}{F_{z.d}} - \frac{L_{f.y}}{2}$ $e_y = \begin{pmatrix} 0 \\ -2.8 \times 10^{-14} \end{pmatrix} \cdot mm$

Effective area of strip

Effective width $L'_{f.y} = L_{f.y} - 2e_y = \begin{pmatrix} 0.4 \\ 0.4 \end{pmatrix} m$

Effective length $L'_{f.x} = L_{f.x}$

Effective area $A'_f = L'_{f.x} \cdot L'_{f.y} = \begin{pmatrix} 0.4 \\ 0.4 \end{pmatrix} m^2$

Strip base pressure

Design base pressure $f_{z.d} = \frac{F_{z.d}}{A'_f}$ $f_{z.d} = \begin{pmatrix} 54.23 \\ 43.95 \end{pmatrix} \cdot kN \cdot m^{-2}$

Net ultimate bearing capacity under drained conditions (Annex D.4)

Effective overburden pressure $q = \left(d_f + h_{soil}\right)\gamma_{soil} - h_{water} \cdot \gamma_{water} = 1.8 \cdot kN \cdot m^{-2}$

Design effective overburden pressure $q' = \frac{q}{\gamma_\gamma}$ $q' = \begin{pmatrix} 1.8 \\ 1.8 \end{pmatrix} \cdot kN \cdot m^{-2}$

Load inclination factors $i_q = 1.0$

$i_\gamma = 1.0$

$i_c = 1.0$

Foundation shape factors $s_q = 1.000$

$s_\gamma = 1.000$

$s_c = 1.000$

Net ultimate bearing capacity $n_{f_c} = c'_{d_c} \cdot N_{c_c} \cdot s_c \cdot i_c + q'_c \cdot N_{q_c} \cdot s_q \cdot i_q + 0.5\gamma_{soil} \cdot L'_{f.y_c} \cdot N_{\gamma_c} \cdot s_\gamma \cdot i_\gamma$

$$n_f = \begin{pmatrix} 222.75 \\ 94.67 \end{pmatrix} \cdot kN \cdot m^{-2}$$

$$r_{bc} = \frac{f_{z.d}}{n_f} = \begin{pmatrix} 0.24 \\ 0.46 \end{pmatrix}$$

$Check(r_{bc}) = $ "O.K."

Concrete design

Check if plain concrete allowed [EC2 12.9.3]

Maximum projection $e_{max} = \frac{L_{f.y} - l_{y.1}}{2} = 0.13\,m$

Limit of projection $e_{lim} = \frac{0.85 \cdot d_f}{\sqrt{\frac{3 f_{z.d_0}}{f_{ct.d.pl}}}} = 0.21\,m$

$OKifLT(e_{max}, e_{lim}) = $ "O.K."

This result shows that the general 100mm thick slab is strong enough to support any of the internal walls outside of the Entertainment room.

<u>5 - Wall O Great room Rear</u>

This is the highest loaded external foundation strip with 2 walls loading it.

Number of walls $w = 1..\,2$

Strip Foundation details

Size in x $L_{f.x} = 1m$

Size in y $L_{f.y} = .6m$

Depth of foundation $d_f = 100mm$

Self weight $F_{swt} = L_{f.y} \cdot d_f \cdot \gamma_{conc} = 1.47 \cdot kN \cdot m^{-1}$

Wall no.1 details

Width of wall $l_{y.1} = 100mm$

position in y-axis $y_1 = 0.1m$

Permanent load in z $F_{Gz_1} = 9.5kN \cdot m^{-1}$

Variable load in z $F_{Qz_1} = 2.1kN \cdot m^{-1}$

Wall no.2 details

Width of wall $l_{y.2} = 63mm$

position in y-axis $y_2 = 0.389m$

Permanent load in z $F_{Gz_2} = 0.9kN \cdot m^{-1}$

Variable load in z $F_{Qz_2} = 0.0kN \cdot m^{-1}$

Bearing resistance (Section 6.5.2)

Forces on foundation

Design Force in z-axis

$$F_{z.d} = \left[\gamma_{Gg} \cdot \left(F_{swt} + \sum F_{Gz}\right) + \gamma_{Qg} \cdot \sum F_{Qz}\right] \cdot L_{f.x}$$

$$F_{z.d} = \begin{pmatrix} 19.18 \\ 14.6 \end{pmatrix} \cdot kN$$

Design moment

$$M_{d.y} = \left[\gamma_{Gg}\left[F_{swt} \cdot \frac{L_{f.y}}{2} + \sum_{w} \left(F_{Gz_w} \cdot y_w\right)\right] + \gamma_{Qg} \cdot \sum_{w} \left(F_{Qz_w} \cdot y_w\right)\right] \cdot L_{f.x}$$

$$M_{d.y} = \begin{pmatrix} 2.67 \\ 2.01 \end{pmatrix} \cdot kN \cdot m$$

Eccentricity of base reaction

in the y-axis

$$e_y = \frac{M_{d.y}}{F_{z.d}} - \frac{L_{f.y}}{2} \qquad e_y = \begin{pmatrix} -161 \\ -162 \end{pmatrix} \cdot mm$$

Effective area of base

Effective width

$$L'_{f.y} = L_{f.y} + 2e_y = \begin{pmatrix} 0.278 \\ 0.276 \end{pmatrix} m$$

Effecvtive length

$$L'_{f.x} = L_{f.x}$$

Effective area

$$A'_f = L'_{f.x} \cdot L'_{f.y} = \begin{pmatrix} 0.28 \\ 0.28 \end{pmatrix} m^2$$

Pad base pressure

Design base pressure

$$f_{z.d} = \frac{F_{z.d}}{A'_f} \qquad f_{z.d} = \begin{pmatrix} 68.97 \\ 52.92 \end{pmatrix} \cdot kN \cdot m^{-2}$$

Net ultimate bearing capacity under drained conditions (Annex D.4)

Effective overburden pressure

$$q = \left(d_f + h_{soil}\right)\gamma_{soil} - h_{water} \cdot \gamma_{water} = 1.8 \cdot kN \cdot m^{-2}$$

Design effective overburden pressure

$$q' = \frac{q}{\gamma_\gamma} \qquad q' = \begin{pmatrix} 1.8 \\ 1.8 \end{pmatrix} \cdot kN \cdot m^{-2}$$

Load inclination factors

$$i_q = 1.0$$

$$i_\gamma = 1.0$$

$$i_c = 1.0$$

Foundation shape factors

$$s_q = 1.000$$

$$s_\gamma = 1.000$$

$$s_c = 1.000$$

Net ultimate bearing capacity

$$n_{f_c} = c'_{d_c} \cdot N_{c_c} \cdot s_c \cdot i_c + q'_c \cdot N_{q_c} \cdot s_q \cdot i_q + 0.5\gamma_{soil} \cdot L'_{f.y_c} \cdot N_{\gamma_c} \cdot s_\gamma \cdot i_\gamma$$

$$n_f = \begin{pmatrix} 173.11 \\ 74.75 \end{pmatrix} \cdot kN \cdot m^{-2}$$

$$r_{bc} = \frac{f_{z.d}}{n_f} = \begin{pmatrix} 0.4 \\ 0.71 \end{pmatrix}$$

Check(r_{bc}) = "O.K."

Concrete design

Rectangular section in shear (Section 6.2)

Design shear force $V_{Ed.x.max} = 3.1kN$

Effective depth factor $k = \min\left(2.0, 1 + \sqrt{\frac{200mm}{d_f}}\right) = 2$

Minimum shear strength $v_{min} = 0.035 \cdot MPa\, k^{1.5} \cdot \sqrt{\frac{f_{c.k}}{MPa}} = 0.49 \cdot MPa$

Shear resistance $V_{Rd.c.x} = v_{min} \cdot \left(L_{f.x} \cdot d_x\right) = 33.16 \cdot kN$

OKifLT$\left(V_{Ed.x.max}, V_{Rd.c.x}\right)$ = "O.K."