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Structural Calculations

74 Fortune Green Road, London NW6 1DS

EX18/132/07

Revision	Date	Changes
-	13.08.18	Initial Issue
A	12.09.18	Design Revision because of revised architect drawings

General Construction Notes and Guidance on using these Calculations

1. Calculations are not to be used for the purpose of ordering materials and should only be used for Building Regulations submissions. All dimensions should be checked by the contractor on site.

2. All steelwork to be mechanically wire brushed and painted two coats of red oxide. Steelwork located in the cavity or below DPC to be suitably protected with 2 coats of bituminous paint.

3. All steelwork connections to use grade 8.8 bolts unless stated otherwise. These are to be spanner tightened using the appropriate podger spanner (min length 460mm) or suitable power tools in accordance with BS2583. If a torque wrench is used the torque applied should be around 90Nm for M16 bolts, 110Nm for M20 & 130Nm for M24.

4. All timber to be grade C24 (SC4), unless stated otherwise. Preservative treated to Architects details.

5. To be read in conjunction with Architects drawings, any inconsistencies between the drawings should be reported. If any site conditions or existing details are found that may affect the structural design, JMS Consulting Engineers are to be notified immediately.

6. For details of fire protection to steelwork, see Architects drawings.

7. The Contractor is to ensure that all existing construction is adequately supported, using needles and props as required. Where a new beam supports the existing construction, adequate pre-load is to be applied and suitable packs such as driven dry-slate introduced, then pointed up with mortar.

8. All blockwork to be 7.3 N/mm^2 in class III mortar below DPC in accordance with BS 5628 : Part 3 : 2005 or suitable 7.0 N/mm^2 foundation quality blocks in class II mortar in accordance with the manufacturer's instructions. All brickwork below DPC to be Engineering Bricks DPC in accordance with BS 5628 : Part 3 : 2005.

9. The project requires the introduction of heavy structural elements such as steel beams or concrete lintels. Although the Construction (Design and Management) Regulation 1994 would not normally apply to this type of construction, the designer still has an obligation to foresee risks and bring to the attention of the builder such risks. In consequence, the builder is to take into consideration the placement of all structural elements, ensuring that the method of lifting and placement is safely carried out. Responsibility for this element lies with the Contractor. As the existing walls need to be propped in order to introduce some of the lintels, this should also be considered in relationship to the risk assessment of the Contractor. Safe working procedures must be adopted. Responsibility for this element lies with the Contractor. Splice details for long-span beams can often be accommodated if required.

10. All construction products should be CE marked in accordance current legislation. This includes all fabricated structural steelwork in accordance with BS EN 1090-1 and BS EN 1090-2. The consequence class is CC2 unless noted otherwise. The service class is SC1 for all buildings, SC2 for all lifting beams, sculptures & fall arrest systems. Production category will be PC1 unless noted otherwise. All site welded items, S355 steelwork & CHS lattice girders will be PC2. As such the execution class for buildings will be EXC2.

11. CLIMATE CHANGE: The Building Research establishment have produced a document CBG 63 "Climate Change: impact on building design and construction". Part of their recommendations are that designers and builders should give consideration to:
    a. Increased wind loading by providing additional laps and fixings to roof coverings

    b. Consider foundation depth on shrinkable clays and to avoid future problems, increase the depth above standard requirements if there is a risk. This should be in accordance with the NHBC Standards, Chapter 4.2 Guidance on Building near Trees. If the calculations do not specifically design the depths of the foundations to take into account any local trees, then this should be checked and agreed with the Building Inspector on site.

Party Wall etc. Act 1996

If part of the work is adjacent to the boundary, the adjacent neighbours right to support could be affected; the issues associated with Party Wall Act may need to be considered. This may include providing information to the adjoining owner, giving sufficient notice of works in compliance with the Act. If the following list applies to this project then the Party Wall Act will apply.

- Installing a new beam into the shared wall between properties
- Demolishing, building or under-pinning an existing shared wall
- Building a new wall at or on the boundary or junction of two properties
- Damp-proofing all the way through a party wall
- Digging foundations that are within 3m of a Party Wall, where the new foundations are deeper than the existing ones
- Where the new foundations are within 6m and lower than a 45° line from the bottom of the existing foundations

Loading

Roof :	Tiles	=	0.65 kN/m²			Dead	Live
	Rafters, felt, insulation etc.	=	0.30 kN/m²				
			0.95 kN/m²/Cos 35	=	1.16 kN/m²		
	Plasterboard	=			0.25 kN/m²		
			TOTAL	**=**		**1.41 kN/m²**	
	Attic	=				0.25	
	Roof snow loading	=	0.6*((60-35)/30)	=		0.50 kN/m²	
			TOTAL	=			**0.75 kN/m²**
Roof :	Tiles	=	0.65 kN/m²			Dead	Live
	Plasterboard	=	0.25 kN/m²				
	Rafters, felt, insulation etc.	=	0.30 kN/m²				
			1.20 kN/m²/Cos 35	=	1.47 kN/m²		
			TOTAL	**=**		**1.47 kN/m²**	
	Attic	=				0.25	
	Roof snow loading	=	0.6*((60-35)/30)	=		0.50 kN/m²	
			TOTAL	=			**0.75 kN/m²**
Roof:	Joists & boarding, finishes	=	0.35 kN/m²				
(flat)	Plasterboard	=	0.25 kN/m²				
			TOTAL	**=**		**0.60 kN/m²**	
	Imposed	=	0.75 kN/m²				
			TOTAL	**=**			**0.75 kN/m²**
Floor:	PCU's	=	2.50 kN/m²				
	Screed	=	1.80 kN/m²				
	Plasterboard	=	0.25 kN/m²				
			TOTAL	**=**		**4.55 kN/m²**	
	Imposed loading	=				1.5	
	Partitions loading	=				1.0kN/m²	
			TOTAL	=			**2.5 kN/m²**
Floor:	Joists	=	0.15 kN/m²				
	Boarding	=	0.2 kN/m²				
	Plasterboard	=	0.25 kN/m²				
			TOTAL	**=**		**0.6 kN/m²**	
	Imposed loading	=				1.5	
	Partitions loading	=				0.5kN/m²	
			TOTAL	=			**2.0 kN/m²**
Walls:	2.4m high, 100mm blockwork	=	2.4*1.4	=	3.36 kN/m		
	Plasterwork both sides	=	2.4*0.25*2	=	1.2 kN/m		
			TOTAL	**=**		**4.56 kN/m**	
	2.4m high, 100mm brickwork	=	2.4*2.1	=	5.04 kN/m		
	Plasterwork both sides	=	2.4*0.25*2	=	1.2 kN/m		
			TOTAL	**=**		**6.24 kN/m**	
	2.4m high, studwork	=	2.4*0.12	=	0.29 kN/m		
	Plasterwork both sides	=	2.4*0.15*2	=	0.72 kN/m		
			TOTAL	**=**		**1.01 kN/m**	
	2.7m high, cavity wall blockwork	=	2.7*(2.1+1.4)	=	9.45 kN/m		
	Plasterwork to one side	=	2.4*0.25	=	0.60 kN/m		
			TOTAL	**=**		**10.05 kN/m**	
	2.7m high, solid brickwork	=	2.7*21*0.215	=	12.2 kN/m		
	Plasterwork to one side	=	2.4*0.25	=	0.60 kN/m		
			TOTAL	**=**		**12.8 kN/m**	
2.4m Bi-fold door:							
	(6mm float glass triple glazed)	=	2.4*0.75 kN/m²				
			TOTAL	**=**		**1.8 kN/m**	

Lintel Calculation (L1)

Solid Wall	12.80*2.7/2.7=12.8		
Parapet wall	12.80*1.0/2.7=4.75		
2nd Floor	2.40/2*0.60=0.72	2.40/2*2.0=2.40	
3rd Floor	2.40/2*0.60=0.72	2.40/2*2.0=2.40	
Joists from			
load bearing studs & roof	3.06*1.95/2.4=2.50	0.75*1.95/2.4=0.61	
Total	**21.5 kN/m Dead**	**5.41 kN/m Live**	**(Unfactored)**

UDL **(21.5+5.41)* 1.1=29.6 kN**

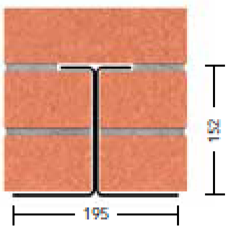
I BEAM (2C)

Manufactured length 150mm increments	600-2100	2250-3000
Height 'h'	152	152
Thickness	2.5	2.9
Total UDL kN	30	30

Suitable for 215mm solid walls.

To achieve loading figures lintel must be built in as shown. Lintels may be propped to facilitate speed of construction. See Lintel Installation on page 12.

Heavy Duty Load



Adopt I Beam (2C) IG lintel or similar.

Lintel Calculation (L2)

Solid Wall	12.80*2.7/2.7=12.8		
Parapet wall	12.80*1.0/2.7=4.75		
Railing	1.0		
2nd Floor	3.50/2*0.60=1.05	3.50/2*2.0=3.50	
3rd Floor	3.50/2*0.60=1.05	3.50/2*2.5=4.375	
Total	**20.65 kN/m Dead**	**7.875 kN/m Live**	**(Unfactored)**

UDL **(20.65+7.875)* 1.5=42.75 kN**

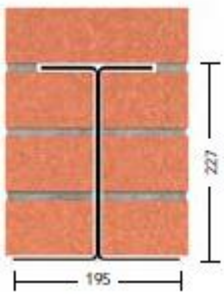
I BEAM (3C)

Manufactured length 150mm increments	600-1800	1950-2100	2250-3000	3150-4050	4200-4800
Height 'h'	227	227	227	227	227
Thickness	2.5	2.5	2.9	3.2	3.2
Total UDL kN	45	40	40	40	35

Suitable for 215mm solid walls.

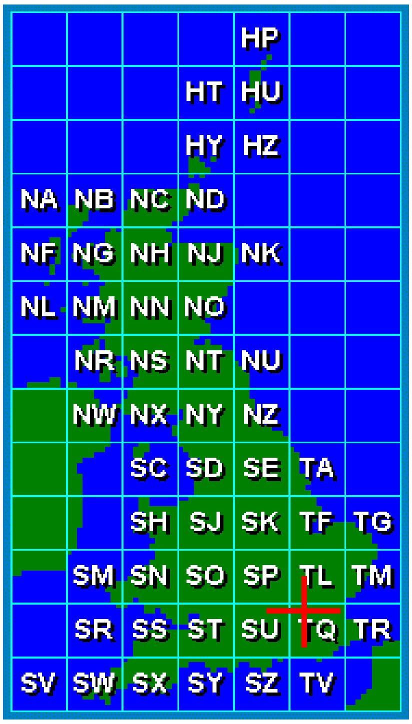
To achieve loading figures lintel must be built in as shown. Lintels may be propped to facilitate speed of construction. See Lintel Installation on page 12.

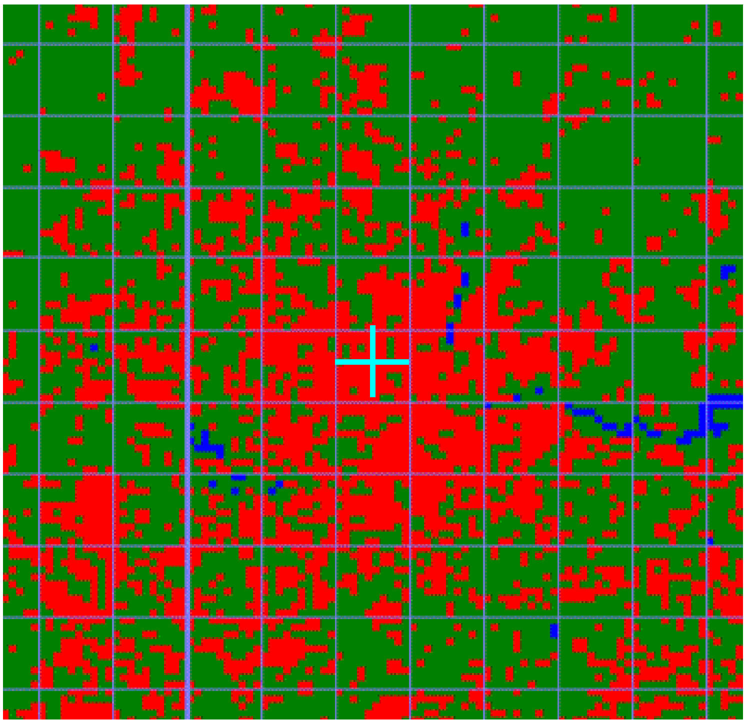
Extra Heavy Duty Load

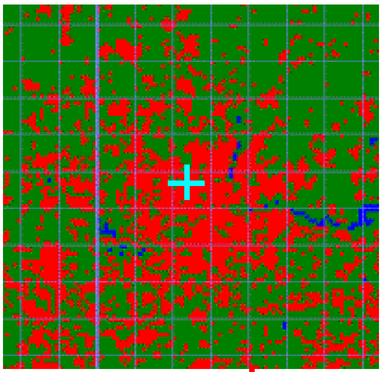


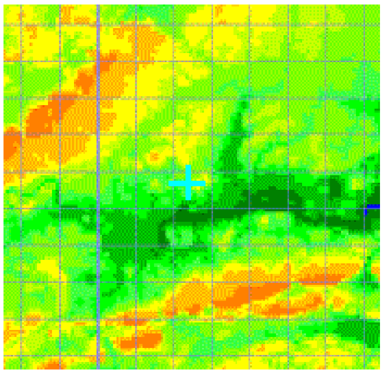
Adopt I Beam (3C) IG lintel or similar.

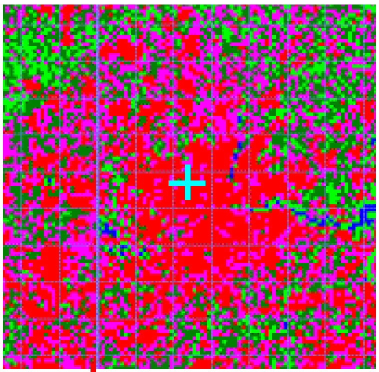
Wind Analysis Load

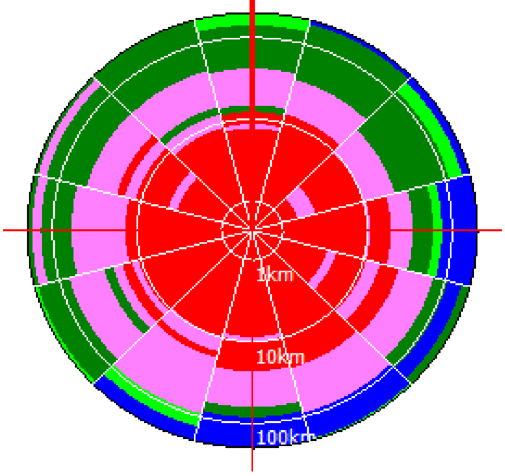


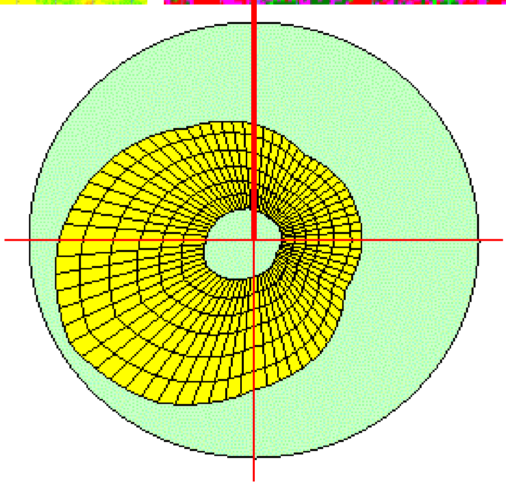












Site Basic Data

Location and Base wind speed — BREVe site data for TQ251855 - Base wind speed, Vb 21.53 m/s

JMS CIVIL & STRUCTURAL CONSULTING ENGINEERS		Project No	: EX18/132/07
LONDON • MIDLANDS • EAST ANGLIA • MANCHESTER • GREECE		Sheet	: Structure / 5
		Made By	: K Papadimitriou
		Date	: September 18
		Checked	: DJS
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Site Range	500 m
Altitude and Obstructions	Site altitude 102 m - Shelter effect from obstructions is included
Seasonal factor, Ss	Season length is All year - Seasonal factor, Ss 1.000
Annual risk and probability factor	Design annual risk 0.02 - Probability factor, Sp 1.000
Topographic Increments	Topographic increment from internal parameters
Heights and Diagonals (m)	Heights above ground 2; 5; 10; 15; 20; 30; 50 and 100

Direction Factors - Using UK direction Factors

Direction (°N)	0	30	60	90	120	150	180	210	240	270	300	330
Direction factor, Cdir	0.780	0.730	0.730	0.740	0.730	0.800	0.850	0.930	1.000	0.990	0.910	0.820

Topography

Crest Height (m)	10.0	5.5	56.5	29.0	19.0	29.0	45.4	62.5	25.0	30.3	26.0	18.0
Site Location (m)	0.0	999.0	999.0	525.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
Upwind Length (m)	200.0	96.0	1189.0	1088.0	317.0	363.0	605.0	781.0	263.0	433.0	325.0	200.0
Downwind Length (m)	133.0	96.0	10000	10000	238.0	580.0	908.0	10000	10000	10000	433.0	225.0
Base Altitude (m)	72.0	76.5	74.0	71.0	63.0	53.0	36.6	55.0	74.0	51.7	56.0	64.0
Upwind Slope	0.050	0.057	0.048	0.027	0.060	0.080	0.075	0.080	0.095	0.070	0.080	0.090

DESIGNER'S GUIDE METHOD

Direction (°N)	0	30	60	90	120	150	180	210	240	270	300	330
Direction factor, Cdir	0.780	0.730	0.730	0.740	0.730	0.800	0.850	0.930	1.000	0.990	0.910	0.820
Distance to Sea (km)	200.0	174.0	158.0	74.0	103.8	87.1	81.9	130.0	200.0	200.0	200.0	200.0
Distance in Town (km)	12.0	12.5	9.5	18.5	26.5	23.5	22.5	9.5	16.5	13.5	19.5	8.5
Altitude factor, Sa	1.072	1.077	1.074	1.071	1.063	1.053	1.037	1.055	1.074	1.052	1.056	1.064
Obstructions Height, Have (m)	10.0	10.0	10.0	10.0	10.0	10.0	10.0	10.0	10.0	10.0	10.0	10.0
Obstructions Spacing, Xo (m)	20.0	20.0	20.0	20.0	20.0	20.0	20.0	20.0	20.0	20.0	20.0	20.0
Displacement Height, Hdis (m)	8.0	8.0	8.0	8.0	8.0	8.0	8.0	8.0	8.0	8.0	8.0	8.0

Height Above Ground = 2.0 m

Direction (°N)	0	30	60	90	120	150	180	210	240	270	300	330
Effective height, Ze (m)	2.00	2.00	2.00	2.00	2.00	2.00	2.00	2.00	2.00	2.00	2.00	2.00
Orography factor, Co	0.099	0.000	0.037	0.028	0.120	0.160	0.151	0.161	0.190	0.140	0.160	0.179
Std. gust, t(s)	2.8	3.3	3.1	3.1	3.1	2.7	2.5	2.3	2.0	2.2	2.4	2.5
Gust Factor, Sg	0.997	0.961	0.998	0.983	0.976	1.017	1.040	1.054	1.050	1.007	1.012	1.026
Effective wind speed, Ve (m/s)	18.2	16.8	17.3	17.2	16.7	18.7	19.9	22.2	24.1	22.5	21.0	19.4
Dynamic Pressure, q (N/m²)	204.0	172.3	183.1	181.8	170.1	213.3	242.0	302.8	355.2	311.3	269.7	230.7

Height Above Ground = 5.0 m

Direction (°N)	0	30	60	90	120	150	180	210	240	270	300	330
Effective height, Ze (m)	2.00	2.00	2.00	2.00	2.00	2.00	2.00	2.00	2.00	2.00	2.00	2.00
Orography factor, Co	0.099	0.000	0.037	0.028	0.120	0.160	0.151	0.161	0.190	0.140	0.160	0.179
Std. gust, t(s)	2.8	3.3	3.1	3.1	3.1	2.7	2.5	2.3	2.0	2.2	2.4	2.5
Gust Factor, Sg	0.997	0.961	0.998	0.983	0.976	1.017	1.040	1.054	1.050	1.007	1.012	1.026
Effective wind speed, Ve (m/s)	18.2	16.8	17.3	17.2	16.7	18.7	19.9	22.2	24.1	22.5	21.0	19.4
Dynamic Pressure, q (N/m²)	204.0	172.3	183.1	181.8	170.1	213.3	242.0	302.8	355.2	311.3	269.7	230.7

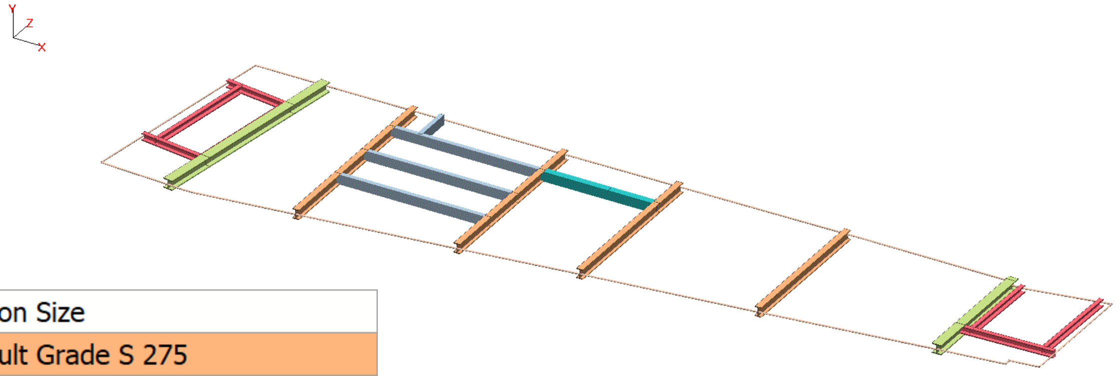
Height Above Ground = 10.0 m

Direction (°N)	0	30	60	90	120	150	180	210	240	270	300	330
Effective height, Ze (m)	4.00	4.00	4.00	4.00	4.00	4.00	4.00	4.00	4.00	4.00	4.00	4.00
Orography factor, Co	0.097	0.000	0.037	0.028	0.119	0.158	0.150	0.160	0.187	0.139	0.158	0.175
Std. gust, t(s)	2.1	2.4	2.3	2.3	2.2	2.0	1.9	1.6	1.5	1.6	1.7	1.9
Gust Factor, Sg	1.239	1.192	1.239	1.220	1.215	1.267	1.295	1.311	1.306	1.252	1.259	1.278
Effective wind speed, Ve (m/s)	22.0	20.2	20.8	20.8	20.2	22.6	24.1	27.0	29.2	27.3	25.4	23.5
Dynamic Pressure, q (N/m²)	298.0	249.4	266.1	264.0	249.2	313.6	355.6	445.4	523.3	457.1	396.6	339.4

Height Above Ground = 15.0 m

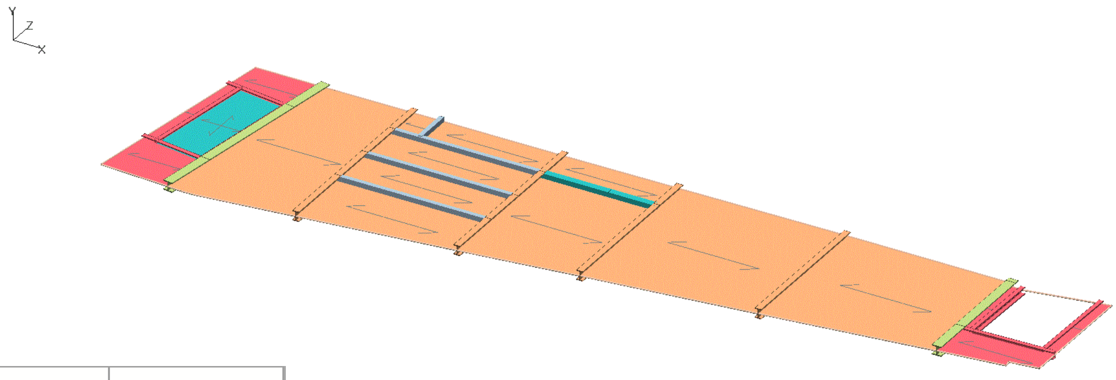
Direction (°N)	0	30	60	90	120	150	180	210	240	270	300	330
Effective height, Ze (m)	7.00	7.00	7.00	7.00	7.00	7.00	7.00	7.00	7.00	7.00	7.00	7.00
Orography factor, Co	0.095	0.000	0.037	0.028	0.116	0.156	0.149	0.159	0.183	0.137	0.156	0.171
Std. gust, t(s)	1.7	2.0	1.9	1.9	1.8	1.6	1.5	1.4	1.2	1.3	1.4	1.5
Gust Factor, Sg	1.432	1.377	1.433	1.411	1.407	1.467	1.499	1.518	1.510	1.448	1.456	1.477
Effective wind speed, Ve (m/s)	25.0	22.8	23.6	23.5	22.9	25.7	27.4	30.7	33.2	31.0	28.9	26.7
Dynamic Pressure, q (N/m²)	382.5	318.7	341.0	338.3	321.0	405.2	460.7	576.1	674.3	590.5	512.1	437.4

Steel Sections at Lower Ground floor structure over



Section Size
Default Grade S 275
203x133 UB 30
203x203 UC 46
150x90 PFC 23.9
94x200 C24
141x200 C24

Area Load Panels at Lower Ground floor structure over



Dead D	Live L
0.600	2.500
1.000	1.000
5.050	2.500

Member Loading

Load to B001 (Area loading included)

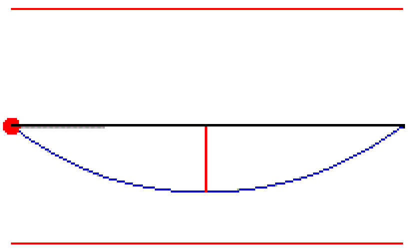
Beam & Block	5.05*1.5/2=3.78	2.50*1.5/2=1.875	
Ground Floor	2.70/2*0.60=0.81	2.70/2*2.5=3.375	
Glass wall	6.0		
Total	**10.59 kN/m Dead**	**5.25 kN/m Live**	**(Unfactored)**

AXIAL WITH MOMENTS (MEMBER)

Beam 001

Members 55-57 (N.2-N.12) @ Level 1 in Load Case 1

Member Loading and Member Forces
Loading Combination : 1 UT + 1.35 D1 + 1.5 L1



Part 1
D1 PTRY -000.232 0.000 0.234 -000.780
D1 PTRY -000.780 0.234 1.141 -000.739
L1 PTRY -000.965 0.000 0.234 -003.250
L1 PTRY -003.250 0.234 1.141 -003.080
D1 TRY -004.158 0.090 1.141 (kN,m,m)
L1 TRY -002.058 0.090 1.141 (kN,m,m)
D1 D 077.010 (kN/m³)
D1 UDLY -012.800 (kN/m)
Part 2
D1 PTRY -000.232 0.000 0.234 -000.780
D1 PTRY -000.780 0.234 1.141 -000.739
L1 PTRY -000.965 0.000 0.234 -003.250
L1 PTRY -003.250 0.234 1.141 -003.080
D1 TRY -004.158 0.090 1.141 (kN,m,m)
L1 TRY -002.058 0.090 1.141 (kN,m,m)
D1 D 077.010 (kN/m³)
D1 UDLY -012.800 (kN/m)
Part 3
D1 PTRY -000.232 0.000 0.234 -000.780
D1 PTRY -000.780 0.234 1.141 -000.739
L1 PTRY -000.965 0.000 0.234 -003.250
L1 PTRY -003.250 0.234 1.141 -003.080
D1 TRY -004.158 0.090 1.141 (kN,m,m)
L1 TRY -002.058 0.090 1.141 (kN,m,m)
D1 D 077.010 (kN/m³)
D1 UDLY -012.800 (kN/m)

Member Forces in Load Case 1 and Maximum Deflection from Load Case 3

Span No.	Axial Force (kN)	Shear Force (kN)		Bending Moment (kNm)		Maximum Moment (kNm)	Maximum Deflection (mm @ m)
		End 1	End 2	Node 1	Node 2		
	2	0.036	68.974	0.000	76.588	13.998	
35	0.038	-67.741	0.000	@ 2.352	@ 2.379		

Classification and Effective Area (EN 1993: 2006)

Section (46.1 kg/m)	203x203 UC 46 [S 275]		
Class = Fn(b/T,d/t,f_y,N,M_y,M_z)	9.25, 22.33, 275, 0, 76.57, 0	(Axial: Non-Slender)	Class 1
Auto Design Load Cases	1		

Moment Capacity Check M.c.y.Rd

$V_{y.Ed}/V_{pl.y.Rd}$	0.727 / 269.498 =	0.003	Low Shear
$M_{c.y.Rd} = f_y.W_{pl.y}/\gamma_{M0}$	275 x 497.4/1	136.785 kN.m	
$M_{y.Ed}/M_{c.y.Rd}$	76.573 / 136.785 =	0.560	OK

Equivalent Uniform Moment Factor C1

C_1= fn(M_1, M_2, M_o, ~y,~m)	0.1, 0.1, 76.5, 0.861, 300.000	1.127	Uniform

Lateral Buckling Check M.b.Rd

Le = 1.2L+2D	1.2 x 4.782 + 2 x 0.203 =	6.145 m	

$M_{cr} = Fn(C_1,L_e,I_z,I_t,I_w,E)$	1.127, 6.145, 1551, 22.15, 0.1429, 210000	166.838 kN.m	
$\lambda_{LT} = \sqrt{W.f_y/M_{cr}}$	√ 497.4 x 275 / 166.838	0.905	
$c_{LT} = Fn(\lambda_{LT}, \lambda_{LT}5950\ )$	0.905, 0.867	0.757	Curve b
$c_{LT}.mod = Fn(c_{LT},\lambda_{LT},k_c,f)$	0.757, 0.905, 0.942, 0.972	0.779	6.3.2.3
$M_{b.Rd} = c\ W_{pl.y}.f_y \le M_{c.y.Rd}$	0.779 x 497.4 x 275 ≤ 136.785 =	106.535 kN.m	
$M_{y.Ed}/M_{b.Rd}$	76.573 / 106.535	0.719	OK

Deflection Check - Load Case 3

Deflection Limits (Internal Beams)	In-span δ ≤ 4782/360 = 13.3 mm Live (Case 2)	2.74 mm	OK
	In-span δ ≤ 4782/250 = 19.1 mm D+L (Case 3)	14 mm	OK

Provide Section :- 203 x 203 UC 46 (S275)

Member Loading

Load to B002 (Area loading included)

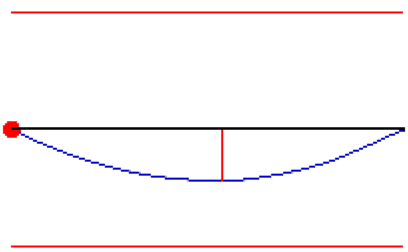
Ground Floor	5.90/2*0.60=1.77	5.90/2*2.5=7.375	
Total	**2.01 kN/m Dead**	**8.375 kN/m Live**	**(Unfactored)**

AXIAL WITH MOMENTS (MEMBER)

Beam 002

Members 50-53 (N.6-N.8) @ Level 1 in Load Case 1

Member Loading and Member Forces
Loading Combination : 1 UT + 1.35 D1 + 1.5 L1



Part 1
D1 TRY -001.373 0.405 1.595 (kN,m,m)
L1 TRY -005.720 0.405 1.595 (kN,m,m)
D1 PTRY -000.789 0.000 1.595 -000.716
L1 PTRY -003.288 0.000 1.595 -002.984
D1 D 077.010 (kN/m³)
Part 2
D1 TRY -001.373 0.405 1.595 (kN,m,m)
L1 TRY -005.720 0.405 1.595 (kN,m,m)
D1 PTRY -000.789 0.000 1.595 -000.716
L1 PTRY -003.288 0.000 1.595 -002.984
D1 D 077.010 (kN/m³)
Part 3
D1 TRY -001.373 0.405 1.595 (kN,m,m)
L1 TRY -005.720 0.405 1.595 (kN,m,m)
D1 PTRY -000.789 0.000 1.595 -000.716
L1 PTRY -003.288 0.000 1.595 -002.984
D1 D 077.010 (kN/m³)
Part 4
D1 TRY -001.373 0.405 1.595 (kN,m,m)
L1 TRY -005.720 0.405 1.595 (kN,m,m)
D1 PTRY -000.789 0.000 1.595 -000.716
L1 PTRY -003.288 0.000 1.595 -002.984
D1 D 077.010 (kN/m³)

Member Forces in Load Case 1 and Maximum Deflection from Load Case 3

Span No.	Axial Force (kN)	Shear Force (kN)		Bending Moment (kNm)		Maximum Moment (kNm)	Maximum Deflection (mm @ m)
		End 1	End 2	Node 1	Node 2		
	6	0.000	30.690	0.000	37.580	8.937	
39	0.000	-30.744	0.000	@ 2.356	@ 2.261		

Classification and Effective Area (EN 1993: 2006)

Section (29.99 kg/m)	203x133 UB 30 [S 275]		
Class = $Fn(b/T,d/t,f_y,N,M_y,M_z)$	6.97, 26.94, 275, 0, 37.58, 0	(Axial: Non-Slender)	Class 1
Auto Design Load Cases	1		

Moment Capacity Check M.c.y.Rd

$V_{y.Ed}/V_{pl.y.Rd}$	0.841 / 231.406 =	0.004	Low Shear
$M_{c.y.Rd} = f_y.W_{pl.y}/\gamma_{M0}$	275 x 314.4/1	86.46 kN.m	
$M_{y.Ed}/M_{c.y.Rd}$	37.551 / 86.46 =	0.434	OK

Equivalent Uniform Moment Factor C1

C_1= fn(M_1, M_2, M_o, ~y,~m)	0.0, 0.0, 37.5, 1.000, 300.000	1.127	Uniform

Lateral Buckling Check M.b.Rd

Le = 1.2L+2D	1.2 x 4.5 + 2 x 0.207 =	5.814 m	
M_{cr} = Fn(C_1,L_e,I_z,I_t,I_w,E)	1.127, 5.814, 385.5, 10.3, 0.03734, 210000	56.451 kN.m	
$\lambda_{LT} = \sqrt{W.f_y/M_{cr}}$	$\sqrt{}$ 314.4 x 275 / 56.451	1.238	
c_{LT} = Fn(λ_{LT}, λ_{LT}5950)	1.238, 1.266	0.558	Curve b
c_{LT}.mod = Fn(c_{LT},λ_{LT},k_c,f)	0.558, 1.238, 0.942, 0.982	0.568	6.3.2.3
$M_{b.Rd} = c\ W_{pl.y}.f_y \le M_{c.y.Rd}$	0.568 x 314.4 x 275 ≤ 86.460 =	49.108 kN.m	
$M_{y.Ed}/M_{b.Rd}$	37.578 / 49.108	0.765	OK

Deflection Check - Load Case 3

Deflection Limits (Internal Beams)	In-span δ ≤ 4500/360 = 12.5 mm Live (Case 2)	6.09 mm	OK
	In-span δ ≤ 4500/250 = 18 mm D+L (Case 3)	8.94 mm	OK

Provide Section :- 203 x 133 UB 30 (S275)

Member Loading

Load to B003 (Area loading included)

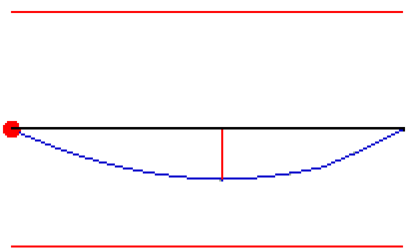
Ground Floor	5.80/2*0.60=1.74	5.80/2*2.5=7.25	
Total	**1.74 kN/m Dead**	**7.25 kN/m Live**	**(Unfactored)**

AXIAL WITH MOMENTS (MEMBER)

Beam 003

Members 46-49 (N.7-N.13) @ Level 1 in Load Case 1

Member Loading and Member Forces
Loading Combination : 1 UT + 1.35 D1 + 1.5 L1



Part 1
D1 TRY -000.769 0.328 1.190 (kN,m,m)
L1 TRY -003.202 0.328 1.190 (kN,m,m)
D1 UDLY -000.986 (kN/m)
L1 UDLY -004.108 (kN/m)
D1 D 077.010 (kN/m³)
Part 2
D1 TRY -000.769 0.328 1.190 (kN,m,m)
L1 TRY -003.202 0.328 1.190 (kN,m,m)
D1 UDLY -000.986 (kN/m)
L1 UDLY -004.108 (kN/m)
D1 D 077.010 (kN/m³)
Part 3
D1 TRY -000.769 0.328 1.190 (kN,m,m)
L1 TRY -003.202 0.328 1.190 (kN,m,m)
D1 UDLY -000.986 (kN/m)
L1 UDLY -004.108 (kN/m)
D1 D 077.010 (kN/m³)
Part 4
D1 TRY -000.769 0.328 1.190 (kN,m,m)
L1 TRY -003.202 0.328 1.190 (kN,m,m)
D1 UDLY -000.986 (kN/m)
L1 UDLY -004.108 (kN/m)
D1 D 077.010 (kN/m³)

Member Forces in Load Case 1 and Maximum Deflection from Load Case 3

Span No.	Axial Force (kN)	Shear Force (kN)		Bending Moment (kNm)		Maximum Moment (kNm)	Maximum Deflection (mm @ m)
		End 1	End 2	Node 1	Node 2		
	7	0.000	30.976	0.000	36.811	7.298	
42	0.000	-37.922	0.000	@ 2.215	@ 2.092		

Classification and Effective Area (EN 1993: 2006)

Section (29.99 kg/m)	203x133 UB 30 [S 275]		
Class = Fn(b/T,d/t,f_y,N,M_y,M_z)	6.97, 26.94, 275, 0, 36.82, 0	(Axial: Non-Slender)	Class 1
Auto Design Load Cases	1		

Moment Capacity Check M.c.y.Rd

$V_{y.Ed}/V_{pl.y.Rd}$	1.539 / 231.406 =	0.007	Low Shear
$M_{c.y.Rd} = f_y.W_{pl.y}/\gamma_{M0}$	275 x 314.4/1	86.46 kN.m	
$M_{y.Ed}/M_{c.y.Rd}$	36.821 / 86.46 =	0.426	OK

Equivalent Uniform Moment Factor C1

C_1= fn(M_1, M_2, M_o, ~y,~m)	0.0, 0.0, 36.4, 0.750, 300.000	1.127	Uniform

Lateral Buckling Check M.b.Rd

Le = 1.2L+2D	1.2 x 4.103 + 2 x 0.207 =	5.337 m	
M_{cr} = Fn(C_1,L_e,I_z,I_t,I_w,E)	1.127, 5.337, 385.5, 10.3, 0.03734, 210000	62.715 kN.m	
$\lambda_{LT} = \sqrt{W.f_y/M_{cr}}$	$\sqrt{314.4 \times 275 / 62.715}$	1.174	
c_{LT} = Fn(λ_{LT}, λ_{LT}5950)	1.174, 1.201	0.594	Curve b
c_{LT}.mod = Fn(c_{LT},λ_{LT},k_c,f)	0.594, 1.174, 0.942, 0.979	0.607	6.3.2.3
$M_{b.Rd} = c\, W_{pl.y}.f_y \le M_{c.y.Rd}$	0.607 x 314.4 x 275 ≤ 86.460 =	52.478 kN.m	

$M_{y.Ed}/M_{b.Rd}$	36.822 / 52.478	0.702	OK

Deflection Check - Load Case 3

Deflection Limits (Internal Beams)	In-span δ ≤ 4103/360 = 11.4 mm Live (Case 2)	4.79 mm	OK
	In-span δ ≤ 4103/250 = 16.4 mm D+L (Case 3)	7.3 mm	OK

Provide Section :- 203 x 133 UB 30 (S275)

Member Loading

Load to B004 (Area loading included)

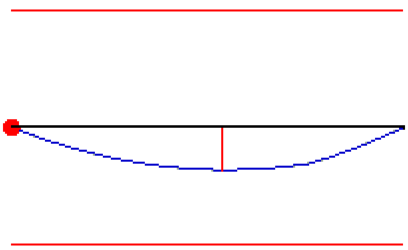
Ground Floor	6.152*0.60=1.85	6.15/2*2.5=7.69	
Total	**1.85 kN/m Dead**	**7.69 kN/m Live**	**(Unfactored)**

AXIAL WITH MOMENTS (MEMBER)

Beam 004

Members 43-44 (N.9-N.33) @ Level 1 in Load Case 1

Member Loading and Member Forces
Loading Combination : 1 UT + 1.35 D1 + 1.5 L1



Part 1
D1 TRY -002.942 0.446 2.915 (kN,m,m)
L1 TRY -012.259 0.446 2.915 (kN,m,m)
D1 UDLY -000.749 (kN/m)
L1 UDLY -003.121 (kN/m)
D1 D 077.010 (kN/m³)
Part 2
D1 TRY -002.942 0.446 2.915 (kN,m,m)
L1 TRY -012.259 0.446 2.915 (kN,m,m)
D1 UDLY -000.749 (kN/m)
L1 UDLY -003.121 (kN/m)
D1 D 077.010 (kN/m³)

Member Forces in Load Case 1 and Maximum Deflection from Load Case 3							
Span No.	Axial Force (kN)	Shear Force (kN)		Bending Moment (kNm)		Maximum Moment (kNm)	Maximum Deflection (mm @ m)
		End 1	End 2	Node 1	Node 2		
	9	0.000	28.180	0.000	31.051	5.307	
44	0.000	-35.707	0.000	@ 2.082	@ 1.933		

Classification and Effective Area (EN 1993: 2006)

Section (29.99 kg/m)	203x133 UB 30 [S 275]		
Class = Fn(b/T,d/t,f_y,N,M_y,M_z)	6.97, 26.94, 275, 0, 31.05, 0	(Axial: Non-Slender)	Class 1
Auto Design Load Cases	1		

Moment Capacity Check M.c.y.Rd

$V_{y.Ed}/V_{pl.y.Rd}$	0.567 / 231.406 =	0.002	Low Shear
$M_{c.y.Rd} = f_y.W_{pl.y}/\gamma_{M0}$	275 x 314.4/1	86.46 kN.m	
$M_{y.Ed}/M_{c.y.Rd}$	31.037 / 86.46 =	0.359	OK

Equivalent Uniform Moment Factor C1

C_1= fn(M_1, M_2, M_o, ~y,~m)	0.0, 0.0, 30.8, 0.714, 300.000	1.127	Uniform

Lateral Buckling Check M.b.Rd

Le = 1.2L+2D	1.2 x 3.801 + 2 x 0.207 =	4.975 m	
M_{cr} = Fn(C_1,L_e,I_z,I_t,I_w,E)	1.127, 4.975, 385.5, 10.3, 0.03734, 210000	68.522 kN.m	
$\lambda_{LT} = \sqrt{W.f_y/M_{cr}}$	√ 314.4 x 275 / 68.522	1.123	
c_{LT} = Fn(λ_{LT}, λ_{LT}5950)	1.123, 1.149	0.625	Curve b
c_{LT}.mod = Fn(c_{LT},λ_{LT},k_c,f)	0.625, 1.123, 0.942, 0.977	0.639	6.3.2.3
$M_{b.Rd} = c\ W_{pl.y}.f_y \le M_{c.y.Rd}$	0.639 x 314.4 x 275 ≤ 86.460 =	55.263 kN.m	
$M_{y.Ed}/M_{b.Rd}$	31.048 / 55.263	0.562	OK

Deflection Check - Load Case 2

Deflection Limits (Internal Beams) In-span $\delta \le 3801/360 = 10.6$ mm Live (Case 2) 3.94 mm OK
In-span $\delta \le 3801/250 = 15.2$ mm D+L (Case 3) 5.31 mm OK

Provide Section :- 203 x 133 UB 30 (S275)

Member Loading

Load to B005 (Area loading included)

Ground Floor	7.30/2*0.60=2.19	7.30/2*2.5=9.13	
Total	**2.19 kN/m Dead**	**9.13 kN/m Live**	**(Unfactored)**

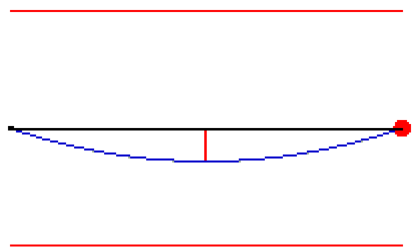
AXIAL WITH MOMENTS (MEMBER)

Beam 005

Member 42 (N.14-N.45) @ Level 1 in Load Case 1

Member Loading and Member Forces
Loading Combination : 1 UT + 1.35 D1 + 1.5 L1

D1 TRY -003.437 0.472 3.392 (kN,m,m)
L1 TRY -014.321 0.472 3.392 (kN,m,m)
D1 TRY -003.687 0.000 3.355 (kN,m,m)
L1 TRY -015.362 0.000 3.355 (kN,m,m)
D1 D 077.010 (kN/m³)



Member Forces in Load Case 1 and Maximum Deflection from Load Case 3							
Span No.	Axial Force (kN)	Shear Force (kN)		Bending Moment (kNm)		Maximum Moment (kNm)	Maximum Deflection (mm @ m)
		End 1	End 2	Node 1	Node 2		
42	14	0.000	-26.935	0.000	-24.266	3.253	
45	0.000	28.554	0.000	@ 1.696	@ 1.696		

Classification and Effective Area (EN 1993: 2006)

Section (29.99 kg/m)	203x133 UB 30 [S 275]		
Class = Fn($b/T, d/t, f_y, N, M_y, M_z$)	6.97, 26.94, 275, 0, 24.27, 0	(Axial: Non-Slender)	Class 1
Auto Design Load Cases	1		

Moment Capacity Check M.c.y.Rd

$V_{y.Ed}/V_{pl.y.Rd}$	0.092 / 231.406 =	0	Low Shear
$M_{c.y.Rd} = f_y.W_{pl.y}/\gamma_{M0}$	275 x 314.4/1	86.46 kN.m	
$M_{y.Ed}/M_{c.y.Rd}$	24.266 / 86.46 =	0.281	OK

Equivalent Uniform Moment Factor C1

C_1= fn(M_1, M_2, M_o, ~y,~m)	0.0, 0.0, 24.2, 0.629, 300.000	1.127	Uniform

Lateral Buckling Check M.b.Rd

Le = 1.2L+2D	1.2 x 3.392 + 2 x 0.207 =	4.484 m	
M_{cr} = Fn(C_1, L_e, I_z, I_t, I_w, E)	1.127, 4.484, 385.5, 10.3, 0.03734, 210000	78.383 kN.m	
$\lambda_{LT} = \sqrt{W.f_y/M_{cr}}$	$\sqrt{314.4 \times 275 / 78.383}$	1.050	
c_{LT} = Fn(λ_{LT}, λ_{LT}5950)	1.050, 1.075	0.669	Curve b
c_{LT}.mod = Fn($c_{LT}, \lambda_{LT}, k_c$, f)	0.669, 1.050, 0.942, 0.975	0.686	6.3.2.3
$M_{b.Rd} = c\,W_{pl.y}.f_y \le M_{c.y.Rd}$	0.686 x 314.4 x 275 ≤ 86.460 =	59.338 kN.m	
$M_{y.Ed}/M_{b.Rd}$	24.266 / 59.338	0.409	OK

Deflection Check - Load Case 2

Deflection Limits (Internal Beams) In-span $\delta \le 3392/360 = 9.4$ mm Live (Case 2) 2.56 mm OK
In-span $\delta \le 3392/250 = 13.6$ mm D+L (Case 3) 3.25 mm OK

Provide Section :- 203 x 133 UB 30 (S275)

Member Loading

Load to B006 (Area loading included)

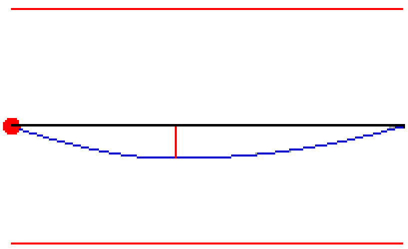
Beam & Block	5.05*2.0/2=5.05	2.50*2.0/2=2.50	
Ground Floor	3.65/2*0.60=1.10	3.65/2*2.5=4.56	
Glass wall	6.0		
Total	**12.15 kN/m Dead**	**7.06 kN/m Live**	**(Unfactored)**

AXIAL WITH MOMENTS (MEMBER)

Beam 006

Members 39-41 (N.15-N.25) @ Level 1 in Load Case 1

Member Loading and Member Forces
Loading Combination : 1 UT + 1.35 D1 + 1.5 L1



Part 1
D1 UDLY -001.089 (kN/m)
L1 UDLY -004.539 (kN/m)
D1 PTY2 -000.308 0.000 0.172 (kN,m,m)
D1 PTRY -003.586 0.172 0.317 -003.535
D1 PTRY -003.535 0.317 0.396 -005.101
D1 PDLY -002.576 0.396 0.901 (kN,m,m)
L1 PTY2 -000.153 0.000 0.172 (kN,m,m)
L1 PTRY -001.775 0.172 0.317 -001.750
L1 PTRY -001.750 0.317 0.396 -002.525
L1 PDLY -001.275 0.396 0.901 (kN,m,m)
D1 D 077.010 (kN/m³)
D1 UDLY -012.800 (kN/m)
Part 2
D1 UDLY -001.089 (kN/m)
L1 UDLY -004.539 (kN/m)
D1 PTY2 -000.308 0.000 0.172 (kN,m,m)
D1 PTRY -003.586 0.172 0.317 -003.535
D1 PTRY -003.535 0.317 0.396 -005.101
D1 PDLY -002.576 0.396 0.901 (kN,m,m)
L1 PTY2 -000.153 0.000 0.172 (kN,m,m)
L1 PTRY -001.775 0.172 0.317 -001.750
L1 PTRY -001.750 0.317 0.396 -002.525
L1 PDLY -001.275 0.396 0.901 (kN,m,m)
D1 D 077.010 (kN/m³)
D1 UDLY -012.800 (kN/m)
Part 3
D1 UDLY -001.089 (kN/m)
L1 UDLY -004.539 (kN/m)
D1 PTY2 -000.308 0.000 0.172 (kN,m,m)
D1 PTRY -003.586 0.172 0.317 -003.535
D1 PTRY -003.535 0.317 0.396 -005.101
D1 PDLY -002.576 0.396 0.901 (kN,m,m)
L1 PTY2 -000.153 0.000 0.172 (kN,m,m)
L1 PTRY -001.775 0.172 0.317 -001.750
L1 PTRY -001.750 0.317 0.396 -002.525
L1 PDLY -001.275 0.396 0.901 (kN,m,m)
D1 D 077.010 (kN/m³)
D1 UDLY -012.800 (kN/m)

Member Forces in Load Case 1 and Maximum Deflection from Load Case 3

Span No.	Axial Force (kN)	Shear Force (kN)		Bending Moment (kNm)		Maximum Moment (kNm)	Maximum Deflection (mm @ m)
		End 1	End 2	Node 1	Node 2		
	15	-0.002	53.466	0.000	36.621	2.447	
46	-0.045	-44.454	0.000	@ 1.283	@ 1.424		

Classification and Effective Area (EN 1993: 2006)

Section (46.1 kg/m) 203x203 UC 46 [S 275]

Class = Fn(b/T,d/t,f_y,N,M_y,M_z)	9.25, 22.33, 275, 0, 36.61, 0	(Axial: Non-Slender)	Class 1
Auto Design Load Cases	1		

Moment Capacity Check M.c.y.Rd

$V_{y.Ed}/V_{pl.y.Rd}$	1.419 / 269.498 =	0.005	Low Shear
$M_{c.y.Rd}$ = $f_y.W_{pl.y}/\gamma_{M0}$	275 x 497.4/1	136.785 kN.m	
$M_{y.Ed}/M_{c.y.Rd}$	36.577 / 136.785 =	0.267	OK

Equivalent Uniform Moment Factor C1

C_1= fn(M_1, M_2, M_o, ~y,~m)	0.0, 0.0, 36.2, 0.918, 300.000	1.127	Uniform

Lateral Buckling Check M.b.Rd

Le = 1.2L+2D	1.2 x 2.92 + 2 x 0.203 =	3.91 m	
M_{cr} = Fn(C_1,L_e,I_z,I_t,I_w,E)	1.127, 3.910, 1551, 22.15, 0.1429, 210000	315.439 kN.m	
λ_{LT} = √ W.f_y/M_{cr}	√ 497.4 x 275 / 315.439	0.659	
c_{LT} = Fn(λ_{LT}, λ_{LT}5950)	0.659, 0.630	0.890	Curve b
c_{LT}.mod = Fn(c_{LT},λ_{LT},k_c,f)	0.890, 0.659, 0.942, 0.972	0.915	6.3.2.3
$M_{b.Rd}$ = c $W_{pl.y}.f_y$ ≤ $M_{c.y.Rd}$	0.915 x 497.4 x 275 ≤ 136.785 =	125.216 kN.m	
$M_{y.Ed}/M_{b.Rd}$	36.616 / 125.216	0.292	OK

Deflection Check - Load Case 3

Deflection Limits (Internal Beams)	In-span δ ≤ 2920/360 = 8.1 mm Live (Case 2)	0.6 mm	OK
	In-span δ ≤ 2920/250 = 11.7 mm D+L (Case 3)	2.45 mm	OK

Provide Section :- 203 x 203 UC 46 (S275)

Member Loading

Load to B007-B013 (Area loading included)

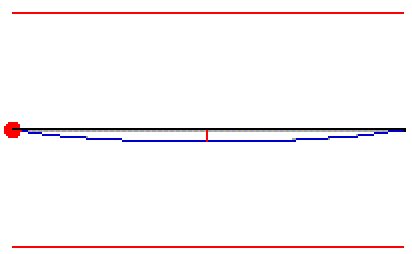
Beam & Block	5.05*2.0/2=5.05	2.50*2.0/2=2.50	
Balustrade	1.00		
Total	**6.05 kN/m Dead**	**2.50 kN/m Live**	**(Unfactored)**

AXIAL WITH MOMENTS (MEMBER)

Beam 007 – Beam 013

Members 1-3 (N.25-N.27) @ Level 1 in Load Case 1

Member Loading and Member Forces
Loading Combination : 1 UT + 1.35 D1 + 1.5 L1



Part 1
D1 UDLY -000.170 (kN/m)
L1 UDLY -000.084 (kN/m)
D1 D 077.010 (kN/m³)
D1 UDLY -004.250 (kN/m)
L1 UDLY -001.500 (kN/m)
Part 2
D1 UDLY -000.170 (kN/m)
L1 UDLY -000.084 (kN/m)
D1 D 077.010 (kN/m³)
D1 UDLY -004.250 (kN/m)
L1 UDLY -001.500 (kN/m)
Part 3
D1 UDLY -000.170 (kN/m)
L1 UDLY -000.084 (kN/m)
D1 D 077.010 (kN/m³)
D1 UDLY -004.250 (kN/m)
L1 UDLY -001.500 (kN/m)

Member Forces in Load Case 1 and Maximum Deflection from Load Case 3							
Span No.	Axial Force (kN)	Shear Force (kN)		Bending Moment (kNm)		Maximum Moment (kNm)	Maximum Deflection (mm @ m)
		End 1	End 2	Node 1	Node 2		
	25	0.009	11.470	0.000	5.015	0.647	

Member Forces in Load Case 1 and Maximum Deflection from Load Case 3							
Span No.	Axial Force (kN)	Shear Force (kN)		Bending Moment (kNm)		Maximum Moment (kNm)	Maximum Deflection (mm @ m)
		End 1	End 2	Node 1	Node 2		
29	-0.007	-11.531	0.000	@ 1.012	@ 1.012		

Classification and Effective Area (EN 1993: 2006)

Section (23.9 kg/m)	150x90 PFC 23.9 [S 275]		
Class = Fn($b/T, d/t, f_y, N, M_y, M_z$)	7.5, 15.69, 275, 0, 5.01, 0	(Axial: Non-Slender)	Class 1
Auto Design Load Cases	1		

Moment Capacity Check M.c.y.Rd

$V_{y.Ed}/V_{pl.y.Rd}$	0.019 / 174.966 =	0	Low Shear
$M_{c.y.Rd} = f_y.W_{pl.y}/\gamma_{M0}$	275 x 179/1	49.225 kN.m	
$M_{y.Ed}/M_{c.y.Rd}$	5.013 / 49.225 =	0.102	OK

Equivalent Uniform Moment Factor C1

C_1= fn(M_1, M_2, M_o, ~y,~m)	0.0, 0.0, 5.0, 0.900, 300.000	1.127	Uniform

Lateral Buckling Check M.b.Rd

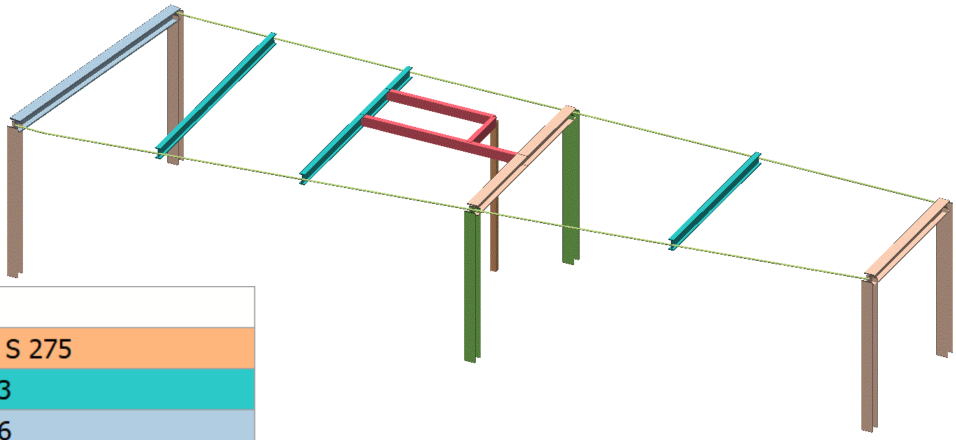
Le = 1.2L+2D	1.2 x 2.021 + 2 x 0.150 =	2.725 m	
M_{cr} = Fn($C_1, L_e, I_z, I_t, I_w, E$)	1.127, 2.725, 253.0, 11.8, 0.008900, 210000	103.800 kN.m	
$\lambda_{LT} = \sqrt{W.f_y/M_{cr}}$	√ 179 x 275 / 103.8	0.689	
c_{LT} = Fn(λ_{LT}, λ_{LT}5950)	0.689, 0.688	0.768	Curve d
c_{LT}.mod = Fn($c_{LT}, \lambda_{LT}, k_c, f$)	0.768, 0.689, 0.942, 0.972	0.791	6.3.2.3
$M_{b.Rd} = c\ W_{pl.y}.f_y \le M_{c.y.Rd}$	0.791 x 179.0 x 275 ≤ 49.225 =	38.913 kN.m	
$M_{y.Ed}/M_{b.Rd}$	5.013 / 38.913	0.129	OK

Deflection Check - Load Case 3

Deflection Limits (Internal Beams)	In-span δ ≤ 2021/360 = 5.6 mm Live (Case 2)	0.15 mm	OK
	In-span δ ≤ 2021/250 = 8.1 mm D+L (Case 3)	0.65 mm	OK

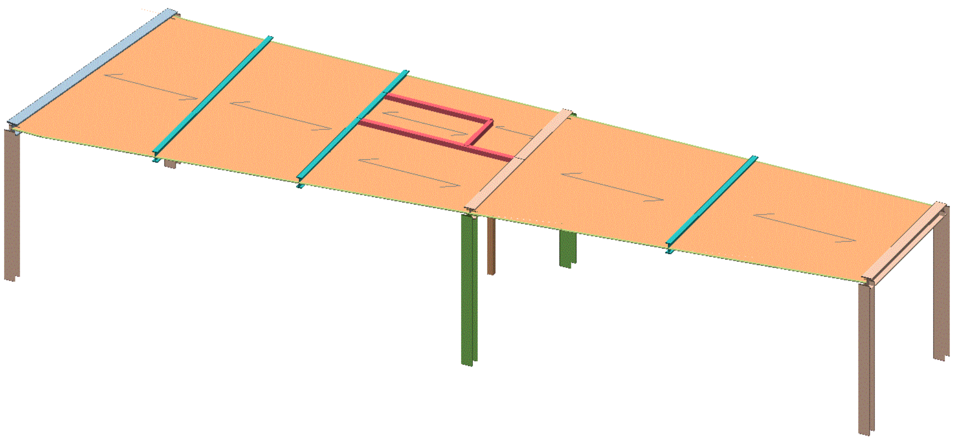
Provide Section :- 150 x 90 PFC 24 (S275) to support skylights & lightwell

Steel Sections at Ground floor structure over



Section Size
Default Grade S 275
203x102 UB 23
203x203 UC 86
203x203 UC 60
203x203 UC 46
100x100 C24
94x175 C24

Area Load Panels at Ground floor structure over



Dead D	Live L
0.600	2.000

Member Loading

Load to B101 (Area loading included)

Solid wall	12.80*3.0/2.7=14.22*2=28.44		
Parapet wall	12.80*1.0/2.7=4.75		
1st Floor	2.60/2*0.60=0.78	2.60/2*2.0=2.60	
2nd Floor	2.60/2*0.60=0.78	2.60/2*2.0=2.60	
3rd Floor	2.60/2*0.60=0.78	2.60/2*2.0=2.60	
Loads from timber wall & mansard roof	2.50	0.61	
Total	**38.03 kN/m Dead**	**8.41 kN/m Live**	**(Unfactored)**

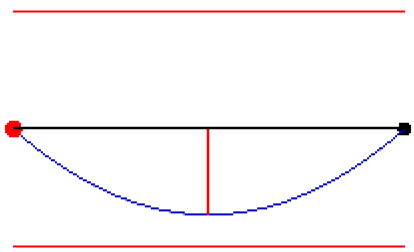
AXIAL WITH MOMENTS (MEMBER)

B101 (Dead & Live Load Case)

Member 18 (N.2-N.6) @ Level 1 in Load Case 1

Member Loading and Member Forces
Loading Combination : 1 UT + 1.35 D1 + 1.5 L1

D1 PTRY -000.217 0.000 0.240 -000.780
D1 PTRY -000.780 0.240 4.795 -000.575
D1 PTRY -000.780 4.795 4.802 -000.575
D1 PTY1 -000.001 4.802 4.833 (kN,m,m)
L1 PTRY -000.725 0.000 0.240 -002.599
L1 PTRY -002.599 0.240 4.795 -001.918
L1 PTRY -002.599 4.795 4.802 -001.918
L1 PTY1 -000.004 4.802 4.833 (kN,m,m)
D1 D 077.010 (kN/m³)
D1 UDLY -037.250 (kN/m)
L1 UDLY -005.810 (kN/m)



Member Forces in Load Case 1 and Maximum Deflection from Load Case 3										
Member No.	Node End 1 End 2	Axial Force (kN)	Torque Moment (kNm)	Shear Force (kN)		Bending Moment (kNm)		Maximum Moment (kNm @ m)		Max Def (mm @ m)
				x-x	y-y	x-x	y-y	x-x	y-y	
18	2	0.01T	0.00	155.91	0.00	0.00	0.00	188.18	0.00	16.76
	6	0.01T	0.00	-155.13	0.00	0.00	0.00	@ 2.389	@ 0.000	@ 2.389

Classification and Effective Area (EN 1993: 2006)

Section (86.06 kg/m)	203x203 UC 86 [S 275]		
Class = Fn(b/T,d/t,f_y,N,M_y,M_z)	5.1, 12.66, 265, 0, 188.19, 0	(Axial: Non-Slender)	Class 1
Auto Design Load Cases	1		

Local Capacity Check

$V_{y.Ed}/V_{pl.y.Rd}$	0.276 / 469.466 =	0.001	Low Shear
$M_{c.y.Rd} = f_y.W_{pl.y}/\gamma_{M0}$	265 x 976.7/1	258.826 kN.m	
$N_{pl.Rd}$ = Ag.fy/γ_{M0}	109.63x265/1 (No bearing / block tearing design)	2905.195 kN	
$n = N_{Ed}/N_{pl.Rd}$	-0.006 / 2905.195 =	0.000	OK
$W_{pl.N.y} = Fn(W_{pl.y}, A_{vy},)$	976.7, 30.685, 0	976.7 cm³	
$M_{N.y.Rd} = W_{pl.N.y}.f_y/\gamma_{M0}$	976.7 x 265/1	258.826 kN.m	
$(M_{y.Ed}/M_{N.y.Rd+(M\_(z.Ed}/M_{N.z.Rd})$	$(188.192/258.826)^2+(0)^1=$	0.529	OK

Equivalent Uniform Moment Factors C1, C.mLT, C.mz, and C.my

C_1= fn(M_1, M_2, M_o, ~y,~m)	0.2, 0.2, 188.0, 0.974, 300.000	1.127	Uniform
C_{mLT}=0.95+0.05α_h	M_h = 0.16, M_s = 188.19, ~y = 0.974, α_s = 0.001	0.95	Table B.3
C_{mz}=Max(0.6+0.4~y, 0.4)	M = 0, ~y = 1.000	1	Table B.3
C_{my}=0.95+0.05α_h	M_h = 0, M_s = 188.19, ~y = 1.000, α_s = 0.000	0.95	Table B.3

Lateral Buckling Check M.b.Rd

Le = 1.2L+2D	1.2 x 4.833 + 2 x 0.222 =	6.244 m	
$M_{cr} = Fn(C_1,L_e,I_z,I_t,I_w,E)$	1.127, 6.244, 3130, 136.8, 0.3177, 210000	518.939 kN.m	
$\lambda_{LT} = \sqrt{W.f_y/M_{cr}}$	√ 976.7 x 265 / 518.939	0.706	
$c_{LT} = Fn(\lambda_{LT}, \lambda_{LT}5950)$	0.706, 0.678	0.867	Curve b
c_{LT}.mod = Fn(c_{LT},λ_{LT},k_c,f)	0.867, 0.706, 0.942, 0.971	0.892	6.3.2.3
$M_{b.Rd} = c\ W_{pl.y}.f_y \le M_{c.y.Rd}$	0.892 x 976.7 x 265 ≤ 258.826 =	230.856 kN.m	

Buckling Resistance

$U_{N.y} = N_{Ed}/(c_y.N_{Rk}/\gamma_{M1})$	0 / 2448.887	0.000	OK
$U_{N.z} = N_{Ed}/(c_z.N_{Rk}/\gamma_{M1})$	0 / 1529.692	0.000	OK
$U_{M.y} = M_{y.Ed}/(c_{LT}.M_{y.Rk}/\gamma_{M1})$	188.192 / 230.856	0.815	OK
$U_{M.z} = M_{z.Ed}/(M_{z.Rk}/\gamma_{M1})$	0 / 120.893	0.000	OK
$k_yy=C_{my}\{1+(\lambda_y-0.2)U_{N.y}\}$		0.950	
$k_zz=C_{mz}\{1+1.4U_{N.z}\}$		1.000	
$k_yz=0.6\ k_zz$		0.600	
$k_zy= 1- \{0.1\lambda_z/(C_{mLT}-0.25)\}U_{N.z}$		1.000	
$U_{Ny}+k_yy.U_{M.y}+k_yz.U_{M.z}$	0.000+0.950x0.815+0.600x0.000	0.774	OK
$U_{Nz}+k_zy.U_{M.y}+k_zz.U_{M.z}$	0.000+1.000x0.815+1.000x0.000	0.815	OK

Deflection Check - Load Case 3

Deflection Limits (Internal Beams)	In-span δ ≤ 4833/360 = 13.4 mm Live (Case 2)	2.89 mm	OK
	In-span δ ≤ 4833/250 = 19.3 mm D+L (Case 3)	16.76 mm	OK

Wind Load to Sway Frame

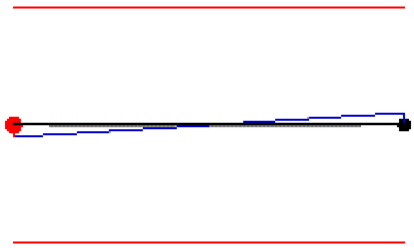
Pw= 0.60*[(7.7/2*2.7/2) + (4.3/2*2.7) + (2.8/2*2.7) + (3.70/2*2.4)]=11.55kN

AXIAL WITH MOMENTS (MEMBER)

B101 (Wind Load Case)

Member 1 (N.1-N.2) @ Level 1 in Load Case 4

Member Loading and Member Forces
Loading Combination : 1 UT + 1.5 W1



Member Forces in Load Case 4 and Maximum Deflection from Load Case 5							
Span No.	Axial Force (kN)	Shear Force (kN)		Bending Moment (kNm)		Maximum Moment (kNm)	Maximum Deflection (mm @ m)
		End 1	End 2	Node 1	Node 2		
1	1	8.658C	-10.717	26.001		0.330	
2	8.658C	-10.717	-25.974		@ 2.425		

Classification and Effective Area (EN 1993: 2006)

Section (86.06 kg/m)	203x203 UC 86 [S 275]		
Class = Fn($b/T,d/t,f_y,N,M_y,M_z$)	5.1, 12.66, 265, 8.66, 26, 0	(Axial: Non-Slender)	Class 1
Auto Design Load Cases	1 & 4		

Local Capacity Check

$V_{y.Ed}/V_{pl.y.Rd}$	10.716 / 469.466 =	0.023	Low Shear
$M_{c.y.Rd} = f_y.W_{pl.y}/\ \gamma_{M0}$	265 x 976.7/1	258.826 kN.m	
$N_{pl.Rd} = A_g.f_y/\ \gamma_{M0}$	109.63 x 265/1 =	2905.195 kN	
$n = N_{Ed}/N_{pl.Rd}$	8.658 / 2905.195 =	0.003	OK
$W_{pl.N.y} = Fn(W_{pl.y}, A_{vy},)$	976.7, 30.685, 0.003	976.7 cm³	
$M_{N.y.Rd} = W_{pl.N.y}.f_y/\ \gamma_{M0}$	976.7 x 265/1	258.826 kN.m	
$(M_{y.Ed}/M_{N.y.Rd+(M\_(z.Ed}/M_{N.z.Rd})$	$(26.001/258.826)^2+(0)^1=$	0.01	OK

Compression Resistance N.b.Rd

$\lambda y = \sqrt{A.fy/Ncr}$	√109.63x265/8328.09	0.59	
$N_{b.y.Rd}$ = Area.c.$f_y/\ \gamma_{M1}$	109.63x0.842x265/10/1 =	2445.842 kN	Curve b
$\lambda z = \sqrt{A.fy/Ncrz}$	√109.63x265/2758.17	1.027	
$N_{b.z.Rd}$ = Area.c.$f_y/\ \gamma_{M1}$	109.63x0.524x265/10/1 =	1523.602 kN	Curve c
Let = Kt.Lx	1x4.85 =	4.85	
$\lambda T = \sqrt{A.fy/NcrT}$	√109.63x265/12061.78	0.491	
$N_{b.T.Rd}$= Area.c.fy/ γ_{M1}	109.63x0.848x265/10/1 =	2463.997 kN	Curve c

Equivalent Uniform Moment Factor C1

C_1= fn(M_1, M_2, ~y)	26.0, -26.0, -0.999	2.550	Not Loaded

C_{mLT}=Max(0.6+0.4~y, 0.4)	M = 25.99, ~y = -0.999	0.4	Table B.3
C_{mz}=Max(0.6+0.4~y, 0.4)	M = 0, ~y = 1.000	1	Table B.3
C_{my}=Max(0.2+0.8α_s, 0.4)	M_h = 26, M_s = 0.01, ~y = -0.999, α_s = 0.000	0.4	Table B.3

Lateral Buckling Check M.b.Rd

Le = 1.2L+2D	1.2 x 4.85 + 2 x 0.222 =	6.264 m	
M_{cr} = Fn(C_1,L_e,I_z,I_t,I_w,E)	2.550, 6.264, 3130, 136.8, 0.3177, 210000	1169.747 kN.m	
λ_{LT} = √ W.f_y/M_{cr}	√ 976.7 x 265 / 1169.747	0.470	
c_{LT} = Fn(λ_{LT}, λ_{LT}5950)	0.470, 0.679	0.972	Curve b
c_{LT}.mod = Fn(c_{LT},λ_{LT},k_c,f)	0.972, 0.470, 0.626, 0.854	1.000	6.3.2.3
$M_{b.Rd}$ = c $W_{pl.y}$.f_y ≤ $M_{c.y.Rd}$	1.000 x 976.7 x 265 ≤ 258.826 =	258.826 kN.m	

Buckling Resistance

$U_{N.y}$ = N_{Ed}/(c_y.N_{Rk}/γ_{M1})	8.658 / 2445.842	0.004	OK
$U_{N.z}$ = N_{Ed}/(c_z.N_{Rk}/γ_{M1})	8.658 / 1523.602	0.006	OK
$U_{M.y}$ = $M_{y.Ed}$/(c_{LT}.$M_{y.Rk}$/γ_{M1})	26.001 / 258.826	0.100	OK
$U_{M.z}$ = $M_{z.Ed}$/($M_{z.Rk}$/γ_{M1})	0 / 120.893	0.000	OK
k_yy=C_{my}{1+(λ_y-0.2)$U_{N.y}$}		0.401	
k_zz=C_{mz}{1+1.4$U_{N.z}$}		1.008	
k_yz=0.6 k_zz		0.605	
k_zy=0.6 k_yy		0.240	
U_{Ny}+k_yy.$U_{M.y}$+k_yz.$U_{M.z}$	0.004+0.401x0.100+0.605x0.000	0.044	OK
U_{Nz}+k_zy.$U_{M.y}$+k_zz.$U_{M.z}$	0.006+0.240x0.100+1.008x0.000	0.030	OK

Deflection Check - Load Case 3

Deflection Limits (Internal Beams)	In-span δ ≤ 4850/250 = 19.4 mm D+L (Case 3)	0.18 mm	OK

Provide Section :- 203 x 203 UC 86 (S275) to support solid walls over & act as sway frame for lateral loads

Member Loading

Load to B102 (Area loading included)

1st Floor	5.32/2*0.60=1.60	5.32/2*2.0=5.32	
Total	**1.60 kN/m Dead**	**5.32 kN/m Live**	**(Unfactored)**

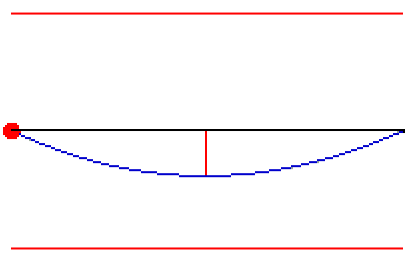
AXIAL WITH MOMENTS (MEMBER)

Beam 102

Member 20 (N.10-N.23) @ Level 1 in Load Case 1

Member Loading and Member Forces
Loading Combination : 1 UT + 1.35 D1 + 1.5 L1

D1 PTRY -000.789 0.000 4.503 -000.582
L1 PTRY -002.630 0.000 4.503 -001.941
D1 TRY -003.451 0.325 4.503 (kN,m,m)
L1 TRY -011.502 0.325 4.503 (kN,m,m)
D1 D 077.010 (kN/m³)



Member Forces in Load Case 1 and Maximum Deflection from Load Case 3										
Member No.	Node End 1 End 2	Axial Force (kN)	Torque Moment (kNm)	Shear Force (kN)		Bending Moment (kNm)		Maximum Moment (kNm @ m)		Max Def (mm @ m)
				x-x	y-y	x-x	y-y	x-x	y-y	
20	10	0.00C	0.00	21.55	0.00	0.00	0.00	24.56		8.02
	23	0.00C	0.00	-21.34	0.00	0.00	0.00	@ 2.228		@ 2.228

Classification and Effective Area (EN 1993: 2006)

Section (23.07 kg/m)	203x102 UB 23 [S 275]		
Class = Fn(b/T,d/t,f_y,N,M_y,M_z)	5.47, 31.37, 275, 0, 24.56, 0	(Axial: Non-Slender)	Class 1
Auto Design Load Cases	1		

Moment Capacity Check M.c.y.Rd

$V_{y.Ed}/V_{pl.y.Rd}$	0.225 / 196.416 =	0.001	Low Shear
$M_{c.y.Rd} = f_y.W_{pl.y}/\gamma_{M0}$	275 x 234.1/1	64.378 kN.m	
$M_{y.Ed}/M_{c.y.Rd}$	24.56 / 64.378 =	0.381	OK

Equivalent Uniform Moment Factor C1

C_1= fn(M_1, M_2, M_o, ~y,~m)	0.0, 0.0, 24.5, 0.395, 300.000	1.127	Uniform

Lateral Buckling Check M.b.Rd

Le = 1.2L+2D	1.2 x 4.503 + 2 x 0.203 =	5.81 m	
M_{cr} = Fn(C_1,L_e,I_z,I_t,I_w,E)	1.127, 5.810, 164.8, 7.019, 0.01537, 210000	29.154 kN.m	
$\lambda_{LT} = \sqrt{W.f_y/M_{cr}}$	√ 234.1 x 275 / 29.154	1.486	
c_{LT} = Fn(λ_{LT}, λ_{LT}5950)	1.486, 1.548	0.433	Curve b
c_{LT}.mod = Fn(c_{LT},λ_{LT},k_c,f)	0.433, 1.486, 0.942, 0.998	0.434	6.3.2.3
$M_{b.Rd} = c\ W_{pl.y}.f_y \le M_{c.y.Rd}$	0.434 x 234.1 x 275 ≤ 64.378 =	27.944 kN.m	
$M_{y.Ed}/M_{b.Rd}$	24.562 / 27.944	0.879	OK

Deflection Check - Load Case 2

Deflection Limits (Internal Beams)	In-span δ ≤ 4503/360 = 12.5 mm Live (Case 2)	5.96 mm	OK
	In-span δ ≤ 4503/250 = 18 mm D+L (Case 3)	8.02 mm	OK

Provide Section :- **203 x 102 UB 23 (S275)**
Bearing :- **Beam to extend to rear of 330mm long x 102mm wide 3 Course Class B Engineering Brick Padstone.**

Member Loading

Load to B103 (Area loading included)

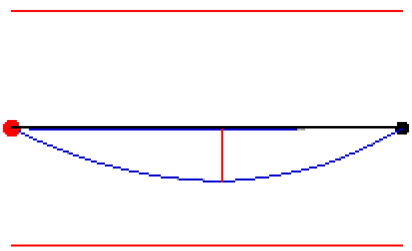
Timber wall	1.01*2.7/2.4=1.14		
1st Floor	5.80/2*0.60=1.74	5.80/2*2.0=5.80	
Total	**2.88 kN/m Dead**	**5.80 kN/m Live**	**(Unfactored)**

AXIAL WITH MOMENTS (MEMBER)

Beam 103

Members 25-27 (N.11-N.15) @ Level 1 in Load Case 1

Member Loading and Member Forces
Loading Combination : 1 UT + 1.35 D1 + 1.5 L1



Part 1
D1 UDLY -000.795 (kN/m)
L1 UDLY -002.650 (kN/m)
D1 TRY -002.002 0.383 2.322 (kN,m,m)
L1 TRY -006.675 0.383 2.322 (kN,m,m)
D1 D 077.010 (kN/m³)
Part 2
D1 UDLY -000.795 (kN/m)
L1 UDLY -002.650 (kN/m)
D1 TRY -002.002 0.383 2.322 (kN,m,m)
L1 TRY -006.675 0.383 2.322 (kN,m,m)
D1 D 077.010 (kN/m³)
D1 UDLY -001.140 (kN/m)
Part 3
D1 UDLY -000.795 (kN/m)
L1 UDLY -002.650 (kN/m)
D1 TRY -002.002 0.383 2.322 (kN,m,m)
L1 TRY -006.675 0.383 2.322 (kN,m,m)
D1 D 077.010 (kN/m³)
D1 UDLY -001.140 (kN/m)

Member Forces in Load Case 1 and Maximum Deflection from Load Case 3										
Member No.	Node End 1 End 2	Axial Force (kN)	Torque Moment (kNm)	Shear Force (kN)		Bending Moment (kNm)		Maximum Moment (kNm @ m)		Max Def (mm @ m)
				x-x	y-y	x-x	y-y	x-x	y-y	
	11	0.00C	0.00	24.44	0.01	0.00	0.00	28.75	0.03	8.08
	24	0.00T	0.00	-28.93	0.00	0.00	0.00	@ 2.251	@ 2.322	@ 2.109

Classification and Effective Area (EN 1993: 2006)

Section (23.07 kg/m)	203x102 UB 23 [S 275]		
Class = Fn($b/T, d/t, f_y, N, M_y, M_z$)	5.47, 31.37, 275, 0, 28.75, 0.03	(Axial: Non-Slender)	Class 1
Auto Design Load Cases	1		

Local Capacity Check

$V_{y.Ed}/V_{pl.y.Rd}$	0.084 / 196.416 =	0	Low Shear
$M_{c.y.Rd} = f_y.W_{pl.y}/\gamma_{M0}$	275 x 234.1/1	64.378 kN.m	
$V_{z.Ed}/V_{pl.z.Rd}$	0.012 / 300.63 =	0	Low Shear
$M_{c.z.Rd} = f_y.W_{pl.z}/\gamma_{M0}$	275 x 49.8/1	13.695 kN.m	
$N_{pl.Rd} = Ag.fy/\gamma_{M0}$	29.39x275/1 (No bearing / block tearing design)	808.225 kN	
$n = N_{Ed}/N_{pl.Rd}$	-0.003 / 808.225 =	0.000	OK
$W_{pl.N.y} = Fn(W_{pl.y}, A_{vy},)$	234.1, 12.371, 0	234.1 cm³	
$M_{N.y.Rd} = W_{pl.N.y}.f_y/\gamma_{M0}$	234.1 x 275/1	64.378 kN.m	
$W_{pl.N.z} = Fn(W_{pl.z}, A_{vz},)$	49.8, 18.935, 0	49.8 cm³	
$M_{N.z.Rd} = W_{pl.N.z}.f_y/\gamma_{M0}$	49.8 x 275/1	13.695 kN.m	
($M_{y.Ed}$/M_(N.y.Rd+($M_{z.Ed}/M_{N.z.Rd}$)	$(28.753/64.378)^2+(0.029/13.695)^1$=	0.202	OK

Compression Resistance N.b.Rd

$\lambda y = \sqrt{A.fy/Ncr}$	$\sqrt{29.39x275/2495.56}$	0.569	
$N_{b.y.Rd}$ = Area.c.f_y/γ_{M1}	29.39x0.901x275/10/1 =	728.394 kN	Curve a
$\lambda z = \sqrt{A.fy/Ncrz}$	$\sqrt{29.39x275/195.3}$	2.033	
$N_{b.z.Rd}$ = Area.c.f_y/γ_{M1}	29.39x0.204x275/10/1 =	164.489 kN	Curve b
Let = Kt.Lx	1x4.182 =	4.182	
$\lambda T = \sqrt{A.fy/NcrT}$	$\sqrt{29.39x275/970.48}$	0.913	
$N_{b.T.Rd}$= Area.c.fy/ γ_{M1}	29.39x0.653x275/10/1 =	527.842 kN	Curve b

Equivalent Uniform Moment Factors C1, C.mLT, C.mz, and C.my

C_1= fn(M_1, M_2, M_o, ~y,~m)	0.0, 0.0, 28.6, 0.778, 300.000	1.127	Uniform
$C_{mLT}=0.95+0.05\alpha_h$	M_h = 0.03, M_s = 28.59, ~y = 0.778, α_s = 0.001	0.95	Table B.3
C_{mz}=Max(0.6+0.4~y, 0.4)	M = 0, ~y = 1.000	1	Table B.3
$C_{my}=0.95+0.05\alpha_h$	M_h = 0, M_s = 28.59, ~y = 1.000, α_s = 0.000	0.95	Table B.3

Lateral Buckling Check M.b.Rd

Le = 1.2L+2D	1.2 x 4.182 + 2 x 0.203 =	5.425 m	
M_{cr} = Fn(C_1,L_e,I_z,I_t,I_w,E)	1.127, 5.425, 164.8, 7.019, 0.01537, 210000	31.550 kN.m	
$\lambda_{LT} = \sqrt{W.f_y/M_{cr}}$	$\sqrt{234.1 \times 275 / 31.55}$	1.428	
c_{LT} = Fn(λ_{LT}, λ_{LT}5950)	1.428, 1.488	0.459	Curve b
c_{LT}.mod = Fn(c_{LT},λ_{LT},k_c,f)	0.459, 1.428, 0.942, 0.994	0.462	6.3.2.3
$M_{b.Rd} = c\ W_{pl.y}.f_y \leq M_{c.y.Rd}$	0.462 x 234.1 x 275 ≤ 64.378 =	29.751 kN.m	

Buckling Resistance

$U_{N.y} = N_{Ed}/(c_y.N_{Rk}/\gamma_{M1})$	0.003 / 728.394	0.000	OK
$U_{N.z} = N_{Ed}/(c_z.N_{Rk}/\gamma_{M1})$	0.003 / 164.489	0.000	OK
$U_{M.y} = M_{y.Ed}/(c_{LT}.M_{y.Rk}/\gamma_{M1})$	28.753 / 29.751	0.966	OK
$U_{M.z} = M_{z.Ed}/(M_{z.Rk}/\gamma_{M1})$	0.029 / 13.695	0.002	OK
$k_y y=C_{my}\{1+(\lambda_y-0.2)U_{N.y}\}$		0.950	
$k_z z=C_{mz}\{1+1.4U_{N.z}\}$		1.000	
$k_y z=0.6\ k_z z$		0.600	
$k_z y$= 1- $\{0.1\lambda_z/(C_{mLT}-0.25)\}U_{N.z}$		1.000	
$U_{Ny}+k_y y.U_{M.y}+k_y z.U_{M.z}$	0.000+0.950x0.966+0.600x0.002	0.919	OK
$U_{Nz}+k_z y.U_{M.y}+k_z z.U_{M.z}$	0.000+1.000x0.966+1.000x0.002	0.969	OK

Deflection Check - Load Case 2

Deflection Limits (Internal Beams)	In-span δ ≤ 4182/360 = 11.6 mm Live (Case 2)	5.71 mm	OK
	In-span δ ≤ 4182/250 = 16.7 mm D+L (Case 3)	8.08 mm	OK

Provide Section :- 203 x 102 UB 23 (S275)

Bearing :- Beam to extend to rear of 440mm long x 102mm wide 3 Course Class B Engineering Brick Padstone.

Member Loading

Load to B104 (Area loading included)

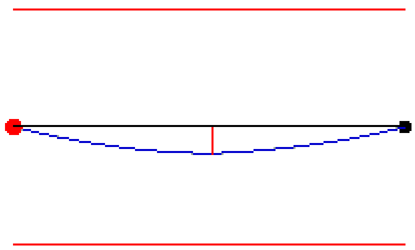
Timber wall	1.01*2.7/2.4=1.14		
1st Floor	6.80/2*0.60=2.04	6.80/2*2.0=6.80	
Total	**3.18 kN/m Dead**	**6.80 kN/m Live**	**(Unfactored)**

AXIAL WITH MOMENTS (MEMBER)

B104 (Dead & Live Load Case)

Members 21-22 (N.12-N.19) @ Level 1 in Load Case 1

Member Loading and Member Forces
Loading Combination : 1 UT + 1.35 D1 + 1.5 L1



Part 1
D1 TRY -001.866 0.463 1.939 (kN,m,m)
L1 TRY -006.220 0.463 1.939 (kN,m,m)
D1 UDLY -000.940 (kN/m)
L1 UDLY -003.133 (kN/m)
D1 D 077.010 (kN/m³)
D1 UDLY -001.140 (kN/m)
Part 2
D1 TRY -001.866 0.463 1.939 (kN,m,m)
L1 TRY -006.220 0.463 1.939 (kN,m,m)
D1 UDLY -000.940 (kN/m)
L1 UDLY -003.133 (kN/m)
D1 D 077.010 (kN/m³)
D1 UDLY -001.140 (kN/m)

Member Forces in Load Case 1 and Maximum Deflection from Load Case 3

Member No.	Node End 1 End 2	Axial Force (kN)	Torque Moment (kNm)	Shear Force (kN)		Bending Moment (kNm)		Maximum Moment (kNm @ m)		Max Def (mm @ m)
				x-x	y-y	x-x	y-y	x-x	y-y	
	12	0.01C	0.00	29.05	-0.03	0.00	0.00	30.89	-0.06	3.26
	25	0.00T	0.00	-29.52	0.03	0.00	0.00	@ 1.939	@ 1.939	@ 1.899

Classification and Effective Area (EN 1993: 2006)

Section (46.1 kg/m)	203x203 UC 46 [S 275]		
Class = Fn(b/T,d/t,f_y,N,M_y,M_z)	9.25, 22.33, 275, 0.01, 30.89, 0.07	(Axial: Non-Slender)	Class 1
Auto Design Load Cases	1		

Local Capacity Check

$V_{y.Ed}/V_{pl.y.Rd}$	5.091 / 269.498 =	0.019	Low Shear
$M_{c.y.Rd} = f_y.W_{pl.y}/\gamma_{M0}$	275 x 497.4/1	136.785 kN.m	
$V_{z.Ed}/V_{pl.z.Rd}$	0.034 / 711.169 =	0	Low Shear
$M_{c.z.Rd} = f_y.W_{pl.z}/\gamma_{M0}$	275 x 230.9/1	63.498 kN.m	
$N_{pl.Rd} = Ag.fy/\gamma_{M0}$	58.73x275/1 (No bearing / block tearing design)	1615.075 kN	
$n = N_{Ed}/N_{pl.Rd}$	-0.004 / 1615.075 =	0.000	OK
$W_{pl.N.y} = Fn(W_{pl.y}, A_{vy},)$	497.4, 16.974, 0	497.4 cm³	
$M_{N.y.Rd} = W_{pl.N.y}.f_y/\gamma_{M0}$	497.4 x 275/1	136.785 kN.m	
$W_{pl.N.z} = Fn(W_{pl.z}, A_{vz},)$	230.9, 44.792, 0	230.9 cm³	
$M_{N.z.Rd} = W_{pl.N.z}.f_y/\gamma_{M0}$	230.9 x 275/1	63.498 kN.m	
$(M_{y.Ed}/M\_(N.y.Rd+(M_{z.Ed}/M_{N.z.Rd})$	$(30.894/136.785)^2+(0.065/63.498)^1=$	0.052	OK

Compression Resistance N.b.Rd

$\lambda y = \sqrt{A.fy/Ncr}$	$\sqrt{58.73x275/6474.99}$	0.5	
$N_{b.y.Rd}$ = Area.c.f_y/γ_{M1}	58.73x0.884x275/10/1 =	1428.404 kN	Curve b
$\lambda z = \sqrt{A.fy/Ncrz}$	$\sqrt{58.73x275/2197.33}$	0.857	
$N_{b.z.Rd}$ = Area.c.f_y/γ_{M1}	58.73x0.626x275/10/1 =	1011.588 kN	Curve c
Let = Kt.Lx	1x3.825 =	3.825	
$\lambda T = \sqrt{A.fy/NcrT}$	$\sqrt{58.73x275/3659.52}$	0.664	
$N_{b.T.Rd}$= Area.c.fy/ γ_{M1}	58.73x0.747x275/10/1 =	1205.874 kN	Curve c

Equivalent Uniform Moment Factors C1, C.mLT, C.mz, and C.my

C_1= fn(M_1, M_2, M_o, ~y,~m)	0.0, 0.0, 30.8, 0.839, 300.000	1.127	Uniform
$C_{mLT}=0.95+0.05\alpha_h$	M_h = 0.03, M_s = 30.85, ~y = 0.839, α_s = 0.001	0.95	Table B.3

C_{mz}=Max(0.6+0.4~y, 0.4)	M = 0, ~y = 1.000	1	Table B.3
C_{my}=0.95+0.05α_h	M_h = 0, M_s = 30.85, ~y = 1.000, α_s = 0.000	0.95	Table B.3

Lateral Buckling Check M.b.Rd

Le = 1.2L+2D	1.2 x 3.825 + 2 x 0.203 =	4.996 m	
M_{cr} = Fn(C_1,L_e,I_z,I_t,I_w,E)	1.127, 4.996, 1551, 22.15, 0.1429, 210000	220.619 kN.m	
λ_{LT} = √ W.f_y/M_{cr}	√ 497.4 x 275 / 220.619	0.787	
c_{LT} = Fn(λ_{LT}, λ_{LT}5950)	0.787, 0.754	0.824	Curve b
c_{LT}.mod = Fn(c_{LT},λ_{LT},k_c,f)	0.824, 0.787, 0.942, 0.971	0.849	6.3.2.3
$M_{b.Rd}$ = c $W_{pl.y}$.f_y ≤ $M_{c.y.Rd}$	0.849 x 497.4 x 275 ≤ 136.785 =	116.082 kN.m	

Buckling Resistance

$U_{N.y}$ = N_{Ed}/(c_y.N_{Rk}/γ_{M1})	0.008 / 1428.404	0.000	OK
$U_{N.z}$ = N_{Ed}/(c_z.N_{Rk}/γ_{M1})	0.008 / 1011.588	0.000	OK
$U_{M.y}$ = $M_{y.Ed}$/(c_{LT}.$M_{y.Rk}$/γ_{M1})	30.894 / 116.082	0.266	OK
$U_{M.z}$ = $M_{z.Ed}$/($M_{z.Rk}$/γ_{M1})	0.065 / 63.498	0.001	OK
k_yy=C_{my}{1+(λ_y-0.2)$U_{N.y}$}		0.950	
k_zz=C_{mz}{1+(2λ_z-0.6)$U_{N.z}$}		1.000	
k_yz=0.6 k_zz		0.600	
k_zy= 1- {0.1/(C_{mLT}-0.25)}$U_{N.z}$		1.000	
U_{Ny}+k_yy.$U_{M.y}$+k_yz.$U_{M.z}$	0.000+0.950x0.266+0.600x0.001	0.253	OK
U_{Nz}+k_zy.$U_{M.y}$+k_zz.$U_{M.z}$	0.000+1.000x0.266+1.000x0.001	0.267	OK

Deflection Check - Load Case 3

Deflection Limits (Internal Beams)	In-span δ ≤ 3825/360 = 10.6 mm Live (Case 2)	2.16 mm	OK
	In-span δ ≤ 3825/250 = 15.3 mm D+L (Case 3)	3.26 mm	OK

Wind Load to Sway Frame

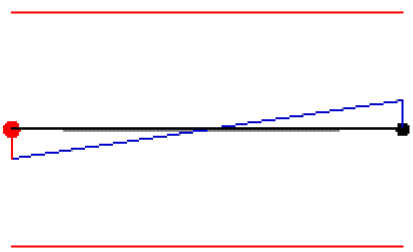
Pw= 0.52*[(15/2*2.7/2) + (4.6/2*2.7) + (8.80/2*2.7)]=14.67kN

AXIAL WITH MOMENTS (MEMBER)

Beam 104 (Wind Load Case)

Member 1 (N.1-N.2) @ Level 1 in Load Case 4

Member Loading and Member Forces
Loading Combination : 1 UT + 1.5 W1



Member Forces in Load Case 4 and Maximum Deflection from Load Case 5							
Span No.	Axial Force (kN)	Shear Force (kN)		Bending Moment (kNm)		Maximum Moment (kNm)	Maximum Deflection (mm @ m)
		End 1	End 2	Node 1	Node 2		
1	1	11.243C	-16.875	33.771		0.603	
2	11.243C	-16.875	-33.729		@ 2.000		

Classification and Effective Area (EN 1993: 2006)

Section (46.1 kg/m)	203x203 UC 46 [S 275]		
Class = Fn(b/T,d/t,f_y,N,M_y,M_z)	9.25, 22.33, 275, 11.24, 33.77, 0	(Axial: Non-Slender)	Class 1
Auto Design Load Cases	1 & 4		

Local Capacity Check

$V_{y.Ed}$/$V_{pl.y.Rd}$	16.875 / 269.498 =	0.063	Low Shear
$M_{c.y.Rd}$ = f_y.$W_{pl.y}$/ γ_{M0}	275 x 497.4/1	136.785 kN.m	
$N_{pl.Rd}$ = A_g.f_y/ γ_{M0}	58.73 x 275/1 =	1615.075 kN	
n = N_{Ed}/$N_{pl.Rd}$	11.243 / 1615.075 =	0.007	OK
$W_{pl.N.y}$ = Fn($W_{pl.y}$, A_{vy},)	497.4, 16.974, 0.007	497.4 cm^3	
$M_{N.y.Rd}$ = $W_{pl.N.y}$.f_y/ γ_{M0}	497.4 x 275/1	136.785 kN.m	
($M_{y.Ed}$/$M_{N.y.Rd}$+(M_(z.Ed/$M_{N.z.Rd}$)	$(33.771/136.785)^2+(0)^1$=	0.061	OK

Compression Resistance N.b.Rd

$\lambda y = \sqrt{}A.fy/Ncr$	√58.73x275/5920.82	0.522	
$N_{b.y.Rd}$ = Area.c.f_y/ γ_{M1}	58.73x0.874x275/10/1 =	1411.834 kN	Curve b
$\lambda z = \sqrt{}A.fy/Ncrz$	√58.73x275/2009.27	0.896	
$N_{b.z.Rd}$ = Area.c.f_y/ γ_{M1}	58.73x0.602x275/10/1 =	972.343 kN	Curve c
Let = Kt.Lx	1x4 =	4	
$\lambda T = \sqrt{}A.fy/NcrT$	√58.73x275/3493.27	0.68	
$N_{b.T.Rd}$= Area.c.fy/ γ_{M1}	58.73x0.737x275/10/1 =	1190.403 kN	Curve c

Equivalent Uniform Moment Factor C1

C_1= fn(M_1, M_2, ~y)	33.8, -33.7, -0.999	2.550	Not Loaded
C_{mLT}=Max(0.6+0.4~y, 0.4)	M = 33.75, ~y = -0.999	0.4	Table B.3
C_{mz}=Max(0.6+0.4~y, 0.4)	M = 0, ~y = 1.000	1	Table B.3
C_{my}=Max(0.2+0.8α_s, 0.4)	M_h = 33.77, M_s = 0.02, ~y = -0.999, α_s = 0.001	0.4	Table B.3

Lateral Buckling Check M.b.Rd

Le = 1.2L+2D	1.2 x 4 + 2 x 0.203 =	5.206 m	
M_{cr} = Fn(C_1,L_e,I_z,I_t,I_w,E)	2.550, 5.206, 1551, 22.15, 0.1429, 210000	471.440 kN.m	
$\lambda_{LT} = \sqrt{}$ W.f_y/M_{cr}	√ 497.4 x 275 / 471.44	0.539	
c_{LT} = Fn(λ_{LT}, λ_{LT}5950)	0.539, 0.776	0.944	Curve b
c_{LT}.mod = Fn(c_{LT},λ_{LT},k_c,f)	0.944, 0.539, 0.626, 0.839	1.000	6.3.2.3
$M_{b.Rd}$ = c $W_{pl.y}.f_y \leq M_{c.y.Rd}$	1.000 x 497.4 x 275 ≤ 136.785 =	136.785 kN.m	

Buckling Resistance

$U_{N.y} = N_{Ed}/(c_y.N_{Rk}/\gamma_{M1})$	11.243 / 1411.834	0.008	OK
$U_{N.z} = N_{Ed}/(c_z.N_{Rk}/\gamma_{M1})$	11.243 / 972.343	0.012	OK
$U_{M.y} = M_{y.Ed}/(c_{LT}.M_{y.Rk}/\gamma_{M1})$	33.771 / 136.785	0.247	OK
$U_{M.z} = M_{z.Ed}/(M_{z.Rk}/\gamma_{M1})$	0 / 63.498	0.000	OK
$k_yy=C_{my}\{1+(\lambda_y-0.2)U_{N.y}\}$		0.401	
$k_zz=C_{mz}\{1+(2\lambda_z-0.6)U_{N.z}\}$		1.014	
k_yz=0.6 k_zz		0.608	
k_zy=0.6 k_yy		0.241	
$U_{Ny}+k_yy.U_{M.y}+k_yz.U_{M.z}$	0.008+0.401x0.247+0.608x0.000	0.107	OK
$U_{Nz}+k_zy.U_{M.y}+k_zz.U_{M.z}$	0.012+0.241x0.247+1.014x0.000	0.071	OK

Deflection Check - Load Case 3

Deflection Limits (Internal Beams)	In-span δ ≤ 4000/250 = 16 mm D+L (Case 3)	0.07 mm	OK

Provide Section :- 203 x 203 UC 46 (S275)

Member Loading

Load to B105 (Area loading included)

1st Floor	7.30/2*0.60=2.19	7.30/2*2.0=7.30	
Total	**2.19 kN/m Dead**	**7.30 kN/m Live**	**(Unfactored)**

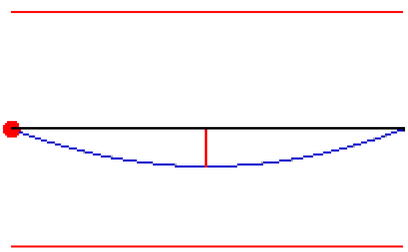
AXIAL WITH MOMENTS (MEMBER)

Beam 105

Member 23 (N.14-N.26) @ Level 1 in Load Case 1

Member Loading and Member Forces
Loading Combination : 1 UT + 1.35 D1 + 1.5 L1

D1 TRY -003.691 0.000 3.362 (kN,m,m)
L1 TRY -012.304 0.000 3.362 (kN,m,m)
D1 TRY -003.431 0.482 3.393 (kN,m,m)
L1 TRY -011.435 0.482 3.393 (kN,m,m)
D1 D 077.010 (kN/m^3)



Member Forces in Load Case 1 and Maximum Deflection from Load Case 3										
Member No.	Node End 1 End 2	Axial Force (kN)	Torque Moment (kNm)	Shear Force (kN)		Bending Moment (kNm)		Maximum Moment (kNm @ m)		Max Def (mm @ m)
				x-x	y-y	x-x	y-y	x-x	y-y	
23	14	0.00C	0.00	22.43	0.00	0.00	0.00	20.24		3.75
	26	0.00C	0.00	-23.83	0.00	0.00	0.00	@ 1.697		@ 1.697

Classification and Effective Area (EN 1993: 2006)

Section (23.07 kg/m)	203x102 UB 23 [S 275]		
Class = Fn(b/T,d/t,f_y,N,M_y,M_z)	5.47, 31.37, 275, 0, 20.24, 0	(Axial: Non-Slender)	Class 1
Auto Design Load Cases	1		

Moment Capacity Check M.c.y.Rd

$V_{y.Ed}/V_{pl.y.Rd}$	0.078 / 196.416 =	0	Low Shear
$M_{c.y.Rd} = f_y.W_{pl.y}/\gamma_{M0}$	275 x 234.1/1	64.378 kN.m	
$M_{y.Ed}/M_{c.y.Rd}$	20.237 / 64.378 =	0.314	OK

Equivalent Uniform Moment Factor C1

C_1= fn(M_1, M_2, M_o, ~y,~m)	0.0, 0.0, 20.2, 0.621, 300.000	1.127	Uniform

Lateral Buckling Check M.b.Rd

Le = 1.2L+2D	1.2 x 3.393 + 2 x 0.203 =	4.478 m	
M_{cr} = Fn(C_1,L_e,I_z,I_t,I_w,E)	1.127, 4.478, 164.8, 7.019, 0.01537, 210000	39.627 kN.m	
$\lambda_{LT} = \sqrt{W.f_y/M_{cr}}$	√ 234.1 x 275 / 39.627	1.275	
c_{LT} = Fn(λ_{LT}, λ_{LT}5950)	1.275, 1.327	0.537	Curve b
c_{LT}.mod = Fn(c_{LT},λ_{LT},k_c,f)	0.537, 1.275, 0.942, 0.984	0.546	6.3.2.3
$M_{b.Rd} = c\ W_{pl.y}.f_y \leq M_{c.y.Rd}$	0.546 x 234.1 x 275 ≤ 64.378 =	35.151 kN.m	
$M_{y.Ed}/M_{b.Rd}$	20.237 / 35.151	0.576	OK

Deflection Check - Load Case 2

Deflection Limits (Internal Beams)	In-span δ ≤ 3393/360 = 9.4 mm Live (Case 2)	2.81 mm	OK
	In-span δ ≤ 3393/250 = 13.6 mm D+L (Case 3)	3.75 mm	OK

Provide Section :- 203 x 102 UB 23 (S275)
Bearing :- Beam to extend to rear of 330mm long x 102mm wide 3 Course Class B Engineering Brick Padstone.

Member Loading

Load to B106 (Area loading included)

Solid wall	12.80*2.7/2.7=12.8*2=25.6		
Parapet wall	12.80*1.5/2.7=7.11		
Railing	1.0		
1st Floor	3.65/2*0.60=1.10	3.65/2*2.0=3.65	
2nd Floor	3.65/2*0.60=1.10	3.65/2*2.0=3.65	
3rd Floor	1.85/2*0.60=0.56	1.85/2*2.5=2.31	
Total	**36.47 kN/m Dead**	**9.61 kN/m Live**	**(Unfactored)**

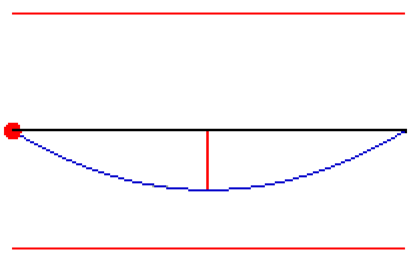
AXIAL WITH MOMENTS (MEMBER)

Beam 106 (Dead & Live Load Case)

Member 19 (N.17-N.27) @ Level 1 in Load Case 1

Member Loading and Member Forces
Loading Combination : 1 UT + 1.35 D1 + 1.5 L1

D1 UDLY -001.088	(kN/m)
L1 UDLY -003.628	(kN/m)
D1 D 077.010	(kN/m^3)
D1 UDLY -035.370	(kN/m)
L1 UDLY -005.960	(kN/m)



Member Forces in Load Case 1 and Maximum Deflection from Load Case 3										
Member No.	Node End 1 End 2	Axial Force (kN)	Torque Moment (kNm)	Shear Force (kN)		Bending Moment (kNm)		Maximum Moment (kNm @ m)		Max Def (mm @ m)
				x-x	y-y	x-x	y-y	x-x	y-y	
19	17	0.00C	0.00	93.68	0.00	0.00	0.00	68.34	0.00	4.57
	27	0.00C	0.00	-93.68	0.00	0.00	0.00	@ 1.459	@ 0.000	@ 1.459

Classification and Effective Area (EN 1993: 2006)

Section (46.1 kg/m)	203x203 UC 46 [S 275]		
Class = Fn(b/T,d/t,f_y,N,M_y,M_z)	9.25, 22.33, 275, 0, 68.34, 0	(Axial: Non-Slender)	Class 1
Auto Design Load Cases	1		

Moment Capacity Check M.c.y.Rd

$V_{y.Ed}/V_{pl.y.Rd}$	0.009 / 269.498 =	0	Low Shear
$M_{c.y.Rd} = f_y.W_{pl.y}/\ \gamma_{M0}$	275 x 497.4/1	136.785 kN.m	
$M_{y.Ed}/M_{c.y.Rd}$	68.34 / 136.785 =	0.500	OK

Equivalent Uniform Moment Factor C1

C_1= fn(M_1, M_2, M_o, ~y,~m)	0.1, 0.1, 68.2, 0.978, 300.000	1.127	Uniform

Lateral Buckling Check M.b.Rd

Le = 1.2L+2D	1.2 x 2.918 + 2 x 0.203 =	3.908 m	
M_{cr} = Fn(C_1,L_e,I_z,I_t,I_w,E)	1.127, 3.908, 1551, 22.15, 0.1429, 210000	315.733 kN.m	
$\lambda_{LT} = \sqrt{W.f_y/M_{cr}}$	√ 497.4 x 275 / 315.733	0.658	
c_{LT} = Fn(λ_{LT}, λ_{LT}5950)	0.658, 0.630	0.890	Curve b
c_{LT}.mod = Fn(c_{LT},λ_{LT},k_c,f)	0.890, 0.658, 0.942, 0.972	0.916	6.3.2.3
$M_{b.Rd}$ = c $W_{pl.y}.f_y \leq M_{c.y.Rd}$	0.916 x 497.4 x 275 ≤ 136.785 =	125.236 kN.m	
$M_{y.Ed}/M_{b.Rd}$	68.34 / 125.236	0.546	OK

Deflection Check - Load Case 3

Deflection Limits (Internal Beams)	In-span δ ≤ 2918/360 = 8.1 mm Live (Case 2)	0.94 mm	OK
	In-span δ ≤ 2918/250 = 11.7 mm D+L (Case 3)	4.57 mm	OK

Wind Load to Sway Frame

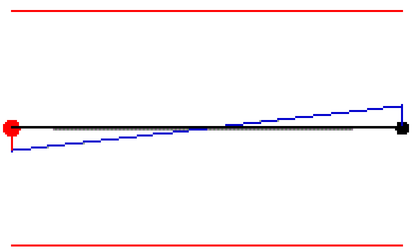
Pw= 0.52*[(7.20/2*2.7/2) + (5.50/2*2.7) + (7.20/2*2.7)]=11.45kN

AXIAL WITH MOMENTS (MEMBER)

Beam 106 (Wind Load Case)

Member 1 (N.1-N.2) @ Level 1 in Load Case 4

Member Loading and Member Forces
Loading Combination : 1 UT + 1.5 W1



Member Forces in Load Case 4 and Maximum Deflection from Load Case 5							
Span No.	Axial Force (kN)	Shear Force (kN)		Bending Moment (kNm)		Maximum Moment (kNm)	Maximum Deflection (mm @ m)
		End 1	End 2	Node 1	Node 2		
1	1	8.583C	-17.175	25.776		0.259	
2	8.583C	-17.175	-25.749		@ 1.500		

Classification and Effective Area (EN 1993: 2006)

Section (46.1 kg/m)	203x203 UC 46 [S 275]		
Class = Fn(b/T,d/t,f_y,N,M_y,M_z)	9.25, 22.33, 275, 8.58, 25.78, 0	(Axial: Non-Slender)	Class 1

Auto Design Load Cases 1 & 4

Local Capacity Check

$V_{y.Ed}/V_{pl.y.Rd}$	17.175 / 269.498 =	0.064	Low Shear
$M_{c.y.Rd} = f_y.W_{pl.y}/\ \gamma_{M0}$	275 x 497.4/1	136.785 kN.m	
$N_{pl.Rd} = A_g.f_y/\ \gamma_{M0}$	58.73 x 275/1 =	1615.075 kN	
$n = N_{Ed}/N_{pl.Rd}$	8.583 / 1615.075 =	0.005	OK
$W_{pl.N.y} = Fn(W_{pl.y}, A_{vy},)$	497.4, 16.974, 0.005	497.4 cm^3	
$M_{N.y.Rd} = W_{pl.N.y}.f_y/\ \gamma_{M0}$	497.4 x 275/1	136.785 kN.m	
$(M_{y.Ed}/M_{N.y.Rd+(M\_(z.Ed}/M_{N.z.Rd})$	$(25.776/136.785)^2+(0)^1=$	0.036	OK

Compression Resistance N.b.Rd

$\lambda y = \sqrt{A.fy/Ncr}$	$\sqrt{58.73x275/10525.9}$	0.392	
$N_{b.y.Rd} = Area.c.f_y/\ \gamma_{M1}$	58.73x0.929x275/10/1 =	1500.914 kN	Curve b
$\lambda z = \sqrt{A.fy/Ncrz}$	$\sqrt{58.73x275/3572.04}$	0.672	
$N_{b.z.Rd} = Area.c.f_y/\ \gamma_{M1}$	58.73x0.742x275/10/1 =	1197.991 kN	Curve c
Let = Kt.Lx	1x3 =	3	
$\lambda T = \sqrt{A.fy/NcrT}$	$\sqrt{58.73x275/4874.8}$	0.576	
$N_{b.T.Rd}$= Area.c.fy/ γ_{M1}	58.73x0.8x275/10/1 =	1291.661 kN	Curve c

Equivalent Uniform Moment Factor C1

C_1= fn(M_1, M_2, ~y)	25.8, -25.7, -0.999	2.550	Not Loaded
C_{mLT}=Max(0.6+0.4~y, 0.4)	M = 25.76, ~y = -0.999	0.4	Table B.3
C_{mz}=Max(0.6+0.4~y, 0.4)	M = 0, ~y = 1.000	1	Table B.3
C_{my}=Max(0.2+0.8α_s, 0.4)	M_h = 25.78, M_s = 0.01, ~y = -0.999, α_s = 0.001	0.4	Table B.3

Lateral Buckling Check M.b.Rd

Le = 1.2L+2D	1.2 x 3 + 2 x 0.203 =	4.006 m	
M_{cr} = Fn(C_1,L_e,I_z,I_t,I_w,E)	2.550, 4.006, 1551, 22.15, 0.1429, 210000	687.999 kN.m	
$\lambda_{LT} = \sqrt{W.f_y/M_{cr}}$	$\sqrt{497.4 \times 275 / 687.999}$	0.446	
c_{LT} = Fn(λ_{LT}, λ_{LT}5950)	0.446, 0.642	0.982	Curve b
c_{LT}.mod = Fn(c_{LT},λ_{LT},k_c,f)	0.982, 0.446, 0.626, 0.860	1.000	6.3.2.3
$M_{b.Rd} = c\ W_{pl.y}.f_y \le M_{c.y.Rd}$	1.000 x 497.4 x 275 ≤ 136.785 =	136.785 kN.m	

Buckling Resistance

$U_{N.y} = N_{Ed}/(c_y.N_{Rk}/\gamma_{M1})$	8.583 / 1500.914	0.006	OK
$U_{N.z} = N_{Ed}/(c_z.N_{Rk}/\gamma_{M1})$	8.583 / 1197.991	0.007	OK
$U_{M.y} = M_{y.Ed}/(c_{LT}.M_{y.Rk}/\gamma_{M1})$	25.776 / 136.785	0.188	OK
$U_{M.z} = M_{z.Ed}/(M_{z.Rk}/\gamma_{M1})$	0 / 63.498	0.000	OK
$k_yy=C_{my}\{1+(\lambda_y-0.2)U_{N.y}\}$		0.400	
$k_zz=C_{mz}\{1+(2\lambda_z-0.6)U_{N.z}\}$		1.005	
k_yz=0.6 k_zz		0.603	
k_zy=0.6 k_yy		0.240	
$U_{Ny}+k_yy.U_{M.y}+k_yz.U_{M.z}$	0.006+0.400x0.188+0.603x0.000	0.081	OK
$U_{Nz}+k_zy.U_{M.y}+k_zz.U_{M.z}$	0.007+0.240x0.188+1.005x0.000	0.052	OK

Deflection Check - Load Case 3

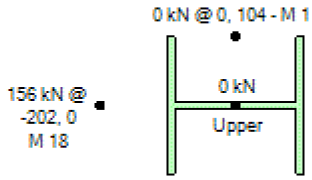
Deflection Limits (Internal Beams)	In-span δ ≤ 3000/250 = 12 mm D+L (Case 3)	0.03 mm	OK

Provide Section :- 203 x 203 UC 46 (S275) to support solid wall over & act as sway frame for lateral loads

COLUMNS IN SIMPLE CONSTRUCTION

Columns C101 – C102 & C105-C106 (Dead & Live Load Case)

Member 28 (N.1-N.2) @ Level 1 in Load Case 1



Column in Simple Construction check not support by EN 1993:2006. Capacities and components calcualted in accoradance with EN 1993 and design check presented according to BS 5950. Caution

Classification and Effective Area (EN 1993: 2006)

Section (46.1 kg/m)	203x203 UC 46 [S 275]		
Class = Fn(b/T,d/t,f_y,N,M_y,M_z)	9.25, 22.33, 275, 157.99, 0, 0	(Axial: Non-Slender)	Class 1
Auto Design Load Cases	1		

Applied Factored Loads

$Fc=F+F_{y1}+F_{y2}+F_{z1}+F_{z2}+Fa$	0+0+155.912+0.249+0+1.832	157.993 kN	
$M_y=-F_{y2}.e_{y2}$	-156x202	31.432 kN.m	
$M_z=F_{z1}.e_{z1}$	0x104	0.026 kN.m	

Compression Resistance N.b.Rd

$\lambda y = \sqrt{A.fy/Ncr}$	√58.73x275/10525.9	0.392	
$N_{b.y.Rd}$ = Area.c.f_y/ γ_{M1}	58.73x0.929x275/10/1 =	1500.914 kN	Curve b
$\lambda z = \sqrt{A.fy/Ncrz}$	√58.73x275/3572.04	0.672	
$N_{b.z.Rd}$ = Area.c.f_y/ γ_{M1}	58.73x0.742x275/10/1 =	1197.991 kN	Curve c
Let = Kt.Lx	1x3 =	3	
$\lambda T = \sqrt{A.fy/NcrT}$	√58.73x275/4874.8	0.576	
$N_{b.T.Rd}$= Area.c.fy/ γ_{M1}	58.73x0.8x275/10/1 =	1291.661 kN	Curve c

Buckling Resistance Moment M.b.Rd

$\lambda LT = 0.5\ L / (i_z.\sim p.\ \sqrt{E/f_y})$	0.5x100x3/ (5.14x ~p x √ 210000/275)	0.34	
$c_{LT} = Fn(\lambda_{LT})$	0.336	1.000	Curve d
$M_{b.Rd} = c\ W_{pl.y}.f_y \leq M_{c.y.Rd}$	1.000 x 497.4 x 275 ≤ 136.785 =	136.785 kN.m	

Columns in Simple Construction

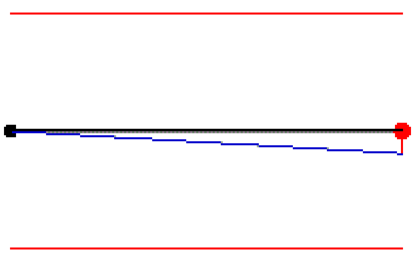
$Fc/N_{b.Rd}+M_y/M_{b.Rd}+M_z/f_y.W_{el.z}$	157.993/1197.991+31.432/136.785+0.026/(275x152.37)	0.362	OK

AXIAL WITH MOMENTS (MEMBER)

Columns C101 – C102 & C105-C106 (Wind Load Case)

Member 3 (N.2-N.4) @ Level 1 in Load Case 4

Member Loading and Member Forces
Loading Combination : 1 UT + 1.5 W1



Member Forces in Load Case 4 and Maximum Deflection from Load Case 5							
Span No.	Axial Force (kN)	Shear Force (kN)		Bending Moment (kNm)		Maximum Moment (kNm)	Maximum Deflection (mm @ m)
		End 1	End 2	Node 1	Node 2		
3	2	10.717C	8.658	-25.974		1.042	
4	10.717C	8.658	0.000		@ 1.260		

Classification and Effective Area (EN 1993: 2006)

Section (46.1 kg/m)	203x203 UC 46 [S 275]		
Class = Fn(b/T,d/t,f_y,N,M_y,M_z)	9.25, 22.33, 275, 10.72, 25.97, 0	(Axial: Non-Slender)	Class 1
Auto Design Load Cases	1 & 4		

Local Capacity Check

$V_{y.Ed}/V_{pl.y.Rd}$	8.658 / 269.498 =	0.032	Low Shear
$M_{c.y.Rd} = f_y.W_{pl.y}/\gamma_{M0}$	275 x 497.4/1	136.785 kN.m	
$N_{pl.Rd} = A_g.f_y/\gamma_{M0}$	58.73 x 275/1 =	1615.075 kN	
$n = N_{Ed}/N_{pl.Rd}$	10.717 / 1615.075 =	0.007	OK
$W_{pl.N.y} = Fn(W_{pl.y}, A_{vy})$	497.4, 16.974, 0.007	497.4 cm^3	
$M_{N.y.Rd} = W_{pl.N.y}.f_y/\gamma_{M0}$	497.4 x 275/1	136.785 kN.m	
$(M_{y.Ed}/M_{N.y.Rd+(M\_(z.Ed}/M_{N.z.Rd})$	$(25.974/136.785)^2+(0)^1=$	0.036	OK

Compression Resistance N.b.Rd

$\lambda y = \sqrt{A.fy/Ncr}$	$\sqrt{58.73x275/10525.9}$	0.392	
$N_{b.y.Rd}$ = Area.c.f_y/γ_{M1}	58.73x0.929x275/10/1 =	1500.914 kN	Curve b
$\lambda z = \sqrt{A.fy/Ncrz}$	$\sqrt{58.73x275/3572.04}$	0.672	
$N_{b.z.Rd}$ = Area.c.f_y/γ_{M1}	58.73x0.742x275/10/1 =	1197.991 kN	Curve c
Let = Kt.Lx	1x3 =	3	
$\lambda T = \sqrt{A.fy/NcrT}$	$\sqrt{58.73x275/4874.8}$	0.576	
$N_{b.T.Rd}$= Area.c.fy/ γ_{M1}	58.73x0.8x275/10/1 =	1291.661 kN	Curve c

Equivalent Uniform Moment Factor C1

C_1= fn(M_1, M_2, ~y)	-26.0, 0.0, 0.000	1.750	Not Loaded
C_{mLT}=Max(0.6+0.4~y, 0.4)	M = -25.97, ~y = 0.000	0.6	Table B.3
C_{mz}=Max(0.6+0.4~y, 0.4)	M = 0, ~y = 1.000	1	Table B.3
C_{my}=Max(0.6+0.4~y, 0.4)	M = -25.97, ~y = 0.000	0.6	Table B.3

Lateral Buckling Check M.b.Rd

Le = 1.2L+2D	1.2 x 3 + 2 x 0.203 =	4.006 m	
M_{cr} = Fn(C_1,L_e,I_z,I_t,I_w,E)	1.750, 4.006, 1551, 22.15, 0.1429, 210000	472.069 kN.m	
$\lambda_{LT} = \sqrt{W.f_y/M_{cr}}$	√ 497.4 x 275 / 472.069	0.538	
c_{LT} = Fn(λ_{LT}, λ_{LT}5950)	0.538, 0.642	0.944	Curve b
c_{LT}.mod = Fn(c_{LT},λ_{LT},k_c,f)	0.944, 0.538, 0.756, 0.895	1.000	6.3.2.3
$M_{b.Rd}$ = c $W_{pl.y}.f_y \leq M_{c.y.Rd}$	1.000 x 497.4 x 275 ≤ 136.785 =	136.785 kN.m	

Buckling Resistance

$U_{N.y} = N_{Ed}/(c_y.N_{Rk}/\gamma_{M1})$	10.717 / 1500.914	0.007	OK
$U_{N.z} = N_{Ed}/(c_z.N_{Rk}/\gamma_{M1})$	10.717 / 1197.991	0.009	OK
$U_{M.y} = M_{y.Ed}/(c_{LT}.M_{y.Rk}/\gamma_{M1})$	25.974 / 136.785	0.190	OK
$U_{M.z} = M_{z.Ed}/(M_{z.Rk}/\gamma_{M1})$	0 / 63.498	0.000	OK
$k_y y=C_{my}\{1+(\lambda_y-0.2)U_{N.y}\}$		0.601	
$k_z z=C_{mz}\{1+(2\lambda_z-0.6)U_{N.z}\}$		1.007	
$k_y z$=0.6 $k_z z$		0.604	
$k_z y$=0.6 $k_y y$		0.360	
$U_{Ny}+k_y y.U_{M.y}+k_y z.U_{M.z}$	0.007+0.601x0.190+0.604x0.000	0.121	OK
$U_{Nz}+k_z y.U_{M.y}+k_z z.U_{M.z}$	0.009+0.360x0.190+1.007x0.000	0.077	OK

Deflection Check - Load Case 3

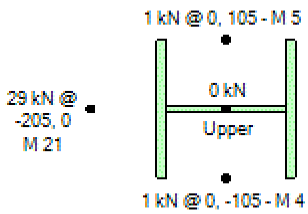
Deflection Limits (Internal Beams)	In-span δ ≤ 3000/250 = 12 mm D+L (Case 3)	0.05 mm	OK

Provide Section :- 203 x 203 UC 46 (S275)

COLUMNS IN SIMPLE CONSTRUCTION

Columns C103 & C104 (Dead & Live Load Case)

Member 30 (N.3-N.12) @ Level 1 in Load Case 1



Column in Simple Construction check not support by EN 1993:2006. Capacities and components calcualted in accoradance with EN 1993 and design check presented according to BS 5950. Caution

Classification and Effective Area (EN 1993: 2006)

Section (59.95 kg/m)	203x203 UC 60 [S 275]		
Class = Fn(b/T,d/t,f_y,N,M_y,M_z)	7.25, 17.11, 275, 32.82, 0, 0	(Axial: Non-Slender)	Class 1
Auto Design Load Cases	1		

Applied Factored Loads

$Fc=F+F_{y1}+F_{y2}+F_{z1}+F_{z2}+Fa$	0+0+29.051+0.539+0.852+2.382	32.824 kN	
$M_y=-F_{y2}.e_{y2}$	-29x205	5.95 kN.m	
$M_z=F_{z1}.e_{z1}-F_{z2}.e_{z2}$	1x105-1x105	0.033 kN.m	

Compression Resistance N.b.Rd

$\lambda y = \sqrt{A.fy/Ncr}$	√76.37x275/14110.83	0.386	
$N_{b.y.Rd}$ = Area.c.f_y/ γ_{M1}	76.37x0.932x275/10/1 =	1956.772 kN	Curve b
$\lambda z = \sqrt{A.fy/Ncrz}$	√76.37x275/4761.26	0.665	
$N_{b.z.Rd}$ = Area.c.f_y/ γ_{M1}	76.37x0.747x275/10/1 =	1567.791 kN	Curve c
Let = Kt.Lx	1x3 =	3	
$\lambda T = \sqrt{A.fy/NcrT}$	√76.37x275/7779.74	0.52	
$N_{b.T.Rd}$= Area.c.fy/ γ_{M1}	76.37x0.832x275/10/1 =	1747.311 kN	Curve c

Buckling Resistance Moment M.b.Rd

$\lambda LT = 0.5\ L / (i_z.\sim p.\ \sqrt{E/f_y})$	0.5x100x3/ (5.2x ~p x √ 210000/275)	0.33	
$c_{LT} = Fn(\lambda_{LT})$	0.332	1.000	Curve d
$M_{b.Rd} = c\ W_{pl.y}.f_y \leq M_{c.y.Rd}$	1.000 x 656.1 x 275 ≤ 180.428 =	180.428 kN.m	

Columns in Simple Construction

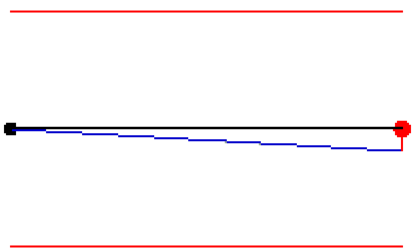
$Fc/N_{b.Rd}+M_y/M_{b.Rd}+M_z/f_y.W_{el.z}$	32.824/1567.791+5.95/180.428+0.033/(275x200.92)	0.055	OK

AXIAL WITH MOMENTS (MEMBER)

Columns C103 & C104 (Wind Load Case)

Member 2 (N.1-N.3) @ Level 1 in Load Case 4

Member Loading and Member Forces
Loading Combination : 1 UT + 1.5 W1



Member Forces in Load Case 4 and Maximum Deflection from Load Case 5							
Span No.	Axial Force (kN)	Shear Force (kN)		Bending Moment (kNm)		Maximum Moment (kNm)	Maximum Deflection (mm @ m)
		End 1	End 2	Node 1	Node 2		
2	1	16.875T	11.257	-33.771		1.010	

Member Forces in Load Case 4 and Maximum Deflection from Load Case 5							
Span No.	Axial Force (kN)	Shear Force (kN)		Bending Moment (kNm)		Maximum Moment (kNm)	Maximum Deflection (mm @ m)
		End 1	End 2	Node 1	Node 2		
3	16.875T	11.257	0.000		@ 1.260		

Classification and Effective Area (EN 1993: 2006)

Section (59.95 kg/m)	203x203 UC 60 [S 275]		
Class = Fn($b/T, d/t, f_y, N, M_y, M_z$)	7.25, 17.11, 275, 0, 33.77, 0	(Axial: Non-Slender)	Class 1
Auto Design Load Cases	1 & 4		

Local Capacity Check

$V_{y.Ed}/V_{pl.y.Rd}$	11.257 / 351.748 =	0.032	Low Shear
$M_{c.y.Rd} = f_y.W_{pl.y}/\gamma_{M0}$	275 x 656.1/1	180.428 kN.m	
$N_{pl.Rd} = Ag.fy/\gamma_{M0}$	76.37x275/1 (No bearing / block tearing design)	2100.175 kN	
$n = N_{Ed}/N_{pl.Rd}$	-16.875 / 2100.175 =	0.008	OK
$W_{pl.N.y} = Fn(W_{pl.y}, A_{vy},)$	656.1, 22.154, 0.008	656.1 cm^3	
$M_{N.y.Rd} = W_{pl.N.y}.f_y/\gamma_{M0}$	656.1 x 275/1	180.428 kN.m	
($M_{y.Ed}/M_{N.y.Rd+(M\_(z.Ed}/M_{N.z.Rd}$)	$(33.771/180.428)^2+(0)^1=$	0.035	OK

Equivalent Uniform Moment Factor C1

C_1= fn(M_1, M_2, ~y)	-33.8, 0.0, 0.000	1.750	Not Loaded
C_{mLT}=Max(0.6+0.4~y, 0.4)	M = -33.76, ~y = 0.000	0.6	Table B.3
C_{mz}=Max(0.6+0.4~y, 0.4)	M = 0, ~y = 1.000	1	Table B.3
C_{my}=Max(0.6+0.4~y, 0.4)	M = -33.77, ~y = 0.000	0.6	Table B.3

Lateral Buckling Check M.b.Rd

Le = 1.2L+2D	1.2 x 3 + 2 x 0.210 =	4.019 m	
M_{cr} = Fn($C_1, L_e, I_z, I_t, I_w, E$)	1.750, 4.019, 2068, 47.23, 0.1969, 210000	717.595 kN.m	
$\lambda_{LT} = \sqrt{W.f_y/M_{cr}}$	$\sqrt{656.1 \times 275 / 717.595}$	0.501	
c_{LT} = Fn(λ_{LT}, λ_{LT}5950)	0.501, 0.598	0.960	Curve b
c_{LT}.mod = Fn($c_{LT}, \lambda_{LT}, k_c, f$)	0.960, 0.501, 0.756, 0.900	1.000	6.3.2.3
$M_{b.Rd} = c\ W_{pl.y}.f_y \le M_{c.y.Rd}$	1.000 x 656.1 x 275 ≤ 180.428 =	180.428 kN.m	

Buckling Resistance

$U_{N.y} = N_{Ed}/(c_y.N_{Rk}/\gamma_{M1})$	0 / 1956.772	0.000	OK
$U_{N.z} = N_{Ed}/(c_z.N_{Rk}/\gamma_{M1})$	0 / 1567.791	0.000	OK
$U_{M.y} = M_{y.Ed}/(c_{LT}.M_{y.Rk}/\gamma_{M1})$	33.771 / 180.428	0.187	OK
$U_{M.z} = M_{z.Ed}/(M_{z.Rk}/\gamma_{M1})$	0 / 83.958	0.000	OK
$k_yy=C_{my}\{1+(\lambda_y-0.2)U_{N.y}\}$		0.600	
$k_zz=C_{mz}\{1+(2\lambda_z-0.6)U_{N.z}\}$		1.000	
k_yz=0.6 k_zz		0.600	
k_zy=0.6 k_yy		0.360	
$U_{Ny}+k_yy.U_{M.y}+k_yz.U_{M.z}$	0.000+0.600x0.187+0.600x0.000	0.112	OK
$U_{Nz}+k_zy.U_{M.y}+k_zz.U_{M.z}$	0.000+0.360x0.187+1.000x0.000	0.067	OK

Deflection Check - Load Case 3

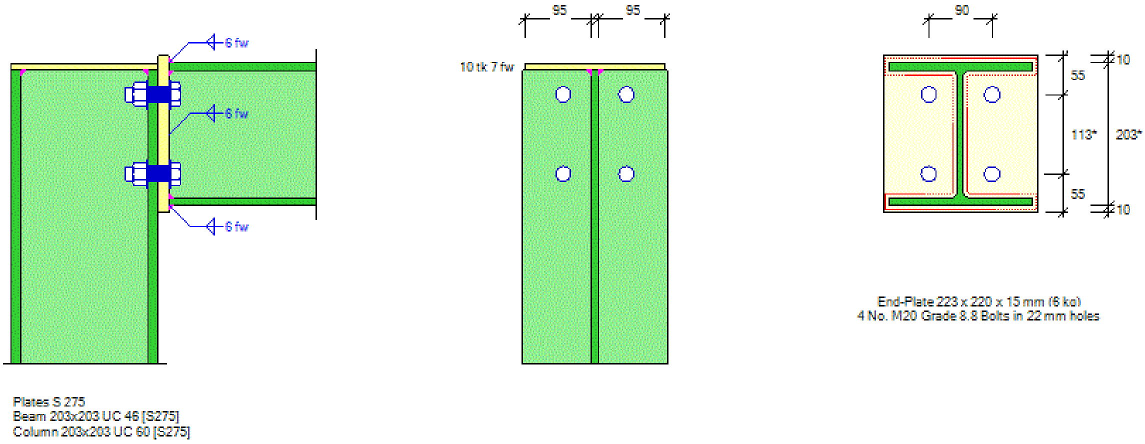
Deflection Limits (Internal Beams)	In-span δ ≤ 3000/250 = 12 mm D+L (Case 3)	0.02 mm	OK

Provide Section :- 203 x 203 UC 60 (S275)

CONNECTION DESIGN

EAVES JOINT AT : N.12 - LEVEL 1 : MEMBER 21 (N.12-N.19)

Connection 1: Beam 104 Connected to Columns C103 & C104



BEAM TO COLUMN FLANGE END-PLATED CONNECTION TO EC 3 (UK NAD)

LOADING CASE 001 : DEAD PLUS LIVE (ULTIMATE)

Basic Data

Integrated Applied Forces at Column/Right Rafter Interface

Resultant Forces M, Fv, F	-35.0 kNm, 28.1 kN, 0.0 kN
Load directions	Bottom of Joint in Tension, Rafter moving Down and in Compression.
Design to	EC 3: Part 1-8: 2005 Design of Joints
	SCI Green Book
	P398: Joints in steel construction: Moment-Resisting Joints to Eurocode 3
Weld Grades	All weld grades provided to suit minimum connected steel grade

Basic Dimensions

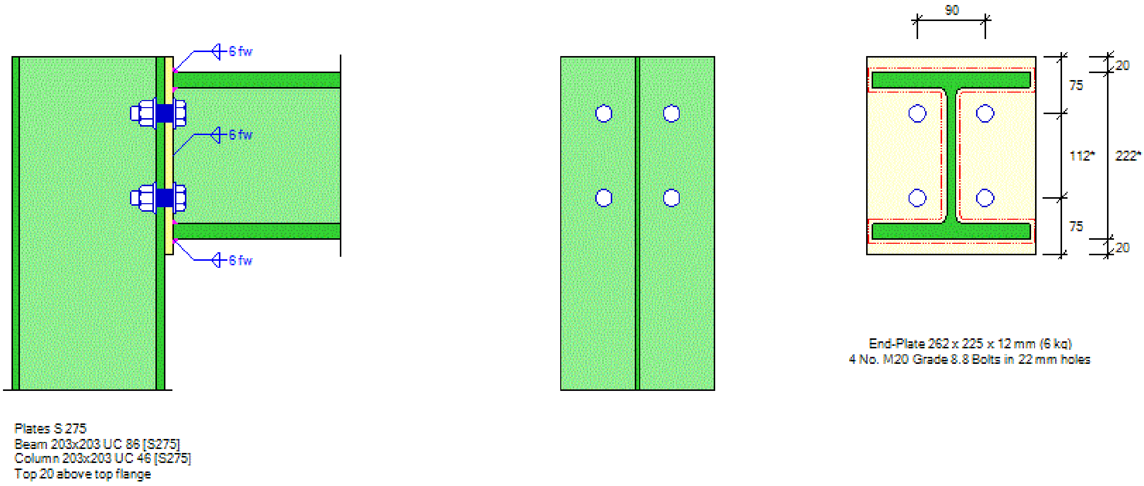
Column-203x203UC60 [28]	D=209.6, B=205.8, T=14.2, t=9.4, r=10.2, py=275		
Beam-203x203UC46 [28]	D=203.2, B=203.6, T=11.0, t=7.2, r=10.2, py=275		
Bolts 20 mm Ø in 22 mm holes	Grade 8.8 Bolts		
Plates S 275	All weld grades provided to suit minimum connected steel grade		
Rafter Capacities Mc, Fvc, Fc	136.8 kN.m, 269.5 kN, 1615.1 kN	Mc = 136.8 kN.m	OK

Summary of Results (Unity Ratios)

Moment Capacity 35.7 kNm (for 1 rows of bolts) (Modified Applied Mom. M_{mod} =35.0 kNm)		0.98	OK
Shear Capacity		0.12	OK
Flange Welds	0.51	0.51	OK
Web Welds	0.86, 0.25	0.86	OK
Column Compression stiff Web Weld	0.65	0.65	OK

EAVES JOINT AT : N.2 - LEVEL 1 : MEMBER 18 (N.2-N.6)

Connection 2: Beam 101 Connected to Columns C101 & C102



Beam to Column Flange End-Plated Connection to EC 3 (UK NAD)

LOADING CASE 001 : DEAD PLUS LIVE (ULTIMATE)

Basic Data

Integrated Applied Forces at Column/Right Rafter Interface

Resultant Forces M, Fv, F	-26.0 kNm, 149.6 kN, 0.0 kN
Load directions	Bottom of Joint in Tension, Rafter moving Down and in Tension.
Design to	EC 3: Part 1-8: 2005 Design of Joints
	SCI Green Book
	P398: Joints in steel construction: Moment-Resisting Joints to Eurocode 3
Weld Grades	All weld grades provided to suit minimum connected steel grade

Basic Dimensions

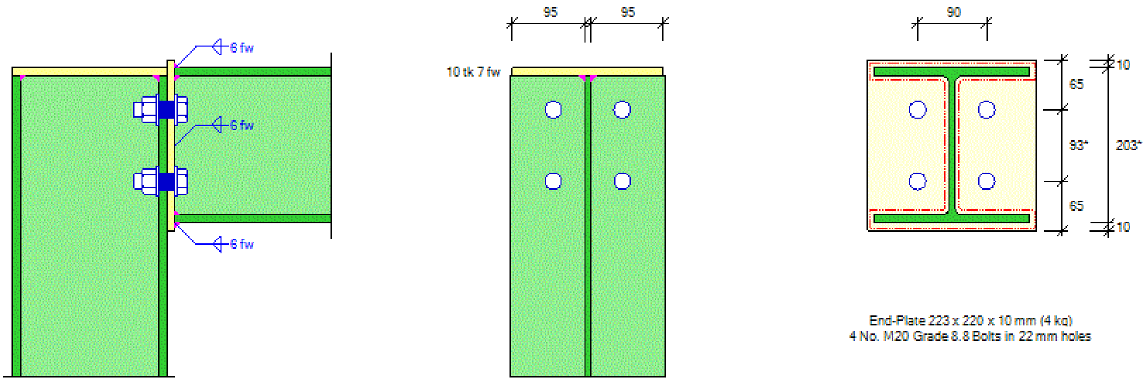
Column-203x203UC46 [28]	D=203.2, B=203.6, T=11.0, t=7.2, r=10.2, py=275		
Beam-203x203UC86 [28]	D=222.2, B=209.1, T=20.5, t=12.7, r=10.2, py=265		
Bolts 20 mm Ø in 22 mm holes	Grade 8.8 Bolts		
Plates S 275	All weld grades provided to suit minimum connected steel grade		
Rafter Capacities Mc, Fvc, Fc	258.8 kN.m, 469.5 kN, 2905.2 kN	Fvc = 469.5 kN	OK

Summary of Results (Unity Ratios)

Tensile Capacity		0.00	OK
Moment Capacity 28.0 kNm (for 1 rows of bolts) (Modified Applied Mom. M_{mod} =26.0 kNm)		0.93	OK
Shear Capacity		0.62	OK
Shear Capacity (Fv & T)		0.40	OK
Flange Welds	0.36	0.36	OK
Web Welds	0.35, 0.35, 0.51	0.51	OK

EAVES JOINT AT : N.17 - LEVEL 1 : MEMBER 19 (N.17-N.27)

Connection 3: Beam 106 Connected to Columns C105 & C106



Plates S 275
Beam 203x203 UC 46 [S275]
Column 203x203 UC 46 [S275]

Beam to Column Flange End-Plated Connection to EC 3 (UK NAD)

LOADING CASE 001 : DEAD PLUS LIVE (ULTIMATE)

Basic Data

Integrated Applied Forces at Column/Right Rafter Interface

Resultant Forces M, Fv, F	-26.0 kNm, 87.2 kN, 0.0 kN
Load directions	Bottom of Joint in Tension, Rafter moving Down and in Tension.
Design to	EC 3: Part 1-8: 2005 Design of Joints
	SCI Green Book
	P398: Joints in steel construction: Moment-Resisting Joints to Eurocode 3
Weld Grades	All weld grades provided to suit minimum connected steel grade

Basic Dimensions

Column-203x203UC46 [28]	D=203.2, B=203.6, T=11.0, t=7.2, r=10.2, py=275		
Beam-203x203UC46 [28]	D=203.2, B=203.6, T=11.0, t=7.2, r=10.2, py=275		
Bolts 20 mm Ø in 22 mm holes	Grade 8.8 Bolts		
Plates S 275	All weld grades provided to suit minimum connected steel grade		
Rafter Capacities Mc, Fvc, Fc	136.8 kN.m, 269.5 kN, 1615.1 kN	Fvc = 269.5 kN	OK

Summary of Results (Unity Ratios)

Tensile Capacity		0.00	OK
Moment Capacity 28.3 kNm (for 1 rows of bolts) (Modified Applied Mom. M_{mod} =26.0 kNm)		0.92	OK
Shear Capacity		0.36	OK
Shear Capacity (Fv & T)		0.23	OK
Flange Welds	0.40	0.40	OK
Web Welds	0.86, 0.94	0.94	OK
Column Compression stiff Web Weld	0.51	0.51	OK

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	Date	: September 18	
	Checked	: DJS	
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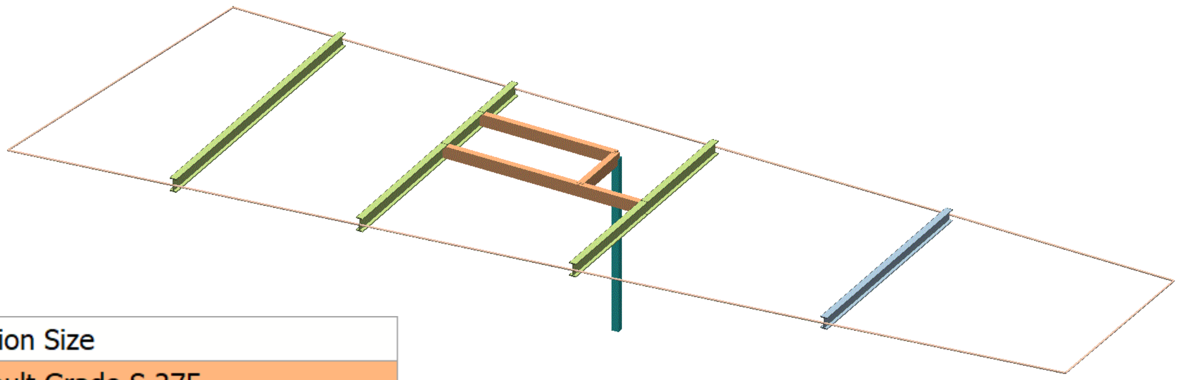
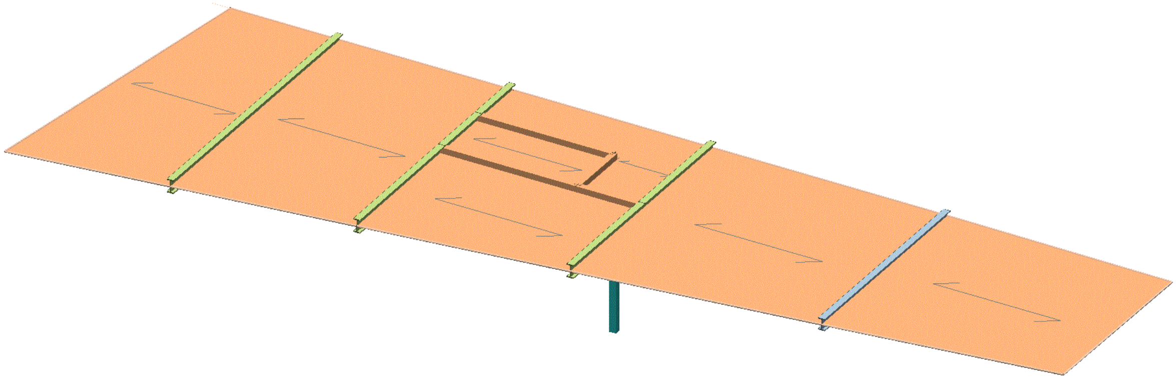
Steel Sections at First floor structure over



Section Size
Default Grade S 275
203x102 UB 23
178x102 UB 19
94x175 C24
100x100 C24

Area Loal Panels at First floor structure over



Dead D	Live L
0.600	2.000

Member Loading

Load to B201 (Area loading included)

2nd Floor	5.15/2*0.60=1.55	5.15/2*2.0=5.15	
Total	**1.55 kN/m Dead**	**5.15 kN/m Live**	**(Unfactored)**

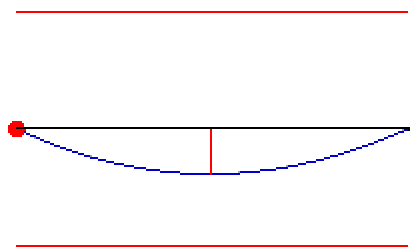
AXIAL WITH MOMENTS (MEMBER)

Beam 201

Member 11 (N.2-N.9) @ Level 1 in Load Case 1

Member Loading and Member Forces
Loading Combination : 1 UT + 1.35 D1 + 1.5 L1

D1 PTRY -000.712 0.000 4.592 -000.514
L1 PTRY -002.373 0.000 4.592 -001.713
D1 PTY2 -000.137 0.000 0.335 (kN,m,m)
D1 PTRY -000.819 0.335 4.592 -000.811
L1 PTY2 -000.457 0.000 0.335 (kN,m,m)
L1 PTRY -002.729 0.335 4.592 -002.703
D1 D 077.010 (kN/m³)



Member Forces in Load Case 1 and Maximum Deflection from Load Case 3							
Span No.	Axial Force (kN)	Shear Force (kN) End 1	Shear Force (kN) End 2	Bending Moment (kNm) Node 1	Bending Moment (kNm) Node 2	Maximum Moment (kNm)	Maximum Deflection (mm @ m)
11	2	0.000	21.173	0.000	24.659	8.377	
9	0.000	-20.999	0.000	@ 2.247	@ 2.296		

Classification and Effective Area (EN 1993: 2006)

Section (23.07 kg/m)	203x102 UB 23 [S 275]		
Class = Fn(b/T,d/t,f_y,N,M_y,M_z)	5.47, 31.37, 275, 0, 24.66, 0	(Axial: Non-Slender)	Class 1
Auto Design Load Cases	1		

Moment Capacity Check M.c.y.Rd

$V_{y.Ed}/V_{pl.y.Rd}$	0.23 / 196.416 =	0.001	Low Shear
$M_{c.y.Rd} = f_y.W_{pl.y}/\ \gamma_{M0}$	275 x 234.1/1	64.378 kN.m	
$M_{y.Ed}/M_{c.y.Rd}$	24.658 / 64.378 =	0.383	OK

Equivalent Uniform Moment Factor C1

C_1= fn(M_1, M_2, M_o, ~y,~m)	0.0, 0.0, 24.6, 0.500, 300.000	1.127	Uniform

Lateral Buckling Check M.b.Rd

Le = 1.2L+2D	1.2 x 4.592 + 2 x 0.203 =	5.917 m	
M_{cr} = Fn(C_1,L_e,I_z,I_t,I_w,E)	1.127, 5.917, 164.8, 7.019, 0.01537, 210000	28.554 kN.m	
$\lambda_{LT} = \sqrt{W.f_y/M_{cr}}$	$\sqrt{}$ 234.1 x 275 / 28.554	1.502	
c_{LT} = Fn(λ_{LT}, λ_{LT}5950)	1.502, 1.564	0.427	Curve b
c_{LT}.mod = Fn(c_{LT},λ_{LT},k_c,f)	0.427, 1.502, 0.942, 1.000	0.427	6.3.2.3
$M_{b.Rd}$ = c $W_{pl.y}.f_y \le M_{c.y.Rd}$	0.427 x 234.1 x 275 ≤ 64.378 =	27.476 kN.m	
$M_{y.Ed}/M_{b.Rd}$	24.658 / 27.476	0.897	OK

Deflection Check - Load Case 2

Deflection Limits (Internal Beams)	In-span δ ≤ 4592/360 = 12.8 mm Live (Case 2)	6.22 mm	OK
	In-span δ ≤ 4592/250 = 18.4 mm D+L (Case 3)	8.38 mm	OK

Provide Section :- 203 x 102 UB 23 (S275)
Bearing :- Beam to extend to rear of 330mm long x 102mm wide 3 Course Class B Engineering Brick Padstone.

Member Loading

Load to B202 (Area loading included)

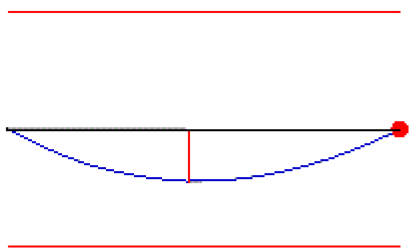
2nd Floor	5.84/2*0.60=1.75	5.84/2*2.0=5.84	
Timber wall	1.01*2.7/2.4=1.14		
Total	**2.89 kN/m Dead**	**5.84 kN/m Live**	**(Unfactored)**

AXIAL WITH MOMENTS (MEMBER)

Beam 202

Members 1-3 (N.3-N.6) @ Level 1 in Load Case 1

Member Loading and Member Forces
Loading Combination : 1 UT + 1.35 D1 + 1.5 L1



Part 1
D1 UDLY -000.814 (kN/m)
L1 UDLY -002.714 (kN/m)
D1 TRY -002.001 0.383 2.328 (kN,m,m)
L1 TRY -006.670 0.383 2.328 (kN,m,m)
D1 D 077.010 (kN/m³)
Part 2
D1 UDLY -000.814 (kN/m)
L1 UDLY -002.714 (kN/m)
D1 TRY -002.001 0.383 2.328 (kN,m,m)
L1 TRY -006.670 0.383 2.328 (kN,m,m)
D1 D 077.010 (kN/m³)
D1 UDLY -001.140 (kN/m)
Part 3
D1 UDLY -000.814 (kN/m)
L1 UDLY -002.714 (kN/m)
D1 TRY -002.001 0.383 2.328 (kN,m,m)
L1 TRY -006.670 0.383 2.328 (kN,m,m)
D1 D 077.010 (kN/m³)
D1 UDLY -001.140 (kN/m)

Member Forces in Load Case 1 and Maximum Deflection from Load Case 3

Span No.	Axial Force (kN)	Shear Force (kN)		Bending Moment (kNm)		Maximum Moment (kNm)	Maximum Deflection (mm @ m)
		End 1	End 2	Node 1	Node 2		
	3	-0.006	-24.449	0.000	-28.514	8.340	
14	-0.003	27.898	0.000	@ 2.233	@ 2.138		

Classification and Effective Area (EN 1993: 2006)

Section (23.07 kg/m)	203x102 UB 23 [S 275]		
Class = Fn(b/T,d/t,f_y,N,M_y,M_z)	5.47, 31.37, 275, 0, 28.51, 0	(Axial: Non-Slender)	Class 1
Auto Design Load Cases	1		

Moment Capacity Check M.c.y.Rd

$V_{y.Ed}/V_{pl.y.Rd}$	0.598 / 196.416 =	0.003	Low Shear
$M_{c.y.Rd} = f_y.W_{pl.y}/\gamma_{M0}$	275 x 234.1/1	64.378 kN.m	
$M_{y.Ed}/M_{c.y.Rd}$	28.499 / 64.378 =	0.443	OK

Equivalent Uniform Moment Factor C1

C_1= fn(M_1, M_2, M_o, ~y,~m)	0.0, 0.0, 28.4, 0.778, 300.000	1.127	Uniform

Lateral Buckling Check M.b.Rd

Le = 1.2L+2D	1.2 x 4.257 + 2 x 0.203 =	5.515 m	
M_{cr} = Fn(C_1,L_e,I_z,I_t,I_w,E)	1.127, 5.515, 164.8, 7.019, 0.01537, 210000	30.954 kN.m	
$\lambda_{LT} = \sqrt{W.f_y/M_{cr}}$	$\sqrt{}$ 234.1 x 275 / 30.954	1.442	
c_{LT} = Fn(λ_{LT}, λ_{LT}5950)	1.442, 1.502	0.453	Curve b
c_{LT}.mod = Fn(c_{LT},λ_{LT},k_c,f)	0.453, 1.442, 0.942, 0.995	0.455	6.3.2.3
$M_{b.Rd} = c\ W_{pl.y}.f_y \le M_{c.y.Rd}$	0.455 x 234.1 x 275 ≤ 64.378 =	29.311 kN.m	
$M_{y.Ed}/M_{b.Rd}$	28.514 / 29.311	0.973	OK

Deflection Check - Load Case 2

Deflection Limits (Internal Beams)	In-span δ ≤ 4257/360 = 11.8 mm Live (Case 2)	5.8 mm	OK

In-span δ ≤ 4257/250 = 17 mm D+L (Case 3) 8.34 mm OK

Provide Section :- **203 x 102 UB 23 (S275)**
Bearing :- **Beam to extend to rear of 330mm long x 102mm wide 3 Course Class B Engineering Brick Padstone.**

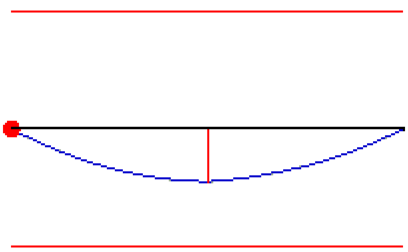
Member Loading

Load to B203 (Area loading included)

Timber wall	1.01*2.7/2.4=1.14		
2nd Floor	6.80/2*0.60=2.04	6.80/2*2.0=6.80	
Total	**3.18 kN/m Dead**	**6.80 kN/m Live**	**(Unfactored)**

AXIAL WITH MOMENTS (MEMBER)
Beam 203
Members 4-5 (N.4-N.11) @ Level 1 in Load Case 1

Member Loading and Member Forces
Loading Combination : 1 UT + 1.35 D1 + 1.5 L1



Part 1
D1 UDLY -000.940 (kN/m)
L1 UDLY -003.133 (kN/m)
D1 TRY -001.893 0.455 1.945 (kN,m,m)
L1 TRY -006.310 0.455 1.945 (kN,m,m)
D1 D 077.010 (kN/m³)
D1 UDLY -001.140 (kN/m)
Part 2
D1 UDLY -000.940 (kN/m)
L1 UDLY -003.133 (kN/m)
D1 TRY -001.893 0.455 1.945 (kN,m,m)
L1 TRY -006.310 0.455 1.945 (kN,m,m)
D1 D 077.010 (kN/m³)
D1 UDLY -001.140 (kN/m)

Member Forces in Load Case 1 and Maximum Deflection from Load Case 3

Span No.	Axial Force (kN)	Shear Force (kN)		Bending Moment (kNm)		Maximum Moment (kNm)	Maximum Deflection (mm @ m)
		End 1	End 2	Node 1	Node 2		
	4	0.000	27.642	0.000	28.601	6.799	
16	0.000	-28.068	0.000	@ 1.945	@ 1.925		

Classification and Effective Area (EN 1993: 2006)

Section (23.07 kg/m)	203x102 UB 23 [S 275]		
Class = Fn($b/T,d/t,f_y,N,M_y,M_z$)	5.47, 31.37, 275, 0, 28.6, 0	(Axial: Non-Slender)	Class 1
Auto Design Load Cases	1		

Moment Capacity Check M.c.y.Rd

$V_{y.Ed}/V_{pl.y.Rd}$	3.606 / 196.416 =	0.018	Low Shear
$M_{c.y.Rd} = f_y.W_{pl.y}/\gamma_{M0}$	275 x 234.1/1	64.378 kN.m	
$M_{y.Ed}/M_{c.y.Rd}$	28.604 / 64.378 =	0.444	OK

Equivalent Uniform Moment Factor C1

C_1= fn(M_1, M_2, M_o, ~y,~m)	0.0, 0.0, 28.6, 0.774, 300.000	1.127	Uniform

Lateral Buckling Check M.b.Rd

Le = 1.2L+2D	1.2 x 3.869 + 2 x 0.203 =	5.049 m	
M_{cr} = Fn(C_1,L_e,I_z,I_t,I_w,E)	1.127, 5.049, 164.8, 7.019, 0.01537, 210000	34.313 kN.m	
$\lambda_{LT} = \sqrt{W.f_y/M_{cr}}$	$\sqrt{234.1 \times 275 / 34.313}$	1.370	
c_{LT} = Fn(λ_{LT}, λ_{LT}5950)	1.370, 1.427	0.488	Curve b
c_{LT}.mod = Fn($c_{LT},\lambda_{LT},k_c,f$)	0.488, 1.370, 0.942, 0.990	0.493	6.3.2.3
$M_{b.Rd} = c\ W_{pl.y}.f_y \leq M_{c.y.Rd}$	0.493 x 234.1 x 275 ≤ 64.378 =	31.717 kN.m	

$M_{y.Ed}/M_{b.Rd}$	28.604 / 31.717	0.902	OK

Deflection Check - Load Case 3

Deflection Limits (Internal Beams)	In-span δ ≤ 3869/360 = 10.7 mm Live (Case 2)	4.48 mm	OK
	In-span δ ≤ 3869/250 = 15.5 mm D+L (Case 3)	6.8 mm	OK

Provide Section :- **203 x 102 UB 23 (S275)**
Bearing :- **Beam to extend to rear of 330mm long x 102mm wide 3 Course Class B Engineering Brick Padstone.**

Member Loading

Load to B204 (Area loading included)

2nd Floor	7.20/2*0.60=2.16	7.20/2*2.0=7.20	
Total	**2.16 kN/m Dead**	**7.20 kN/m Live**	**(Unfactored)**

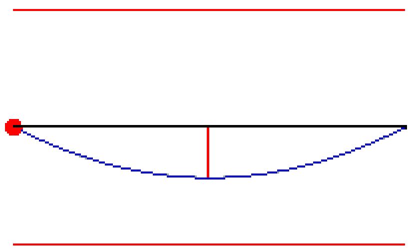
AXIAL WITH MOMENTS (MEMBER)

Beam 204

Member 6 (N.5-N.17) @ Level 1 in Load Case 1

Member Loading and Member Forces
Loading Combination : 1 UT + 1.35 D1 + 1.5 L1

D1 UDLY -001.102 (kN/m)
L1 UDLY -003.674 (kN/m)
D1 PTY2 -000.245 0.000 0.464 (kN,m,m)
D1 PTRY -001.056 0.464 3.370 -001.063
D1 PTY1 -000.020 3.370 3.408 (kN,m,m)
L1 PTY2 -000.817 0.000 0.464 (kN,m,m)
L1 PTRY -003.521 0.464 3.370 -003.542
L1 PTY1 -000.067 3.370 3.408 (kN,m,m)
D1 D 077.010 (kN/m³)



Member Forces in Load Case 1 and Maximum Deflection from Load Case 3

Span No.	Axial Force (kN)	Shear Force (kN) End 1	Shear Force (kN) End 2	Bending Moment (kNm) Node 1	Bending Moment (kNm) Node 2	Maximum Moment (kNm)	Maximum Deflection (mm @ m)
6	5	0.000	22.316	0.000	20.168	5.844	
17	0.000	-23.629	0.000	@ 1.722	@ 1.722		

Classification and Effective Area (EN 1993: 2006)

Section (19.04 kg/m)	178x102 UB 19 [S 275]		
Class = Fn(b/T,d/t,f_y,N,M_y,M_z)	6.41, 30.58, 275, 0, 20.17, 0	(Axial: Non-Slender)	Class 1
Auto Design Load Cases	1		

Moment Capacity Check M.c.y.Rd

$V_{y.Ed}/V_{pl.y.Rd}$	0.076 / 156.396 =	0	Low Shear
$M_{c.y.Rd} = f_y.W_{pl.y}/\gamma_{M0}$	275 x 171.3/1	47.108 kN.m	
$M_{y.Ed}/M_{c.y.Rd}$	20.166 / 47.108 =	0.428	OK

Equivalent Uniform Moment Factor C1

C_1= fn(M_1, M_2, M_o, ~y,~m)	0.0, 0.0, 20.1, 0.826, 300.000	1.127	Uniform

Lateral Buckling Check M.b.Rd

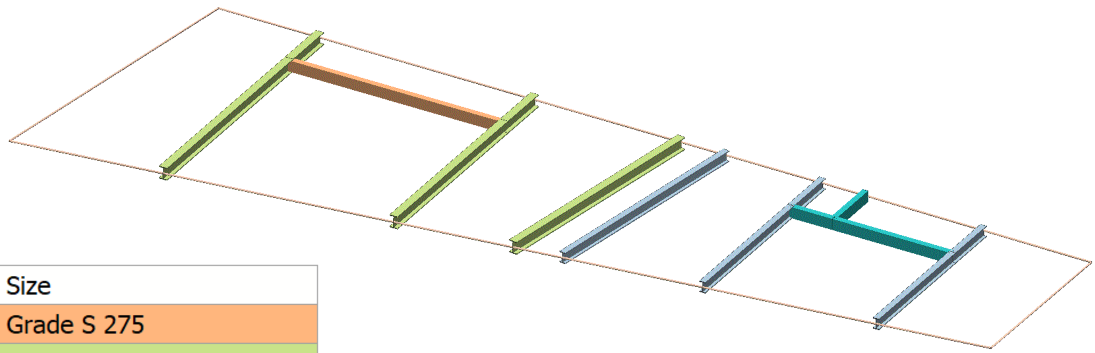
Le = 1.2L+2D	1.2 x 3.408 + 2 x 0.178 =	4.445 m	
M_{cr} = Fn(C_1,L_e,I_z,I_t,I_w,E)	1.127, 4.445, 137.6, 4.408, 0.009848, 210000	29.018 kN.m	
$\lambda_{LT} = \sqrt{W.f_y/M_{cr}}$	√ 171.3 x 275 / 29.018	1.274	
c_{LT} = Fn(λ_{LT}, λ_{LT}5950)	1.274, 1.319	0.538	Curve b
c_{LT}.mod = Fn(c_{LT},λ_{LT},k_c,f)	0.538, 1.274, 0.942, 0.984	0.546	6.3.2.3
$M_{b.Rd} = c\ W_{pl.y}.f_y \leq M_{c.y.Rd}$	0.546 x 171.3 x 275 ≤ 47.108 =	25.735 kN.m	
$M_{y.Ed}/M_{b.Rd}$	20.166 / 25.735	0.784	OK

Deflection Check - Load Case 2

Deflection Limits (Internal Beams)	In-span δ ≤ 3408/360 = 9.5 mm Live (Case 2)	4.41 mm	OK
	In-span δ ≤ 3408/250 = 13.6 mm D+L (Case 3)	5.84 mm	OK

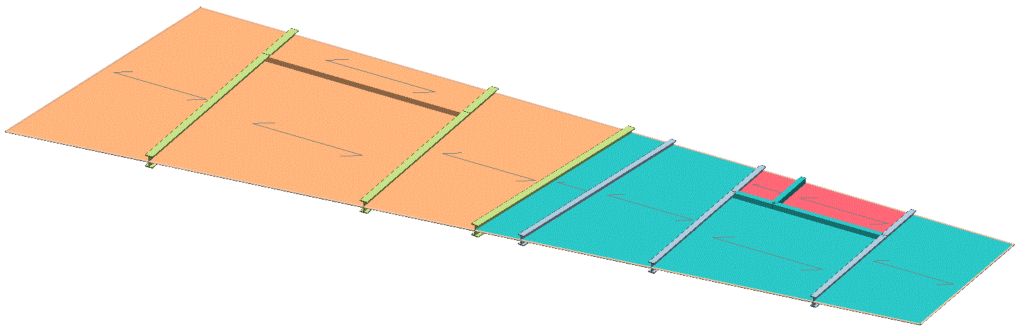
Provide Section :- **178 x 102 UB 19 (S275)**
Bearing :- **Beam to extend to rear of 330mm long x 102mm wide 3 Course Class B Engineering Brick Padstone.**

Steel Sections at Second floor structure over



Section Size
Default Grade S 275
203x133 UB 25
178x102 UB 19
94x175 C24
94x175 C24

Area Loads at Second floor structure over



Dead D	Live L
0.600	2.000
0.600	2.500
1.000	1.500

Member Loading

Load to B301 (Area loading included)

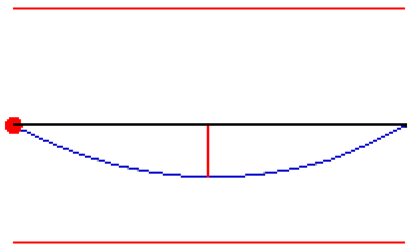
Joists from			
load bearing studs & roof	1.52	0.30	
3rd Floor	6.10/2*0.60=1.83	6.10/2*2.0=6.10	
Total	**3.35 kN/m Dead**	**6.40 kN/m Live**	**(Unfactored)**

AXIAL WITH MOMENTS (MEMBER)

Beam 301

Members 27-28 (N.2-N.7) @ Level 1 in Load Case 1

Member Loading and Member Forces
Loading Combination : 1 UT + 1.35 D1 + 1.5 L1



Part 1
D1 PTRY -000.713 0.000 3.686 -000.553
L1 PTRY -002.376 0.000 3.686 -001.844
D1 TRY -003.767 0.454 3.686 (kN,m,m)
L1 TRY -012.556 0.454 3.686 (kN,m,m)
D1 D 077.010 (kN/m³)
Part 2
D1 PTRY -000.713 0.000 3.686 -000.553
L1 PTRY -002.376 0.000 3.686 -001.844
D1 TRY -003.767 0.454 3.686 (kN,m,m)
L1 TRY -012.556 0.454 3.686 (kN,m,m)
D1 D 077.010 (kN/m³)

Member Forces in Load Case 1 and Maximum Deflection from Load Case 3							
Span No.	Axial Force (kN)	Shear Force (kN)		Bending Moment (kNm)		Maximum Moment (kNm)	Maximum Deflection (mm @ m)
		End 1	End 2	Node 1	Node 2		
	2	0.000	25.283	0.000	30.950	9.509	
13	0.000	-28.131	0.000	@ 2.328	@ 2.289		

Classification and Effective Area (EN 1993: 2006)

Section (25.09 kg/m)	203x133 UB 25 [S 275]		
Class = Fn(b/T,d/t,f_y,N,M_y,M_z)	8.54, 30.25, 275, 0, 30.94, 0	(Axial: Non-Slender)	Class 1
Auto Design Load Cases	1		

Moment Capacity Check M.c.y.Rd

$V_{y.Ed}/V_{pl.y.Rd}$	0.566 / 203.402 =	0.003	Low Shear
$M_{c.y.Rd} = f_y.W_{pl.y}/\gamma_{M0}$	275 x 257.7/1	70.868 kN.m	
$M_{y.Ed}/M_{c.y.Rd}$	30.928 / 70.868 =	0.436	OK

Equivalent Uniform Moment Factor C1

C_1= fn(M_1, M_2, M_o, ~y,~m)	0.0, 0.0, 30.9, 0.724, 300.000	1.127	Uniform

Lateral Buckling Check M.b.Rd

Le = 1.2L+2D	1.2 x 4.588 + 2 x 0.203 =	5.912 m	
M_{cr} = Fn(C_1,L_e,I_z,I_t,I_w,E)	1.127, 5.912, 308.5, 5.964, 0.02933, 210000	39.030 kN.m	
$\lambda_{LT} = \sqrt{W.f_y/M_{cr}}$	√ 257.7 x 275 / 39.03	1.347	
c_{LT} = Fn(λ_{LT}, λ_{LT}5950)	1.347, 1.377	0.499	Curve b
c_{LT}.mod = Fn(c_{LT},λ_{LT},k_c,f)	0.499, 1.347, 0.942, 0.988	0.505	6.3.2.3
$M_{b.Rd} = c\ W_{pl.y}.f_y \leq M_{c.y.Rd}$	0.505 x 257.7 x 275 ≤ 70.868 =	35.768 kN.m	
$M_{y.Ed}/M_{b.Rd}$	30.942 / 35.768	0.865	OK

Deflection Check - Load Case 2

Deflection Limits (Internal Beams)	In-span δ ≤ 4588/360 = 12.7 mm Live (Case 2)	7.04 mm	OK
	In-span δ ≤ 4588/250 = 18.4 mm D+L (Case 3)	9.51 mm	OK

Provide Section :- **203 x 133 UB 25 (S275)**
Bearing :- **Beam to extend to rear of 330mm long x 102mm wide 3 Course Class B Engineering Brick Padstone.**

Member Loading

Load to B302 (Area loading included)

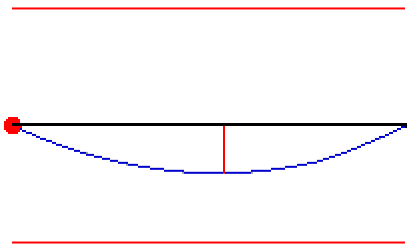
3rd Floor	6.10/2*0.60=1.83	6.10/2*2.0=6.10	
Total	**1.83 kN/m Dead**	**6.10 kN/m Live**	**(Unfactored)**

AXIAL WITH MOMENTS (MEMBER)

Beam 302

Members 31-32 (N.3-N.11) @ Level 1 in Load Case 1

Member Loading and Member Forces
Loading Combination : 1 UT + 1.35 D1 + 1.5 L1



Part 1
D1 UDLY -001.089 (kN/m)
L1 UDLY -003.630 (kN/m)
D1 PTY2 -000.066 0.000 0.235 (kN,m,m)
D1 PTRY -000.564 0.235 3.232 -000.703
L1 PTY2 -000.221 0.000 0.235 (kN,m,m)
L1 PTRY -001.879 0.235 3.232 -002.343
D1 D 077.010 (kN/m³)
Part 2
D1 UDLY -001.089 (kN/m)
L1 UDLY -003.630 (kN/m)
D1 PTY2 -000.066 0.000 0.235 (kN,m,m)
D1 PTRY -000.564 0.235 3.232 -000.703
L1 PTY2 -000.221 0.000 0.235 (kN,m,m)
L1 PTRY -001.879 0.235 3.232 -002.343
D1 D 077.010 (kN/m³)

Member Forces in Load Case 1 and Maximum Deflection from Load Case 3							
Span No.	Axial Force (kN)	Shear Force (kN)		Bending Moment (kNm)		Maximum Moment (kNm)	Maximum Deflection (mm @ m)
		End 1	End 2	Node 1	Node 2		
	3	0.000	23.429	0.000	25.936	7.173	
16	0.000	-26.841	0.000	@ 2.155	@ 2.085		

Classification and Effective Area (EN 1993: 2006)

Section (23.07 kg/m)	203x102 UB 23 [S 275]		
Class = Fn($b/T, d/t, f_y, N, M_y, M_z$)	5.47, 31.37, 275, 0, 25.93, 0	(Axial: Non-Slender)	Class 1
Auto Design Load Cases	1		

Moment Capacity Check M.c.y.Rd

$V_{y.Ed}/V_{pl.y.Rd}$	0.75 / 196.416 =	0.004	Low Shear
$M_{c.y.Rd} = f_y.W_{pl.y}/\gamma_{M0}$	275 x 234.1/1	64.378 kN.m	
$M_{y.Ed}/M_{c.y.Rd}$	25.905 / 64.378 =	0.402	OK

Equivalent Uniform Moment Factor C1

C_1= fn(M_1, M_2, M_o, ~y,~m)	0.0, 0.0, 25.9, 0.769, 300.000	1.127	Uniform

Lateral Buckling Check M.b.Rd

Le = 1.2L+2D	1.2 x 4.129 + 2 x 0.203 =	5.361 m	
M_{cr} = Fn($C_1, L_e, I_z, I_t, I_w, E$)	1.127, 5.361, 164.8, 7.019, 0.01537, 210000	31.985 kN.m	
$\lambda_{LT} = \sqrt{W.f_y/M_{cr}}$	√ 234.1 x 275 / 31.985	1.419	
c_{LT} = Fn(λ_{LT}, λ_{LT}5950)	1.419, 1.478	0.464	Curve b
c_{LT}.mod = Fn($c_{LT}, \lambda_{LT}, k_c, f$)	0.464, 1.419, 0.942, 0.993	0.467	6.3.2.3
$M_{b.Rd} = c\ W_{pl.y}.f_y \leq M_{c.y.Rd}$	0.467 x 234.1 x 275 ≤ 64.378 =	30.069 kN.m	
$M_{y.Ed}/M_{b.Rd}$	25.928 / 30.069	0.862	OK

Deflection Check - Load Case 2

Deflection Limits (Internal Beams)	In-span δ ≤ 4129/360 = 11.5 mm Live (Case 2)	5.32 mm	OK
	In-span δ ≤ 4129/250 = 16.5 mm D+L (Case 3)	7.17 mm	OK

Provide Section :- **203 x 102 UB 23 (S275)**
Bearing :- **Beam to extend to rear of 330mm long x 102mm wide 3 Course Class B Engineering Brick Padstone.**

Member Loading

Load to B303 (Area loading included)

Rafters	3.50*1.2/2=2.1	0.75*1.2/2=0.45	
Flat mansard roof	0.60*0.80/2=0.24	0.75*0.80/2=0.30	
Timber wall	1.50		
3rd Floor	3.25/2*0.60=0.98	3.25/2*2.0=3.25	
Total	**4.82 kN/m Dead**	**4.00 kN/m Live**	**(Unfactored)**

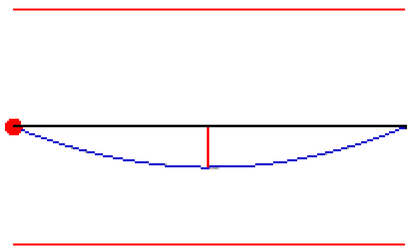
AXIAL WITH MOMENTS (MEMBER)

Beam 303

Member 26 (N.4-N.18) @ Level 1 in Load Case 1

Member Loading and Member Forces
Loading Combination : 1 UT + 1.35 D1 + 1.5 L1

D1 PTRY -000.557 0.000 3.937 -000.736
L1 PTRY -001.857 0.000 3.937 -002.452
D1 TRY -000.883 0.098 3.937 (kN,m,m)
L1 TRY -003.679 0.098 3.937 (kN,m,m)
D1 D 077.010 (kN/m^3)
D1 UDLY -003.840 (kN/m)
L1 UDLY -000.750 (kN/m)



Member Forces in Load Case 1 and Maximum Deflection from Load Case 3							
Span No.	Axial Force (kN)	Shear Force (kN)		Bending Moment (kNm)		Maximum Moment (kNm)	Maximum Deflection (mm @ m)
		End 1	End 2	Node 1	Node 2		
26	4	0.000	24.096	0.000	24.167	5.606	
18	0.000	-24.922	0.000	@ 1.989	@ 1.989		

Classification and Effective Area (EN 1993: 2006)

Section (25.09 kg/m)	203x133 UB 25 [S 275]		
Class = Fn(b/T,d/t,f_y,N,M_y,M_z)	8.54, 30.25, 275, 0, 24.16, 0	(Axial: Non-Slender)	Class 1
Auto Design Load Cases	1		

Moment Capacity Check M.c.y.Rd

$V_{y.Ed}/V_{pl.y.Rd}$	0.189 / 203.402 =	0.001	Low Shear
$M_{c.y.Rd} = f_y.W_{pl.y}/\gamma_{M0}$	275 x 257.7/1	70.868 kN.m	
$M_{y.Ed}/M_{c.y.Rd}$	24.161 / 70.868 =	0.341	OK

Equivalent Uniform Moment Factor C1

C_1= fn(M_1, M_2, M_o, ~y,~m)	0.0, 0.0, 24.1, 0.432, 300.000	1.127	Uniform

Lateral Buckling Check M.b.Rd

Le = 1.2L+2D	1.2 x 3.937 + 2 x 0.203 =	5.131 m	
M_{cr} = Fn(C_1,L_e,I_z,I_t,I_w,E)	1.127, 5.131, 308.5, 5.964, 0.02933, 210000	46.887 kN.m	
$\lambda_{LT} = \sqrt{W.f_y/M_{cr}}$	$\sqrt{257.7 \times 275 / 46.887}$	1.229	
c_{LT} = Fn(λ_{LT}, λ_{LT}5950)	1.229, 1.256	0.562	Curve b
c_{LT}.mod = Fn(c_{LT},λ_{LT},k_c,f)	0.562, 1.229, 0.942, 0.982	0.573	6.3.2.3
$M_{b.Rd} = c\ W_{pl.y}.f_y \leq M_{c.y.Rd}$	0.573 x 257.7 x 275 ≤ 70.868 =	40.601 kN.m	
$M_{y.Ed}/M_{b.Rd}$	24.164 / 40.601	0.595	OK

Deflection Check - Load Case 3

Deflection Limits (Internal Beams)	In-span δ ≤ 3937/360 = 10.9 mm Live (Case 2)	2.45 mm	OK
	In-span δ ≤ 3937/250 = 15.7 mm D+L (Case 3)	5.61 mm	OK

Provide Section :- **203 x 133 UB 25 (S275) to support external studwall & rafters**

Bearing :- **Beam to extend to rear of 330mm long x 102mm wide 3 Course Class B Engineering Brick Padstone.**

Member Loading

Load to B304

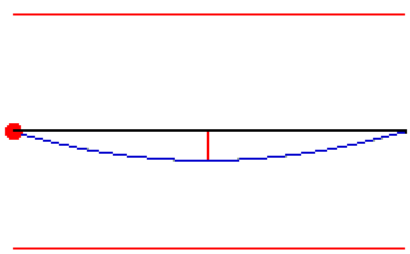
Terrace joists	3.0/2*0.60=0.90	3.0/2*2.50=3.75	
Total	**2.50 kN/m Dead**	**3.75 kN/m Live**	**(Unfactored)**

AXIAL WITH MOMENTS (MEMBER)

Beam 304

Member 25 (N.5-N.19) @ Level 1 in Load Case 1

Member Loading and Member Forces
Loading Combination : 1 UT + 1.35 D1 + 1.5 L1



D1 PTY2 -000.090 0.000 0.279 (kN,m,m)
D1 PTRY -000.641 0.279 2.893 -000.524
D1 PTRY -000.524 2.893 3.837 -000.482
L1 PTY2 -000.373 0.000 0.279 (kN,m,m)
L1 PTRY -002.672 0.279 2.893 -002.184
L1 PTRY -002.184 2.893 3.837 -002.008
D1 UDLY -000.225 (kN/m)
L1 UDLY -000.938 (kN/m)
D1 D 077.010 (kN/m^3)

Member Forces in Load Case 1 and Maximum Deflection from Load Case 3

Span No.	Axial Force (kN)	Shear Force (kN)		Bending Moment (kNm)		Maximum Moment (kNm)	Maximum Deflection (mm @ m)
		End 1	End 2	Node 1	Node 2		
25	5	0.000	11.787	0.000	11.525	4.228	
19	0.000	-11.610	0.000	@ 1.877	@ 1.919		

Classification and Effective Area (EN 1993: 2006)

Section (19.04 kg/m)	178x102 UB 19 [S 275]		
Class = Fn(b/T,d/t,f_y,N,M_y,M_z)	6.41, 30.58, 275, 0, 11.53, 0	(Axial: Non-Slender)	Class 1
Auto Design Load Cases	1		

Moment Capacity Check M.c.y.Rd

$V_{y.Ed}/V_{pl.y.Rd}$	0.189 / 156.396 =	0.001	Low Shear
$M_{c.y.Rd} = f_y.W_{pl.y}/\gamma_{M0}$	275 x 171.3/1	47.108 kN.m	
$M_{y.Ed}/M_{c.y.Rd}$	11.523 / 47.108 =	0.245	OK

Equivalent Uniform Moment Factor C1

C_1= fn(M_1, M_2, M_o, ~y,~m)	0.0, 0.0, 11.5, 0.467, 300.000	1.127	Uniform

Lateral Buckling Check M.b.Rd

Le = 1.2L+2D	1.2 x 3.837 + 2 x 0.178 =	4.96 m	
M_{cr} = Fn(C_1,L_e,I_z,I_t,I_w,E)	1.127, 4.960, 137.6, 4.408, 0.009848, 210000	25.425 kN.m	
$\lambda_{LT} = \sqrt{W.f_y/M_{cr}}$	$\sqrt{171.3 \times 275 / 25.425}$	1.361	
c_{LT} = Fn(λ_{LT}, λ_{LT}5950)	1.361, 1.410	0.492	Curve b
c_{LT}.mod = Fn(c_{LT},λ_{LT},k_c,f)	0.492, 1.361, 0.942, 0.989	0.497	6.3.2.3
$M_{b.Rd} = c\, W_{pl.y}.f_y \le M_{c.y.Rd}$	0.497 x 171.3 x 275 ≤ 47.108 =	23.425 kN.m	
$M_{y.Ed}/M_{b.Rd}$	11.524 / 23.425	0.492	OK

Deflection Check - Load Case 2

Deflection Limits (Internal Beams)	In-span δ ≤ 3837/360 = 10.7 mm Live (Case 2)	3.26 mm	OK
	In-span δ ≤ 3837/250 = 15.3 mm D+L (Case 3)	4.23 mm	OK

Provide Section :- 178 x 102 UB 19 (S275)
Bearing :- Beam to extend to rear of 215mm long x 102mm wide 2 Course Class B Engineering Brick Padstone.

Member Loading

Load to B305 & B306 (Area Loads included) – worst case

Terrace joists	4.95/2*0.60=1.485	4.95/2*2.50=6.19	
Total	**1.485 kN/m Dead**	**6.19 kN/m Live**	**(Unfactored)**

AXIAL WITH MOMENTS (MEMBER)

Beam 305 & Beam 306

Members 23-24 (N.6-N.12) @ Level 1 in Load Case 1

Member Loading and Member Forces

```
Loading Combination : 1 UT + 1.35 D1 + 1.5 L1

Part 1
D1 TRY  -001.979 0.342 2.584 (kN,m,m)
L1 TRY  -008.246 0.342 2.584 (kN,m,m)
D1 PTRY -000.649 0.000 2.584 -000.530
L1 PTRY -002.703 0.000 2.584 -002.210
D1 D     077.010             ( kN/m³)
Part 2
D1 TRY  -001.979 0.342 2.584 (kN,m,m)
L1 TRY  -008.246 0.342 2.584 (kN,m,m)
D1 PTRY -000.649 0.000 2.584 -000.530
L1 PTRY -002.703 0.000 2.584 -002.210
D1 D     077.010             ( kN/m³)
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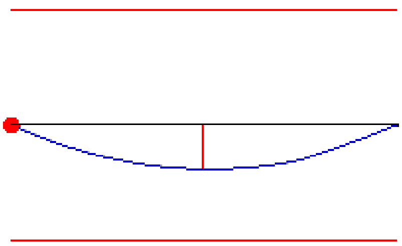


Member Forces in Load Case 1 and Maximum Deflection from Load Case 3

Mem ber No.	Node End1 End2	Torque Moment (kN.m)	Shear Force (kN)	Bending Moment (kN.m)	Maximum Moment (kN.m @ m)	Maximum Deflection (mm @ m)
	6	0.000	19.050	0.000	17.931	5.545
	20	0.000	-18.353	0.000	@ 1.822	@ 1.768

Classification and Effective Area (EN 1993: 2006)

Section (19.04 kg/m)	178x102 UB 19 [S 275]		
Class = Fn($b/T, d/t, f_y, N, M_y, M_z$)	6.41, 30.58, 275, 0, 17.93, 0	(Axial: Non-Slender)	Class 1
Auto Design Load Cases	1		

Moment Capacity Check M.c.y.Rd

$V_{y.Ed}/V_{pl.y.Rd}$	0.588 / 156.396 =	0.004	Low Shear
$M_{c.y.Rd} = f_y.W_{pl.y}/\gamma_{M0}$	275 x 171.3/1	47.108 kN.m	
$M_{y.Ed}/M_{c.y.Rd}$	17.909 / 47.108 =	0.380	OK

Equivalent Uniform Moment Factor C1

C_1= fn(M_1, M_2, M_o, ψ,μ)	0.0, 0.0, 17.9, 0.882, 300.000	1.127	Uniform

Lateral Buckling Check M.b.Rd

Le = 1.2L+2D	1.2 x 3.515 + 2 x 0.178 =	4.574 m	
M_{cr}= Fn(C_1,L_e,I_z,I_t,I_w,E)	1.127, 4.574, 137.6, 4.408, 0.009848, 210000	28.028 kN.m	
$\lambda_{LT} = \sqrt{W.f_y/M_{cr}}$	$\sqrt{171.3 \times 275 / 28.028}$	1.296	
χ_{LT}= Fn(λ_{LT}, λ_{LT5950})	1.296, 1.342	0.526	Curve b
χ_{LT}.mod = Fn(χ_{LT},λ_{LT},k_c,f)	0.526, 1.296, 0.942, 0.985	0.533	6.3.2.3
$M_{b.Rd} = \chi\, W_{pl.y}.f_y \leq M_{c.y.Rd}$	0.533 x 171.3 x 275 ≤ 47.108 =	25.125 kN.m	
$M_{y.Ed}/M_{b.Rd}$	17.926 / 25.125	0.713	OK

Deflection Check - Load Case 2

Deflection Limits (Internal Beams)	In-span $\delta \leq$ 3515/360 = 9.8 mm Live (Case 2)	4.17 mm	OK
	In-span $\delta \leq$ 3515/250 = 14.1 mm D+L (Case 3)	5.54 mm	OK

Provide Section :- **178 x 102 UB 19 (S275)**

Bearing :- **Beam to extend to rear of 330mm long x 102mm wide 3 Course Class B Engineering Brick Padstone.**

Member Loading

Load to B401/403

Roof rafters	1.0/2*3.50=1.75	1.0/2*0.75=0.375	
Timber wall	1.50		
Flat roof	3.20/2*0.60=0.96	3.20/2*0.75=1.20	
Flat roof (windows)	0.80/2*0.60=0.24	0.80/2*0.75=0.30	
Total	**4.80 kN/m Dead**	**1.95 kN/m Live**	**(Unfactored)**

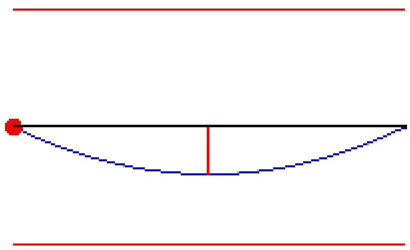
AXIAL WITH MOMENTS (MEMBER)

Beam 401 & 403

Member 1 (N.1-N.2) @ Level 1 in Load Case 1

Member Loading and Member Forces
Loading Combination : 1 UT + 1.35 D1 + 1.5 L1

D1 UDLY -004.800 (kN/m)
L1 UDLY -001.950 (kN/m)



Member Forces in Load Case 1 and Maximum Deflection from Load Case 3							
Span No.	Axial Force (kN)	Shear Force (kN)		Bending Moment (kNm)		Maximum Moment (kNm)	Maximum Deflection (mm @ m)
		End 1	End 2	Node 1	Node 2		
1	1	0.000C	23.507	0.000	28.355	10.048	
2	0.000C	-23.507	0.000	@ 2.413	@ 2.413		

Classification and Effective Area (EN 1993: 2006)

Section (25.09 kg/m)	203x133 UB 25 [S 275]		
Class = Fn($b/T, d/t, f_y, N, M_y, M_z$)	8.54, 30.25, 275, 0, 28.35, 0	(Axial: Non-Slender)	Class 1
Auto Design Load Cases	1		

Moment Capacity Check M.c.y.Rd

$V_{y.Ed}/V_{pl.y.Rd}$	0.003 / 203.402 =	0	Low Shear
$M_{c.y.Rd} = f_y.W_{pl.y}/\gamma_{M0}$	275 x 257.7/1	70.868 kN.m	
$M_{y.Ed}/M_{c.y.Rd}$	28.35 / 70.868 =	0.400	OK

Equivalent Uniform Moment Factor C1

C_1= fn(M_1, M_2, M_o, ~y,~m)	0.0, 0.0, 28.3, 0.913, 300.000	1.127	Uniform

Lateral Buckling Check M.b.Rd

Le = 1.2L+2D	1.2 x 4.825 + 2 x 0.203 =	6.196 m	
M_{cr} = Fn($C_1, L_e, I_z, I_t, I_w, E$)	1.127, 6.196, 308.5, 5.964, 0.02933, 210000	36.793 kN.m	
$\lambda_{LT} = \sqrt{W.f_y/M_{cr}}$	$\sqrt{257.7 \times 275 / 36.793}$	1.388	
c_{LT} = Fn(λ_{LT}, λ_{LT}5950)	1.388, 1.418	0.479	Curve b
c_{LT}.mod = Fn($c_{LT}, \lambda_{LT}, k_c, f$)	0.479, 1.388, 0.942, 0.991	0.483	6.3.2.3
$M_{b.Rd} = c\, W_{pl.y}.f_y \leq M_{c.y.Rd}$	0.483 x 257.7 x 275 ≤ 70.868 =	34.233 kN.m	
$M_{y.Ed}/M_{b.Rd}$	28.35 / 34.233	0.828	OK

Deflection Check - Load Case 3

Deflection Limits (Internal Beams)	In-span δ ≤ 4825/360 = 13.4 mm Live (Case 2)	2.8 mm	OK
	In-span δ ≤ 4825/250 = 19.3 mm D+L (Case 3)	10.05 mm	OK

Provide Section :- **203 x 133 UB 25 (S275)**
Bearing :- **Beam to extend to rear of 330mm long x 102mm wide 2 Course Class B Engineering Brick Padstone.**

Member Loading

Load to B402

Flat roof	6.50/2*0.60=1.95	6.50/2*0.75=2.44	
Total	**1.95 kN/m Dead**	**2.44 kN/m Live**	**(Unfactored)**

AXIAL WITH MOMENTS (MEMBER)

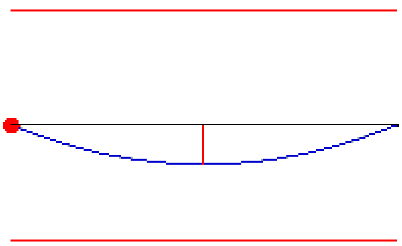
Beam 402

Member 1 (N.1-N.2) @ Level 1 in Load Case 1

Member Loading and Member Forces

```
Loading Combination : 1 UT + 1.35 D1 + 1.5 L1

D1 UDLY -001.950          ( kN/m )
L1 UDLY -002.440          ( kN/m )
```



Member Forces in Load Case 1 and Maximum Deflection from Load Case 3						
Mem ber No.	Node End1 End2	Axial Force (kN)	Shear Force (kN)	Bending Moment (kN.m)	Maximum Moment (kN.m @ m)	Maximum Deflection (mm @ m)
1	1	0.000C	14.409	0.000	15.850	7.844
	2	0.000C	-14.409	0.000	@ 2.200	@ 2.200

Classification and Effective Area (EN 1993: 2006)

Section (19.04 kg/m)	178x102 UB 19 [S 275]		
Class = Fn(b/T,d/t,f_y,N,M_y,M_z)	6.41, 30.58, 275, 0, 15.85, 0	(Axial: Non-Slender)	Class 1
Auto Design Load Cases	1		

Moment Capacity Check M.c.y.Rd

$V_{y.Ed}/V_{pl.y.Rd}$	0.002 / 156.396 =	0	Low Shear
$M_{c.y.Rd}$ = $f_y.W_{pl.y}/\gamma_{M0}$	275 x 171.3/1	47.108 kN.m	
$M_{y.Ed}/M_{c.y.Rd}$	15.848 / 47.108 =	0.336	OK

Equivalent Uniform Moment Factor C1

C_1= fn(M_1, M_2, M_o, ψ,μ)	0.0, 0.0, 15.8, 0.800, 300.000	1.127	Uniform

Lateral Buckling Check M.b.Rd

Le = 1.2L+2D	1.2 x 4.4 + 2 x 0.178 =	5.636 m	
M_{cr}= Fn(C_1,L_e,I_z,I_t,I_w,E)	1.127, 5.636, 137.6, 4.408, 0.009848, 210000	21.895 kN.m	
λ_{LT}= $\sqrt{}$ W.f_y/M_{cr}	$\sqrt{}$ 171.3 x 275 / 21.895	1.467	
χ_{LT}= Fn(λ_{LT}, λ_{LT5950})	1.467, 1.519	0.442	Curve b
χ_{LT}.mod = Fn(χ_{LT},λ_{LT},k_c,f)	0.442, 1.467, 0.942, 0.997	0.443	6.3.2.3
$M_{b.Rd}$ = χ $W_{pl.y}.f_y \leq M_{c.y.Rd}$	0.443 x 171.3 x 275 $\leq$ 47.108 =	20.879 kN.m	
$M_{y.Ed}/M_{b.Rd}$	15.848 / 20.879	0.759	OK

Deflection Check - Load Case 3

Deflection Limits (Internal Beams)	In-span $\delta \leq$ 4400/360 = 12.2 mm Live (Case 2)	4.18 mm	OK
	In-span $\delta \leq$ 4400/250 = 17.6 mm D+L (Case 3)	7.84 mm	OK

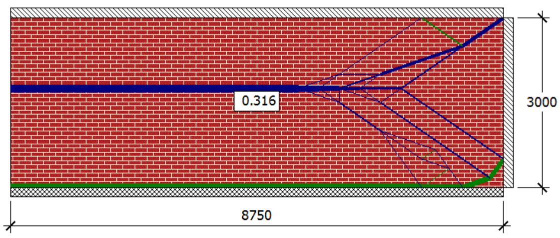
Provide Section :- **178 x 102 UB 19 (S275)**

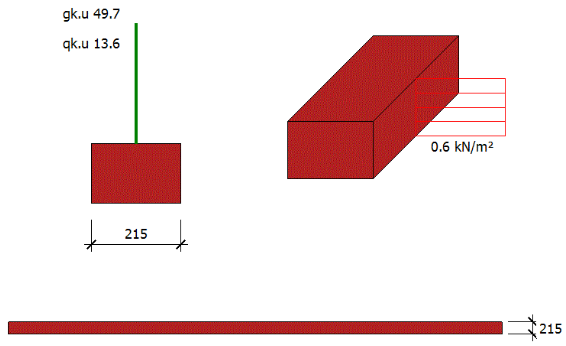
Bearing :- **Beam to extend to rear of 215mm long x 102mm wide 2 Course Class B Engineering Brick Padstone.**

TWO WAY SPANNING, VERTICALLY AND LATERALLY LOADED, SINGLE-LEAF WALL

DESIGN TO BS EN 1996-1-1:2005

Masonry wall check to resist lateral loads





Summary of Design Data

EuroCode National Annex	Using UK values:A1 2012		
Wall Dimensions	h=3.000 m, hef=2.961 m (Eqn. 5.6), L=8.750 m, Lef=8.750 m		
Support Conditions	Bottom Cont., Top Simple, Left Free, Right Simple		
Lateral Loads	Wx=0.6 kN/m²		
Single-leaf Wall (mm)	t=215, tef=215		
Limiting Dimensions	λ=13.8<=λ_{lim}=27, L/t_{ef}=40.7, H/t_{ef}=14, Hence H/t_{ef}<=49.3	0.510	OK

Wall Design

Partial Safety Factor (γmc/γmf)	Units Category II, Execution Control Class 2	3/2.7	Table NA.1
Material	Clay bricks with water absorption over 12%		
Units and Mortar Strength	fb = 5 N/mm², fm = Mortar designation M4/(iii)		
Compressive Strength (f_k)	Group 2, γ=20 kN/m³	1.87 N/mm²	Table NA.4
Loads from above	Dead Load=49.7 kN/m, Live Load=13.6 kN/m		
Section Properties	Area=2150 cm²/m, Zp=7704 cm³/m		
Flexural Strength f_{xk2} (Perpendicular)		0.9 N/mm²	Table NA.6
Flexural Strength f_{xk1} (Parallel)	f_{xk1}=0.3, gd=0.261 N/mm²		
	f_{xk1}=f_{xk1}+min(gd, 0.2•f_k/γmc)γmf	0.637 N/mm²	Table NA.6
Critical axail compressive case	1.35(γ.tk.h+gku)+1.05qku		
Max local stress @	X=0 m,Y=1.5 m < fk/γ_{mc}	0.46 N/mm²	OK
Critical axial buckling case	1.35(γ.tk.h+gku)+1.05qku		
Max axial buckling force @	X=4.375 m, Y=1.5 m averaged over width of 2.15 m	98.79kN/m	
Moments from Lateral Load	$M_{wx,top}$=0.000 kN.m, $M_{wx,mid}$=0.000 kN.m		
Capacity reduction factor top, ~F	ex=0.0 mm, hef=2961 mm, tef=215.0 mm, t=215.0 mm	0.900	
Capacity reduction factor mid, ~F_m	Creep coef. =1.5, ehm = 0.000 mm, h_{ef} = 2.961	0.771	
Fr=~F.f_k.tk/γmc	0.771x1.87x215/3	103.3 kN/m	
Fd/Fr	98.8/103.3	0.956	OK
Mr=f_{xk2}.Zp/γmf	0.9x7704/2.7	2.568 kN.m/m	
Mr=f_{xk1}.Zb/γmf	0.637x7704/2.7	1.817 kN.m/m	

Design for Lateral Loads

Design Lateral Load Wd	1.5 Wx	0.900 kN/m²	
Yield Line Analysis	Load Factor, λ_p	3.166	
Ut=1/λ_p	1 / 3.166	0.316	OK

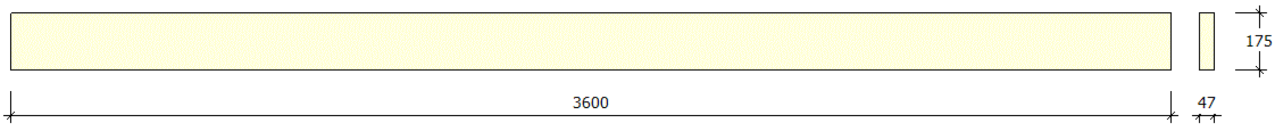
Existing solid masonry wall is adequate to resist lateral loads from the edge to the sway frame.

TIMBER DESIGN

MASTERKEY : TIMBER DESIGN

JOIST DESIGN TO BS 5268 : PART 2

Proposed Floor Joists



Summary Design Data

Section Size	b = 47, h = 175, 175x47 in Strength Class C24
Section Properties (c,cm^4,cm^3)	Area 82.3, Ix 2099.1, Z 239.9
Specification	1 : Internal use in continuously heated building Long Term loading, Load sharing
Joist data	Span 3.6 m, Spacing 0.4 m, Bearing length B 75, Distance to Bearing 150 mm
Joist loading	Dead load 0.6 kN/m², Live load 2.0 kN/m²

Grade and Admissible Stresses (Strength Class C24)

$\sigma_{m.adm} = K_2.K_3.K_7.K_8.\sigma_m$	1.00 x 1.00 x 1.06 x 1.10 x 7.50	8.75 N/mm²	
$\sigma_{c\perp.adm} = K_2.K_3.K_4.K_8.\sigma_{c\perp}$	1.00 x 1.00 x 1.14 x 1.10 x 1.90	2.38 N/mm²	
$\tau_{adm} = K_2.K_{3\_}.K_8.\tau$	1.00 x 1.00 x 1.10 x 0.71	0.78 N/mm²	
$E = K_2.E_m$ean	1.00 x 10800	10800 N/mm²	

Design Loads

Density and Selfweight	Timber Density 420 kg/m³, Fj 0.03 kN/m		
w = (Fd+Fl).s+Fj	(0.60 + 2.00) x 0.4 + 0.03	1.07 kN/m	

Bending Check

$M=w.L^2/8$	$1.07 \times 3.6^2 / 8$	1.733 kN.m	
$\sigma_{m.a} = M/Z$	$1.733 / 239.9 \le 8.75$	7.23 N/mm²	OK

Shear and Bearing Check

$Fv=w.L/2$	1.07 x 3.6 / 2	1.926 kN	
$\tau_a = 1.5\ Fv / Area$	$1.5 \times 1.926 / 82.25 \le 0.78$	0.35 N/mm²	OK
$\sigma_{c\perp a} = Fv / (b.B)$	$1.926 / (47 \times 75) \le 2.38$	0.55 N/mm²	OK

Deflection Check

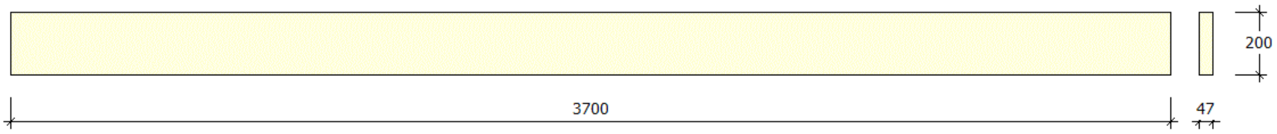
$\delta_m = 5.w.L^4/(384.E.Ix)$	$5 \times 1.07 \times 3.6^4 /(384 \times 10800 \times 2099.1)$	10.32 mm	
$\delta_s = 12.w.L^2 / (5.E.Area)$	$12 \times 1.07 \times 3.6^2 / (5 \times 10800 \times 82.3)$	0.37 mm	
$\delta = \delta_m + \delta_s$	$10.32 + 0.37 \le L/333$ or 14mm	10.70 mm	OK

Adopt floor joists 47x175mm C24 at 400mm c/c . Max span 3.60m

MASTERKEY : TIMBER DESIGN

JOIST DESIGN TO BS 5268 : PART 2

Proposed floor joists at Ground floor level (office use)



Summary Design Data

Section Size	b = 47, h = 200, 200x47 in Strength Class C24
Section Properties (c,cm^4,cm^3)	Area 94, Ix 3133.3, Z 313.3
Specification	1 : Internal use in continuously heated building
	Long Term loading, Load sharing
Joist data	Span 3.7 m, Spacing 0.4 m, Bearing length B 75, Distance to Bearing 150 mm
Joist loading	Dead load 0.6 kN/m^2, Live load 2.5 kN/m^2

Grade and Admissible Stresses (Strength Class C24)

$\sigma_{m.adm} = K_2.K_3.K_7.K_8.\sigma_m$	1.00 x 1.00 x 1.05 x 1.10 x 7.50	8.63 N/mm²	
$\sigma_{c\perp.adm} = K_2.K_3.K_4.K_8.\sigma_{c\perp}$	1.00 x 1.00 x 1.14 x 1.10 x 1.90	2.38 N/mm²	
$\tau_{adm} = K_2.K_{3\_}.K_8.\tau$	1.00 x 1.00 x 1.10 x 0.71	0.78 N/mm²	
$E = K_2.E_m$ean	1.00 x 10800	10800 N/mm²	

Design Loads

Density and Selfweight	Timber Density 420 kg/m³, Fj 0.04 kN/m		
w = (Fd+Fl).s+Fj	(0.60 + 2.50) x 0.4 + 0.04	1.28 kN/m	

Bending Check

$M=w.L^2/8$	1.28 x 3.7 ² / 8	2.190 kN.m	
$\sigma_{m.a}$ = M/Z	2.190 / 313.33 ≤ 8.63	6.99 N/mm²	OK

Shear and Bearing Check

Fv=w.L/2	1.28 x 3.7 / 2	2.368 kN	
τ_a = 1.5 Fv / Area	1.5 x 2.368 / 94 ≤ 0.78	0.38 N/mm²	OK
$\sigma_{c\perp a}$ = Fv / (b.B)	2.368 / (47 x 75) ≤ 2.38	0.67 N/mm²	OK

Deflection Check

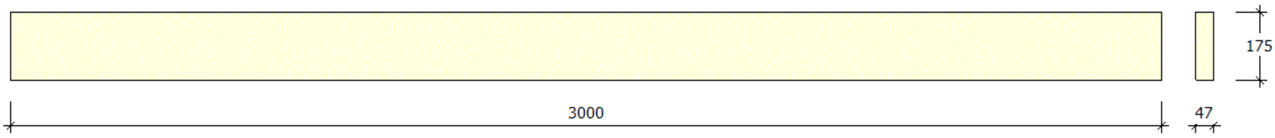
$\delta_m = 5.w.L^4/(384.E.Ix)$	5 x 1.28 x 3.7⁴ /(384 x 10800 x 3133.3)	9.23 mm	
$\delta_s = 12.w.L^2 / (5.E.Area)$	12 x 1.28 x 3.7² / (5 x 10800 x 94)	0.41 mm	
$\delta = \delta_m + \delta_s$	9.23 + 0.41 ≤ L/333 or 14mm	9.64 mm	OK

Adopt timber floor joists 47x200mm C24 at 400mm c/c at ground floor level for office use. Max span 3.70m

MASTERKEY : TIMBER DESIGN

JOIST DESIGN TO BS 5268 : PART 2

Terrace Joists at third floor level



Summary Design Data

Section Size	b = 47, h = 175, 175x47 in Strength Class C24
Section Properties (c,cm^4,cm^3)	Area 82.3, Ix 2099.1, Z 239.9
Specification	2 : Covered and generally unheat, Long Term loading Load sharing
Joist data	Span 3.0 m, Spacing 0.4 m, Bearing length B 75, Distance to Bearing 150 mm
Joist loading	Dead load 0.6 kN/m², Live load 2.5 kN/m²

Grade and Admissible Stresses (Strength Class C24)

$\sigma_{m.adm} = K_2.K_3.K_7.K_8.\sigma_m$	1.00 x 1.00 x 1.06 x 1.10 x 7.50	8.75 N/mm²	
$\sigma_{c\perp.adm} = K_2.K_3.K_4.K_8.\sigma_{c\perp}$	1.00 x 1.00 x 1.14 x 1.10 x 1.90	2.38 N/mm²	
$\tau_{adm} = K_2.K_{3\_}.K_8.\tau$	1.00 x 1.00 x 1.10 x 0.71	0.78 N/mm²	
$E = K_2.E_m ean$	1.00 x 10800	10800 N/mm²	

Design Loads

Density and Selfweight	Timber Density 420 kg/m³, Fj 0.03 kN/m		
w = (Fd+Fl).s+Fj	(0.60 + 2.50) x 0.4 + 0.03	1.27 kN/m	

Bending Check

$M=w.L^2/8$	$1.27 \times 3.0\ ^2 / 8$	1.429 kN.m	
$\sigma_{m.a} = M/Z$	$1.429 / 239.9 \le 8.75$	5.96 N/mm²	OK

Shear and Bearing Check

$Fv=w.L/2$	1.27 x 3.0 / 2	1.905 kN	
$\tau_a = 1.5\ Fv / Area$	$1.5 \times 1.905 / 82.25 \le 0.78$	0.35 N/mm²	OK
$\sigma_{c\perp a} = Fv / (b.B)$	$1.905 / (47 \times 75) \le 2.38$	0.54 N/mm²	OK

Deflection Check

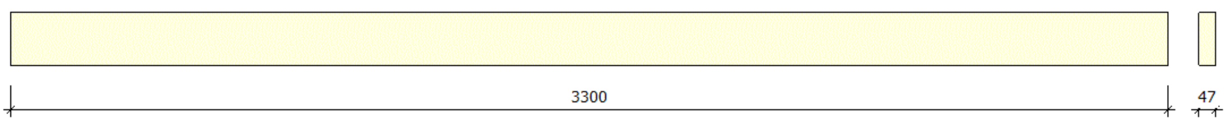
$\delta_m = 5.w.L^4/(384.E.Ix)$	$5 \times 1.27 \times 3.0^4 /(384 \times 10800 \times 2099.1)$	5.91 mm	
$\delta_s = 12.w.L^2 / (5.E.Area)$	$12 \times 1.27 \times 3.0^2 / (5 \times 10800 \times 82.3)$	0.31 mm	
$\delta = \delta_m + \delta_s$	$5.91 + 0.31 \le L/333$ or 14mm	6.22 mm	OK

Adopt terrace floor joists 47x175mm C24 at 400mm c/c . Max span 3.00m

MASTERKEY : TIMBER DESIGN

JOIST DESIGN TO BS 5268 : PART 2

Flat roof Joists at Roof Level



Summary Design Data

Section Size	b = 47, h = 150, 150x47 in Strength Class C24		
Section Properties (c,cm^4,cm^3)	Area 70.5, Ix 1321.9, Z 176.3		
Specification	2 : Covered and generally unheat, Long Term loading Load sharing		
Joist data	Span 3.3 m, Spacing 0.4 m, Bearing length B 75, Distance to Bearing 150 mm		
Joist loading	Dead load 0.6 kN/m², Live load 0.75 kN/m²		

Grade and Admissible Stresses (Strength Class C24)

$\sigma_{m.adm} = K_2.K_3.K_7.K_8.\sigma_m$	1.00 x 1.00 x 1.08 x 1.10 x 7.50	8.90 N/mm²	
$\sigma_{c\perp.adm} = K_2.K_3.K_4.K_8.\sigma_{c\perp}$	1.00 x 1.00 x 1.14 x 1.10 x 1.90	2.38 N/mm²	
$\tau_{adm} = K_2.K_{3\_}.K_8.\tau$	1.00 x 1.00 x 1.10 x 0.71	0.78 N/mm²	
$E = K_2.E_m$ean	1.00 x 10800	10800 N/mm²	

Design Loads

Density and Selfweight	Timber Density 420 kg/m³, Fj 0.03 kN/m		
w = (Fd+Fl).s+Fj	(0.60 + 0.75) x 0.4 + 0.03	0.57 kN/m	

Bending Check

$M=w.L^2/8$	$0.57 \times 3.3\ ^2 / 8$	0.776 kN.m	
$\sigma_{m.a} = M/Z$	$0.776 / 176.25 \leq 8.90$	4.40 N/mm²	OK

Shear and Bearing Check

Fv=w.L/2	0.57 x 3.3 / 2	0.941 kN	
$\tau_a = 1.5\ Fv / Area$	$1.5 \times 0.941 / 70.5 \leq 0.78$	0.20 N/mm²	OK
$\sigma_{c\perp a} = Fv / (b.B)$	$0.941 / (47 \times 75) \leq 2.38$	0.27 N/mm²	OK

Deflection Check

$\delta_m = 5.w.L^4/(384.E.Ix)$	$5 \times 0.57 \times 3.3^4 /(384 \times 10800 \times 1321.9)$	6.17 mm	
$\delta_s = 12.w.L^2 / (5.E.Area)$	$12 \times 0.57 \times 3.3^2 / (5 \times 10800 \times 70.5)$	0.20 mm	
$\delta = \delta_m + \delta_s$	$6.17 + 0.20 \leq L/333$ or 14mm	6.36 mm	OK

Adopt flat roof joists 47x150mm C24 at 400mm c/c . Max span 3.30m

Member Loading

Load to floor joists

Timber studwall	1.01		
Mansard roof	1.0/2*3.50=1.75	1.0/2*0.75=0375	
Flat roof	1.0/2*0.60=0.30	1.0/2*0.75=0375	
Total	**3.06 kN/m Dead**	**0.75 kN/m Live**	**(Unfactored)**

MASTERKEY : TIMBER DESIGN

AXIAL LOAD WITH MOMENT DESIGN TO BS EN 1995-1-1:2004 + A1:2008

New Floor Joists check to support mansard roof at third floor level

Members 1-2 (N.1-N.3) @ Level 1

Summary Design Data

Eurocode National Annex	Using UK values
Strength class code	BS EN 338:2009
Design Cases Covered	1.35 D1 + 1.5 L1
Deflection Cases Covered	1.0 L1, 1.0 D1 + 1.0 L1
Section Size	b = 47, h = 175, 175x47 in Strength Class C24
Section Properties (c,cm³,cm)	Area 82.3, $W_{el.y}$ 239.9, $W_{el.z}$ 64.4, i_y 5.05, i_z 1.36
Specification	1 : Internal use in continuously heated building Long Term loading
Integrated Design	Critical Case 1
Member Details	N_{Ed} = -0.017 kN, L = 2.4 m, L_y = 2.4 m, L_z = 1.5 m, $L_{cr.y}$ = 1.0 L_y , $L_{cr.z}$ = 1.0 L_z Bearing length 75, Distance to Bearing 150 mm

Grade and Admissible Stresses (Strength Class C24)

$f_{m.y.d} = K_{mod}.K_{hy}.K_{sys}.f_{m.k}/\gamma_m$	0.70 x 1.00 x 1.10 x 24.00/1.3	14.22 N/mm²	
$f_{m.z.d} = K_{mod}.K_{hz}.K_{sys}.f_{m.k}/\gamma_m$	0.70 x 1.26 x 1.10 x 24.00/1.3	17.93 N/mm²	
$f_{t.0.d} = K_{mod}.K_h.K_{sys}.f_{t.0.k}/\gamma_m$	0.70 x 1.00 x 1.10 x 14.00/1.3	8.29 N/mm²	
$f_{c.90.d} = K_{mod}.K_{c.90}.K_{sys}.f_{c.90.k}/\gamma_m$	0.70 x 1.50 x 1.10 x 2.50/1.3	2.22 N/mm²	
$f_{v.d} = K_{mod}.K_{sys}.f_{v.k}/\gamma_m$	0.70 x 1.10 x 4.00/1.3	2.37 N/mm²	
E_{m}ean	Instantaneous Deflection	11000 N/mm²	Deflection

Tensile Resistance

$\sigma_{t.a} = N_{Ed}$ /Area	0.017 / 82.25 ≤ 8.29	0.00 N/mm²	OK

Axial Load with Moments Check

Critical Design Location	X = 0.937		
$\sigma_{m.y.d} = M_y/W_{el.y}$	1.710 / 239.9 ≤ 14.22	7.13 N/mm²	OK
$U_t = \sigma_{c.0.d}/f_{t.0.d}$	0.002/8.292	0.000	OK
$U_{m.y} = \sigma_{m.y.d}/f_{m.y.d}$	7.130/14.215	0.502	OK
$U_t+U_{m.y}$	0.000+0.502	0.502	OK

Shear and Bearing Check

Critical Design Location	X = 0.000		
$\tau_a = 1.5\ V_{y.Ed}$ / Area / k_{cr}	1.5 x 3.696 / 82.25 / 0.67 ≤ 2.37	1.01 N/mm²	OK
$\sigma_{c\perp ax} = V_{y.Ed}$ / (b.l_y)	3.696 / (47 x 75) ≤ 2.22	1.05 N/mm²	OK

Deflection Check (Shear Deflection NOT Included)

Critical Load Case 003 : Dead Plus Live (Serviceability)

$\delta = \delta_m$	In-span 3.15 ≤ L/250	3.15 mm	OK

Adopt timber floor joists min. 47x175mm C24 at 400mm c/c to support mansard roof over. Max span 2.40m

MASTERKEY : TIMBER DESIGN

AXIAL LOAD WITH MOMENT DESIGN TO BS EN 1995-1-1:2004 + A1:2008

Roof trimmers adjacent to windows

Members 50-52 (N.2-N.6) @ Level 1

Summary Design Data

Eurocode National Annex	Using UK values
Strength class code	BS EN 338:2009
Design Cases Covered	1.35 D1 + 1.5 L1
Deflection Cases Covered	1.0 L1, 1.0 D1 + 1.0 L1
Section Size	b = 141, h = 200, 200x141 in Strength Class C24
Section Properties (cm^2,cm^3,cm)	Area 282, $W_{el.y}$ 940, $W_{el.z}$ 662.7, i_y 5.77, i_z 4.07
Specification	3 : External uses fully exposed, Long term loading Calculate final deflection
Integrated Design	Critical Case 1
Member Details	N_{Ed} = -0.945 kN, L = 2.592 m, L_y = 2.592 m, L_z = 2.592 m, $L_{cr.y}$ = 1.0 L_y, $L_{cr.z}$ = 1.0 L_z Bearing length 75, Distance to Bearing 150 mm

Grade and Admissible Stresses (Strength Class C24)

$f_{m.y.d} = K_{mod}.K_{hy}.K_{sys}.f_{m.k}/\gamma_m$	0.55 x 1.00 x 1.00 x 24.00/1.3	10.15 N/mm^2	
$f_{m.z.d} = K_{mod}.K_{hz}.K_{sys}.f_{m.k}/\gamma_m$	0.55 x 1.01 x 1.00 x 24.00/1.3	10.28 N/mm^2	
$f_{t.0.d} = K_{mod}.K_h.K_{sys}.f_{t.0.k}/\gamma_m$	0.55 x 1.00 x 1.00 x 14.00/1.3	5.92 N/mm^2	
$f_{c.90.d} = K_{mod}.K_{c.90}.K_{sys}.f_{c.90.k}/\gamma_m$	0.55 x 1.50 x 1.00 x 2.50/1.3	1.59 N/mm^2	
$f_{v.d} = K_{mod}.K_{sys}.f_{v.k}/\gamma_m$	0.55 x 1.00 x 4.00/1.3	1.69 N/mm^2	
$E_{mean.fin} = E_{mean}/(1+K_{def})$	11000/(1+2.00)	3667 N/mm^2	Deflection

Tensile Resistance

$\sigma_{t.a} = N_{Ed}$/Area	0.945 / 282 ≤ 5.92	0.03 N/mm^2	OK

Axial Load with Moments Check

Critical Design Location	X = 0.552		
$\sigma_{m.y.d} = M_y/W_{el.y}$	5.339 / 940 ≤ 10.15	5.68 N/mm^2	OK
$\sigma_{m.z.d} = M_z/W_{el.z}$	1.150 / 662.7 ≤ 10.28	1.74 N/mm^2	OK
$U_t = \sigma_{c.0.d}/f_{t.0.d}$	0.034/5.923	0.006	OK
$U_{m.y} = \sigma_{m.y.d}/f_{m.y.d}$	5.680/10.154	0.559	OK
$U_{m.z} = \sigma_{m.z.d}/f_{m.z.d}$	1.740/10.280	0.169	OK
$U_t+U_{m.y}+K_m.U_{m.z}$	0.006+0.559+0.7x0.169	0.684	OK
$U_t+K_m.U_{m.y}+U_{m.z}$	0.006+0.7x0.559+0.169	0.566	OK

Shear and Bearing Check

Critical Design Location	X = 0.000		
τ_a= 1.5 $\sqrt{}(V_{y.Ed}{}^2+V_{z.Ed}{}^2)$ / Area / kcr	1.5 $\sqrt{}(9.617^2+2.075^2)$ / 282 / 0.67 ≤ 1.69	0.78 N/mm^2	OK
$\sigma_{c\perp ax} = V_{y.Ed}$ / $(b.l_y)$	9.617 / (141 x 75) ≤ 1.59	0.91 N/mm^2	OK
$\sigma_{c\perp ay} = V_{z.Ed}$ / $(h.l_z)$	2.075 / (200 x 75) ≤ 1.59	0.14 N/mm^2	OK

Deflection Check (Shear Deflection NOT Included)

Critical Load Case 003 : Dead Plus Live (Serviceability)			
$\delta = \delta_m$	In-span 7.27 ≤ L/250	7.27 mm	OK

Adopt timber rfater trimmer joists 3No.47x200mm C24 bolted together with M12 bolts & toothed washers at 500mm c/c to support external mansard roof windows.

MASTERKEY : TIMBER DESIGN

AXIAL LOAD WITH MOMENT DESIGN TO BS EN 1995-1-1:2004 + A1:2008

Timber Trimmers TB1

Member 1 (N.1-N.2) @ Level 1

Summary Design Data

Eurocode National Annex	Using UK values
Strength class code	BS EN 338:2009
Design Cases Covered	1.35 D1 + 1.5 L1
Deflection Cases Covered	1.0 L1, 1.0 D1 + 1.0 L1
Section Size	b = 94, h = 100, 100x94 in Strength Class C24
Section Properties (c,cm³,cm)	Area 94, $W_{el.y}$ 156.7, $W_{el.z}$ 147.3, i_y 2.89, i_z 2.71
Specification	2 : Covered and generally unheat, Long term loading Calculate final deflection
Integrated Design	Critical Case 1
Member Details	N_{Ed} = 0.0 kN, L = 1.0 m, L_y = 1.0 m, L_z = 1.0 m, $L_{cr.y}$ = 1.0 L_y , $L_{cr.z}$ = 1.0 L_z Bearing length 75, Distance to Bearing 150 mm

Grade and Admissible Stresses (Strength Class C24)

$f_{m.y.d} = K_{mod}.K_{hy}.K_{sys}.f_{m.k}/\gamma_m$	0.70 x 1.08 x 1.00 x 24.00/1.3	14.01 N/mm²	
$f_{m.z.d} = K_{mod}.K_{hz}.K_{sys}.f_{m.k}/\gamma_m$	0.70 x 1.10 x 1.00 x 24.00/1.3	14.19 N/mm²	
$f_{c.90.d} = K_{mod}.K_{c.90}.K_{sys}.f_{c.90.k}/\gamma_m$	0.70 x 1.50 x 1.00 x 2.50/1.3	2.02 N/mm²	
$f_{v.d} = K_{mod}.K_{sys}.f_{v.k}/\gamma_m$	0.70 x 1.00 x 4.00/1.3	2.15 N/mm²	
$E_{mean.fin} = E_{m}ean/(1+K_{def})$	11000/(1+0.80)	6111 N/mm²	Deflection

Axial Load with Moments Check

Critical Design Location	X = 0.488		
$\sigma_{m.y.d} = M_y/W_{el.y}$	0.282 / 156.67 ≤ 14.01	1.80 N/mm²	OK
$U_{m.y} = \sigma_{m.y.d}/f_{m.y.d}$	1.800/14.015	0.128	OK
$U_{m.y}$	0.128	0.128	OK
$L_{eff}=L.K_{LTB}$	1.000x1.000	1.000	
$\sigma_{mcrit}=\sim p\sqrt{}(E_{05}.I_z.G_{05}.J)/(L_{eff}.W_y)$	~p√(7.40x692.15x0.46x1235.72)/(1.000x156.67)	343.711	
$\lambda_{r,elm}=\sqrt{}(f_{mk}/\sigma_{mcrit})$	√(24.00/343.71)	0.264	
k_{Crit}	$\lambda_{r,elm}$ < 0.75	1.000	
$\sigma_{m.y.d}/(k_{Crit}\bullet f_{m.y.d})$	1.800/(1.000x14.015)	0.128	OK

Shear and Bearing Check

Critical Design Location	X = 0.000		
τ_a = 1.5 $V_{y.Ed}$ / Area / k_{cr}	1.5 x 1.132 / 94 / 0.67 ≤ 2.15	0.27 N/mm²	OK
$\sigma_{c\perp ax} = V_{y.Ed}$ / (b.l_y)	1.132 / (94 x 75) ≤ 2.02	0.16 N/mm²	OK

Deflection Check (Shear Deflection NOT Included)

Critical Load Case 003 : Dead plus Live (Serviceability)

$\delta = \delta_m$	In-span 0.44 ≤ L/250	0.44 mm	OK

Adopt timber trimmer joists 2No.47x100mm C24 bolted together with M12 bolts & toothed washers at 500mm c/c

MASTERKEY : TIMBER DESIGN

AXIAL LOAD WITH MOMENT DESIGN TO BS EN 1995-1-1:2004 + A1:2008

Timber Trimmers around Staircase (1st to 3rd floor)

Member 23 (N.6-N.8) @ Level 1

Summary Design Data

Eurocode National Annex	Using UK values
Strength class code	BS EN 338:2009
Design Cases Covered	1.35 D1 + 1.5 L1
Deflection Cases Covered	1.0 L1, 1.0 D1 + 1.0 L1
Section Size	b = 94, h = 175, 175x94 in Strength Class C24
Section Properties (c,cm³,cm)	Area 164.5, $W_{el.y}$ 479.8, $W_{el.z}$ 257.7, i_y 5.05, i_z 2.71
Specification	1 : Internal use in continuously heated building Long term loading, Calculate final deflection
Integrated Design	Critical Case 1
Member Details	N_{Ed} = 0.0 kN, L = 2.486 m, L_y = 2.486 m, L_z = 2.486 m, $L_{cr.y}$ = 1.0 L_y , $L_{cr.z}$ = 1.0 L_z Bearing length 75, Distance to Bearing 150 mm

Grade and Admissible Stresses (Strength Class C24)

$f_{m.y.d} = K_{mod}.K_{hy}.K_{sys}.f_{m.k}/\gamma_m$	0.70 x 1.00 x 1.00 x 24.00/1.3	12.92 N/mm²	
$f_{m.z.d} = K_{mod}.K_{hz}.K_{sys}.f_{m.k}/\gamma_m$	0.70 x 1.10 x 1.00 x 24.00/1.3	14.19 N/mm²	
$f_{c.90.d} = K_{mod}.K_{c.90}.K_{sys}.f_{c.90.k}/\gamma_m$	0.70 x 1.50 x 1.00 x 2.50/1.3	2.02 N/mm²	
$f_{v.d} = K_{mod}.K_{sys}.f_{v.k}/\gamma_m$	0.70 x 1.00 x 4.00/1.3	2.15 N/mm²	
$E_{mean.fin} = E_m ean/(1+K_{def})$	11000/(1+0.60)	6875 N/mm²	Deflection

Axial Load with Moments Check

Critical Design Location	X = 1.243		
$\sigma_{m.y.d} = M_y/W_{el.y}$	2.520 / 479.79 ≤ 12.92	5.25 N/mm²	OK
$U_{m.y} = \sigma_{m.y.d}/f_{m.y.d}$	5.250/12.923	0.406	OK
$U_{m.y}$	0.406	0.406	OK
$L_{eff}=L.K_{LTB}$	2.486x1.000	2.486	
$\sigma_{mcrit}=\sim p\sqrt{(E_{05}.I_z.G_{05}.J)/(L_{eff}.W_y)}$	~p√(7.40x1211.27x0.46x3216.88)/(2.486x479.79)	96.359	
$\lambda_{r,elm}=\sqrt{(f_{mk}/\sigma_{mcrit})}$	√(24.00/96.36)	0.499	
k_{Crit}	$\lambda_{r,elm} < 0.75$	1.000	
$\sigma_{m.y.d}/(k_{Crit} \bullet f_{m.y.d})$	5.250/(1.000x12.923)	0.406	OK

Shear and Bearing Check

Critical Design Location	X = 0.000		
$\tau_a = 1.5\ V_{y.Ed}$ / Area / k_{cr}	1.5 x 4.058 / 164.5 / 0.67 ≤ 2.15	0.55 N/mm²	OK
$\sigma_{c}\perp_{ax} = V_{y.Ed}$ / (b.l_y)	4.058 / (94 x 75) ≤ 2.02	0.58 N/mm²	OK

Deflection Check (Shear Deflection NOT Included)

Critical Load Case 003 : Dead Plus Live (Serviceability)

$\delta = \delta_m$	In-span 4.02 ≤ L/250	4.02 mm	OK

Adopt timber trimmer joists 2No. 47x175mm C24 bolted together with M12 bolts and toothed washers at 500mm c/c around staircase.

MASTERKEY : TIMBER DESIGN

AXIAL LOAD WITH MOMENT DESIGN TO BS EN 1995-1-1:2004 + A1:2008

Trimmer Joists to support staircase to Terrace at Third floor level

Members 1-2 (N.12-N.14) @ Level 1

Summary Design Data

Eurocode National Annex	Using UK values
Strength class code	BS EN 338:2009
Design Cases Covered	1.35 D1 + 1.5 L1
Deflection Cases Covered	1.0 L1, 1.0 D1 + 1.0 L1
Section Size	b = 94, h = 175, 175x94 in Strength Class C24
Section Properties (cm^2,cm^3,cm)	Area 164.5, $W_{el.y}$ 479.8, $W_{el.z}$ 257.7, i_y 5.05, i_z 2.71
Specification	2 : Covered and generally unheated, Long term loading Calculate final deflection
Integrated Design	Critical Case 1
Member Details	N_{Ed} = 0.0 kN, L = 2.734 m, L_y = 2.734 m, L_z = 2.734 m, $L_{cr.y}$ = 1.0 L_y, $L_{cr.z}$ = 1.0 L_z Bearing length 75, Distance to Bearing 150 mm

Grade and Admissible Stresses (Strength Class C24)

$f_{m.y.d} = K_{mod}.K_{hy}.K_{sys}.f_{m.k}/\gamma_m$	0.70 x 1.00 x 1.00 x 24.00/1.3	12.92 N/mm²	
$f_{m.z.d} = K_{mod}.K_{hz}.K_{sys}.f_{m.k}/\gamma_m$	0.70 x 1.10 x 1.00 x 24.00/1.3	14.19 N/mm²	
$f_{c.90.d} = K_{mod}.K_{c.90}.K_{sys}.f_{c.90.k}/\gamma_m$	0.70 x 1.50 x 1.00 x 2.50/1.3	2.02 N/mm²	
$f_{v.d} = K_{mod}.K_{sys}.f_{v.k}/\gamma_m$	0.70 x 1.00 x 4.00/1.3	2.15 N/mm²	
$E_{mean.fin} = E_{mean}/(1+K_{def})$	11000/(1+0.80)	6111 N/mm²	Deflection

Axial Load with Moments Check

Critical Design Location	X = 1.108		
$\sigma_{m.y.d} = M_y/W_{el.y}$	3.115 / 479.79 $\leq$ 12.92	6.49 N/mm²	OK
$U_{m.y} = \sigma_{m.y.d}/f_{m.y.d}$	6.490/12.923	0.502	OK
$U_{m.y}$	0.502	0.502	OK
$L_{eff}=L.K_{LTB}$	2.734x1.000	2.734	
$\sigma_{mcrit}=\pi\sqrt{}(E_{05}.I_z.G_{05}.J)/(L_{eff}.W_y)$	$\pi\sqrt{}$(7.40x1211.27x0.46x3216.88)/(2.734x479.79)	87.619	
$\lambda_{r,elm}=\sqrt{}(f_{mk}/\sigma_{mcrit})$	$\sqrt{}$(24.00/87.62)	0.523	
k_{Crit}	$\lambda_{r,elm}$ < 0.75	1.000	
$\sigma_{m.y.d}/(k_{Crit}\bullet f_{m.y.d})$	6.490/(1.000x12.923)	0.502	OK

Shear and Bearing Check

Critical Design Location	X = 0.000		
τ_a= 1.5 $V_{y.Ed}$ / Area / k_{cr}	1.5 x 5.079 / 164.5 / 0.67 $\leq$ 2.15	0.69 N/mm²	OK
$\sigma_{c\perp ax}$ = $V_{y.Ed}$ / (b.l_y)	5.079 / (94 x 75) $\leq$ 2.02	0.72 N/mm²	OK

Deflection Check (Shear Deflection NOT Included)

Critical Load Case 003 : Dead Plus Live (Serviceability)			
$\delta = \delta_m$	In-span 6.61 $\leq$ L/250	6.61 mm	OK

Adopt timber trimmer joists 2No. 47x175mm C24 bolted together with M12 bolts and toothed washers at 500mm c/c around staircase to terrace at third floor level.

MASTERKEY : TIMBER DESIGN

AXIAL LOAD WITH MOMENT DESIGN TO BS EN 1995-1-1:2004 + A1:2008

Timber Post to be supported at ground floor level

Member 34 (N.21-N.28) @ Level 1

Summary Design Data

Eurocode National Annex	Using UK values
Strength class code	BS EN 338:2009
Design Cases Covered	1.35 D1 + 1.5 L1
Deflection Cases Covered	1.0 L1, 1.0 D1 + 1.0 L1
Section Size	b = 100, h = 100, 100x100 in Strength Class C24
Section Properties (c,cm³,cm)	Area 100, $W_{el.y}$ 166.7, $W_{el.z}$ 166.7, i_y 2.89, i_z 2.89
Specification	1 : Internal use in continuously heated building Long term loading, Calculate final deflection
Integrated Design	Critical Case 1
Member Details	N_{Ed} = 14.724 kN, L = 3.0 m, L_y = 3.0 m, L_z = 3.0 m, $L_{cr.y}$ = 1.0 L_y , $L_{cr.z}$ = 1.0 L_z Bearing length 75, Distance to Bearing 150 mm

Grade and Admissible Stresses (Strength Class C24)

$f_{m.y.d} = K_{mod}.K_{hy}.K_{sys}.f_{m.k}/\gamma_m$	0.70 x 1.08 x 1.00 x 24.00/1.3	14.01 N/mm²	
$f_{m.z.d} = K_{mod}.K_{hz}.K_{sys}.f_{m.k}/\gamma_m$	0.70 x 1.08 x 1.00 x 24.00/1.3	14.01 N/mm²	
$f_{c.0.d} = K_{mod}.K_{sys}.f_{c.0.k}/\gamma_m$	0.70 x 1.00 x 21.00/1.3	11.31 N/mm²	
$f_{c.90.d} = K_{mod}.K_{c.90}.K_{sys}.f_{c.90.k}/\gamma_m$	0.70 x 1.50 x 1.00 x 2.50/1.3	2.02 N/mm²	
$f_{v.d} = K_{mod}.K_{sys}.f_{v.k}/\gamma_m$	0.70 x 1.00 x 4.00/1.3	2.15 N/mm²	
$E_{mean.fin} = E_m ean/(1+K_{def})$	11000/(1+0.60)	6875 N/mm²	Deflection

Compression Resistance

$\lambda_y = L_{ey}/i_y$	300/2.887 ≤ 180	103.9	OK
$\lambda_{rel.y} = \lambda_y/\sim p\ \sqrt{(f_{c.0.k}/E_{0.05})}$	103.9/~p √ 21/7400	1.762	
$k_{c.y}$ = fn(β_c, $\lambda_{rel.y}$, k_y)	0.2, 1.762, 2.199	0.285	
$f_{c.y.0.d} = k_{c.y} . f_{c.0.d}$	0.285 x 11.31	3.22 N/mm²	
$\lambda_z = L_{ez}/i_z$	300/2.887 ≤ 180	103.9	OK
$\lambda_{rel.z} = \lambda_z/\sim p\ \sqrt{(f_{c.0.k}/E_{0.05})}$	103.9/~p √ 21/7400	1.762	
$k_{c.z}$ = fn(β_c, $\lambda_{rel.z}$, k_z)	0.2, 1.762, 2.199	0.285	
$f_{c.z.0.d} = k_{c.z} . f_{c.0.d}$	0.285 x 11.31	3.22 N/mm²	
$\sigma_{c.a} = N_{Ed}$ /Area	14.724 / 100 ≤ 3.22	1.47 N/mm²	OK

Axial Load with Moments Check

Critical Design Location	X = 0.000		
$\sigma_{m.y.d} = M_y/W_{el.y}$	0.120 / 166.67 ≤ 14.01	0.72 N/mm²	OK
$\sigma_{m.z.d} = M_z/W_{el.z}$	0.054 / 166.67 ≤ 14.01	0.32 N/mm²	OK
$U_{c.y} = \sigma_{c.0.d}/(k_{c.y} \bullet f_{c.0.d})$	1.472/(0.285x11.308)	0.458	OK
$U_{c.z} = \sigma_{c.0.d}/(k_{c.z} \bullet f_{c.0.d})$	1.472/(0.285x11.308)	0.458	OK
$U_{m.y} = \sigma_{m.y.d}/f_{m.y.d}$	0.720/14.015	0.051	OK
$U_{m.z} = \sigma_{m.z.d}/f_{m.z.d}$	0.320/14.015	0.023	OK
$U_{c.y}+U_{m.y}+k_m.U_{m.z}$	0.458+0.051+0.7x0.023	0.525	OK
$U_{c.z}+k_m.U_{m.y}+U_{m.z}$	0.458+0.7x0.051+0.023	0.516	OK

Shear and Bearing Check

Critical Design Location	X = 0.000		
$\tau_a = 1.5\ \sqrt{(V_{y.Ed}{}^2+V_{z.Ed}{}^2)}$ / Area / kcr	1.5 √(0.04²+0.018²) / 100 / 0.67 ≤ 2.15	0.01 N/mm²	OK
$\sigma_{c\perp ax} = V_{y.Ed}$ / (b.l_y)	0.04 / (100 x 75) ≤ 2.02	0.01 N/mm²	OK
$\sigma_{c\perp ay} = V_{z.Ed}$ / (h.l_z)	0.018 / (100 x 75) ≤ 2.02	0.00 N/mm²	OK

Deflection Check (Shear Deflection NOT Included)

Critical Load Case 003 : Dead Plus Live (Serviceability)			
$\delta = \delta_m$	In-span 0.91 ≤ L/250	0.91 mm	OK

Adopt timber trimmer post 100x100mm C24 to form staircase at first & second floor level and supported onto trimmer joists at ground floor level.

MASTERKEY : TIMBER DESIGN

AXIAL LOAD WITH MOMENT DESIGN TO BS EN 1995-1-1:2004 + A1:2008

Trimmers around skylight & around staircase at ground floor level

Members 34-36 (N.36-N.37) @

Summary Design Data

Eurocode National Annex	Using UK values
Strength class code	BS EN 338:2009
Design Cases Covered	1.35 D1 + 1.5 L1
Deflection Cases Covered	1.0 L1, 1.0 D1 + 1.0 L1
Section Size	b = 94, h = 200, 200x94 in Strength Class C24
Section Properties (c,cm³,cm)	Area 188, $W_{el.y}$ 626.7, $W_{el.z}$ 294.5, i_y 5.77, i_z 2.71
Specification	1 : Internal use in continuously heated building Long term loading, Calculate final deflection
Integrated Design	Critical Case 1
Member Details	N_{Ed} = 0.0 kN, L = 3.774 m, L_y = 3.774 m, L_z = 3.774 m, $L_{cr.y}$ = 1.0 L_y , $L_{cr.z}$ = 1.0 L_z Bearing length 75, Distance to Bearing 150 mm

Grade and Admissible Stresses (Strength Class C24)

$f_{m.y.d} = K_{mod}.K_{hy}.K_{sys}.f_{m.k}/\gamma_m$	0.70 x 1.00 x 1.00 x 24.00/1.3	12.92 N/mm²	
$f_{m.z.d} = K_{mod}.K_{hz}.K_{sys}.f_{m.k}/\gamma_m$	0.70 x 1.10 x 1.00 x 24.00/1.3	14.19 N/mm²	
$f_{c.90.d} = K_{mod}.K_{c.90}.K_{sys}.f_{c.90.k}/\gamma_m$	0.70 x 1.50 x 1.00 x 2.50/1.3	2.02 N/mm²	
$f_{v.d} = K_{mod}.K_{sys}.f_{v.k}/\gamma_m$	0.70 x 1.00 x 4.00/1.3	2.15 N/mm²	
$E_{mean.fin} = E_m ean/(1+K_{def})$	11000/(1+0.60)	6875 N/mm²	Deflection

Axial Load with Moments Check

Critical Design Location	X = 1.934		
$\sigma_{m.y.d} = M_y/W_{el.y}$	2.590 / 626.67 ≤ 12.92	4.13 N/mm²	OK
$U_{m.y} = \sigma_{m.y.d}/f_{m.y.d}$	4.130/12.923	0.320	OK
$U_{m.y}$	0.320	0.320	OK
$L_{eff}=L.K_{LTB}$	3.774x1.000	3.774	
$\sigma_{mcrit}=\sim p\sqrt{(E_{05}.I_z.G_{05}.J)}/(L_{eff}.W_y)$	~p√(7.40x1384.31x0.46x3904.32)/(3.774x626.67)	57.235	
$\lambda_{r,elm}=\sqrt{(f_{mk}/\sigma_{mcrit})}$	√(24.00/57.23)	0.648	
k_{Crit}	$\lambda_{r,elm}$ < 0.75	1.000	
$\sigma_{m.y.d}/(k_{Crit} \bullet f_{m.y.d})$	4.130/(1.000x12.923)	0.320	OK

Shear and Bearing Check

Critical Design Location	X = 3.774		
$\tau_a = 1.5\ V_{y.Ed}$ / Area / k_{cr}	1.5 x 2.951 / 188 / 0.67 ≤ 2.15	0.35 N/mm²	OK
$\sigma_{c\perp ax} = V_{y.Ed}$ / (b.l_y)	2.951 / (94 x 75) ≤ 2.02	0.42 N/mm²	OK

Deflection Check (Shear Deflection NOT Included)

Critical Load Case 003 : Dead Plus Live (Serviceability)

$\delta = \delta_m$	In-span 6.30 ≤ L/250	6.30 mm	OK

Adopt timber trimmer joists 2No. 47x200mm C24 bolted together with M12 bolts and toothed washers at 500mm c/c around staircase & internal skylight at ground floor level.

RACKING WALL DESIGN

In accordance with EN1995-1-1:2004+A1:2008 incorporating corrigendum June 2006 and the UK National Annex incorporating Corrrigendum No.2 and the Published Document PD6693-1:2012 Non-Contradictory Complementary Information to Eurocode 5.

Tedds calculation version 1.0.01

Compression force of the leeward end of the racking wall shall be checked indepently in combination with the stud buckling design and the top rail bearing design in accordance with Eurocode 5. For a wall diaphragm with more than two studs within the 0.1 L of its leeward end in a dwelling of less than three storeys, the compressive force at the leeward end may be disregarded (clause 21.5.2.10 of PD6693-1).

Single storey racking wall without openings

Wall panel properties

Geometry

Wall height;	H = **2700** mm;	Wall length;	L = **2050** mm
Width of stud;	b_s = **38** mm;	Depth of stud;	h_s = **89** mm
Stud spacing;	s_s = **400** mm;	Sheathing layers;	**1**;

Timber frame material

Material;	Solid timber;	Strength class;	C24
Characteristic density;	ρ_k = **350** kg/m^3		

Sheathing materials

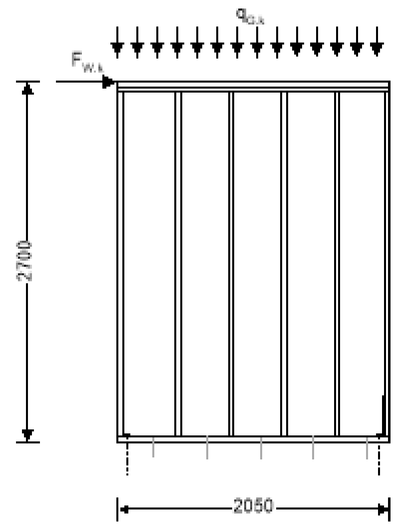
Sheathing material;	OSB/3;	t_{s1} = **9** mm;	ρ_{k1} = **575** kg/m^3
Fastener;	d_{n1} = **3.1** mm;	l_{n1} = **50** mm;	s_1 = **150** mm

Connection to substrate

Sole plate detail;	Open panel sole plate detail;	Holding down restrain;	Strap dimensions 50
mm×721 mm and no6 nails			
;		Characteristic restrain capacity;	$F_{hd.k}$ = **6.90** kN

Actions on the wall

Self weight;	$q_{sw.k}$ = **0.20** kN/m^2;	Permanent UDL;	$q_{G.k}$ = **0.80** kN/m
Uplift wind;	$q_{uplift.k}$ = **0.00** kN/m;	Horizontal racking load;	$F_{W.k}$ = **3.33** kN



Sole plate fixing detail

Number of shear planes; N_{sp} = **2**

Sub-connection 1 - Bottom rail to sole plate, C1; BRT ring shanked nail 3.1x75 mm

Charact withdrawal capacity; $F_{sp.w.k1}$ = **0.19** kN; Charact shear capacity; $F_{sp.v.k1}$ = **0.83** kN

Number of parallel fasteners; n_{sp1} = **1**; Spacing of the fasteners; s_{sp1} = **150** mm

Sub-connection 2 - Sole plate to foundations, C2; Shot-fired smooth round nail 3.5x70 mm

Charact withdrawal capacity; $F_{sp.w.k2}$ = **0.05** kN; Charact shear capacity; $F_{sp.v.k2}$ = **1.50** kN

Number of parallel fasteners; n_{sp2} = **1**; Spacing of the fasteners; s_{sp2} = **300** mm

Partial safety factors

Service class conditions – cl 2.3.1.3

Racking wall panel; SC = **2**; Sole plate fixing; SC_{sp} = **2**

Safety factors – Table NA.A1.2(A/B)

Limit state (EQU and STR); Permanent favourable actions; $\gamma_{G.fav}$ = **1.00**

Limit state (STR/GEO); Variable unfavourable actions; $\gamma_{Q.unfav.str}$ = **1.50**

Material factors – Table NA.3

Sheathing panel; $\gamma_{M.sht1}$ = **1.20**; Timbers in frame; $\gamma_{M.frame}$ = **1.30**

Connections; $\gamma_{M.conn}$ = **1.30**

Modification factors – Table 3.1

Timber in the frame; $k_{mod.frame}$ = **1.10**

Primary sheathing material; $k_{mod.sht1}$ = **0.90**; Sheathed frame side 1; $k_{mod.wall1}$ = **0.99**

Sole plate shear plane 1; $k_{mod.sp1}$ = **1.10**

Sole plate shear plane 2; $k_{mod.sp2}$ = **1.05**

Determination of design fastener capacities

Sole plate fixing fasteners

Shear plane 1

Design withdrawal capacity; $f_{sp.w.d1}$ = **1.08** kN/m; Design lateral load capacity; $f_{sp.v.d1}$ = **4.70** kN/m

Shear plane 2

Design withdrawal capacity; $f_{sp.w.d2}$ = **0.13** kN/m; Design lateral load capacity; $f_{sp.v.d2}$ = **4.04** kN/m

Minimum withdrawal capacity; $f_{sp.w.d}$ = **0.13** kN/m; Minimum lateral load capacity; $f_{sp.v.d}$ = **4.04** kN/m;

Primary sheathing to frame connection

Fastener spacing; s_1 = **150** mm; Design lateral load capacity; $F_{v.Rd1}$ = **0.54** kN

Design shear by unit length; $f_{p.d1}$ = **4.67** kN/m

Design loads acting on shear wall

Design permanent udl; $q_{G.d}$ = **8.65** kN/m; Design wind uplift udl; $q_{uplift.d}$ = **0.00** kN/m

Self-weight of the wall panel; $q_{sw.d}$ = **0.54** kN/m; Design horizontal wind load; $F_{w.d}$ = **7.95** kN

Design holding down tension; $F_{hd.d}$ = **5.28** kN

Stabilising moments

Design destabilising moments;

Distance to top sheathing; $h_{dst.top}$ = **38** mm; Distance to base sheathing; $h_{dst.base}$ = **2700** mm

Destabilising moment at top; $M_{d.dst.top}$ = **0.30** kNm; Destabilising moment at base; $M_{d.dst.base}$ = **13.49** kNm

Design stabilising moments;

Design stabilising vertical load; $f_{stb.d}$ = **1.34** kN/m; Total vertical load; $F_{stb.d}$ = **2.75** kN

Stabilising moment load; $M_{d.stb.f}$ = **2.82** kNm; Stabilising moment h-down; $M_{d.stb.hd}$ = **10.83** kNm

Total stabilising moment; $M_{d.stb}$ = **13.64** kNm;

Design for overturning stability

Net stabilising moment - top; $M_{n.top}$ = **13.64** kNm; Factor of Utilisation; $M_{d.dst.top}$ / $M_{d.stb}$ = **0.022**

PASS - Design stabilising moment exceeds design destabilising moment at base of the shear wall

Net stabilising moment - base; $M_{n.base}$ = **0.16** kNm; Factor of Utilisation; $M_{d.dst.base}$ / $M_{d.stb}$ = **0.989**

PASS - Design stabilising moment exceeds design destabilising moment at base of the shear wall

Design for sliding stability

Coefficient of friction; μ_{fr} = 0.4; Frictional resistance; $F_{friction}$ = **1.10** kN

Sole plate shear resistance; $F_{sp.v.d}$ = **8.29** kN; Sliding resistance; $F_{sliding}$ =**9.38** kN

Design horizontal wind load; $F_{W.d}$ = **7.95** kN; Factor of Utilisation; $F_{W.d}$ / $F_{sliding}$ = **0.847**

PASS - Design sliding resistance exceeds horizontal design wind load

<u>Design racking strength</u>

Opening factor – cl 21.26

Percentage of opening area; p = **0.00**; Opening factor; $K_{opening}$ = **1.00**

Panel shape factor – cl 21.5.2.5

Ratio of shear capacities; μ = **0.03**; Shape factor; K_w = **0.53**

Design racking strength – cl 21.5.2

Shear strength of sole plate; $F_{sp.v.d}$ = **8.29** kN; Shear strength of panel; $F_{wall.v.d}$ = **5.12** kN

Design racking strength; $F_{v.d}$ = **5.12** kN; FoU; $F_{W.d}$ / $F_{v.d}$ = **1.551**

FAIL - Design racking strength is less than racking load due to wind

Serviceability load limit; $F_{SLS.lim}$ = **6.07** kN/m; Modified serviceability load; $F_{SLS.mod}$ = **2.50** kN/m

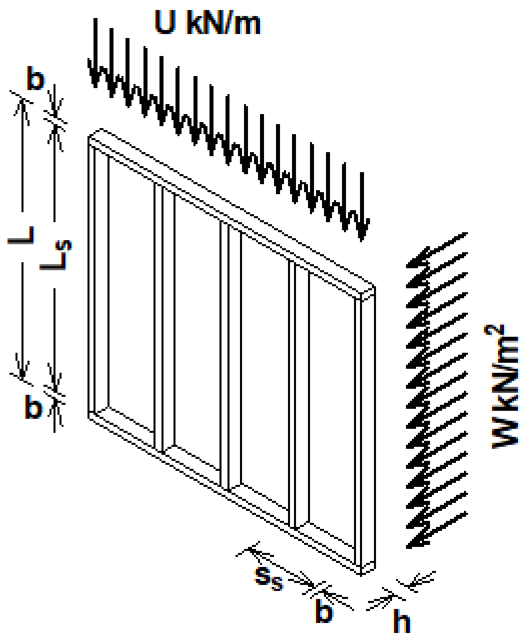
; FoU; $F_{SLS.mod}$ / $F_{SLS.lim}$ = **0.412**

PASS - The condition 21.5.2.3 of PD 6693-1 is met

Provide internal timber studwall 38x89mm C24 @400mm c/c sheathed with 9mm OSB3 on one side , nailed onto studs at 150mm c/c and holding down with tension straps 50x721m to resist lateral loads.

TIMBER STUD DESIGN (BS5268-2:2002)

TEDDS calculation version 1.0.05



Stud details			
Stud breadth;	b = **38** mm;	Stud depth;	h = **140** mm
Number of studs;	N_s = **1**;	Stud grade;	**C24**
Section properties			
Cross sectional area;	A = **5320** mm^2;	Section modulus;	Z = **124133** mm^3
Moment of inertia major axis;	I_x = **8689333** mm^4;	Moment of inertia minor axis;	I_y = **640173** mm^4
Radius of gyration major axis;	r_x = **40.4** mm;	Radius of gyration minor axis;	r_y = **11.0** mm
Panel details - Studs restrained by sheathing in the plane of the panel			
Panel height;	L = **2400** mm;	Stud length;	L_s = **2324** mm
Standard stud spacing;	s_s = **400** mm;	Panel opening;	O = **0** mm
Loaded panel length;	s = **400** mm;	Effective length major axis;	L_{ex} = **1975** mm
		Slenderness ratio;	λ = **48.88**
Vertical loading details	**Dead loads**	**Imposed loads**	
Wall UDL;	U_{w_d} = **1.01** kN/m		
Roof UDL;	U_{r_d} = **0.30** kN/m;	U_{r_i} = **0.38** kN/m	
Floor UDL;	U_{f_d} = **0.60** kN/m;	U_{f_i} = **2.00** kN/m	
Imposed floor load duration;	**Long term**		
Lateral loading			
Wind loading;	W = **0.60** kN/m^2;	Wind load duration;	**Very short term**
Modification factors			
Section depth factor;	K_7 = **1.09**;	Load sharing factor;	K_8 = **1.10**
Check bending stress			
Permissible bending stress;	σ_{m_adm} = **15.700** N/mm^2;	Applied bending stress;	σ_{m_max} = **1.392** N/mm^2

PASS - Applied bending stress under very short term loads is within permissible limits

Check compressive stress on stud

Permissible comp.stress; σ_{c_adm} = **10.373** N/mm^2; Applied comp.stress; σ_{c_max} = **0.322** N/mm^2

PASS - Applied compressive stress under very short term loads is within permissible limits

Check compressive stress on rail

Permissible comp.stress; σ_{cp1_adm} = **5.698** N/mm^2; Applied comp.stress; σ_{cp1_max} = **0.322** N/mm^2

PASS - Applied compressive stress under very short term loads is within permissible limits

Check combined axial compression and bending

Combined axial compression and bending value; K = **0.121**; **< 1**

PASS - Combined compressive and bending stresses under very short term loads are within permissible limits

Check stud deflection

Permissible deflection; δ_{adm} = **4.300** mm; Actual deflection; δ_{max} = **0.971** mm

PASS - Deflection due to wind loading is less than permissible limit

Check compressive stress on stud

Permissible comp.stress; σ_{c_adm} = **7.860** N/mm^2; Applied comp.stress; σ_{c_max} = **0.322** N/mm^2

PASS - Applied compressive stress under medium term loads is within permissible limits

Check compressive stress on rail

Permissible comp.stress; σ_{cp1_adm} = **4.070** N/mm^2; Applied comp.stress; σ_{cp1_max} = **0.322** N/mm^2

PASS - Applied compressive stress under medium term loads is within permissible limits

Check compressive stress on stud

Permissible comp.stress; σ_{c_adm} = **6.451** N/mm^2; Applied comp.stress; σ_{c_max} = **0.294** N/mm^2

PASS - Applied compressive stress under long term loads is within permissible limits

Check compressive stress on rail

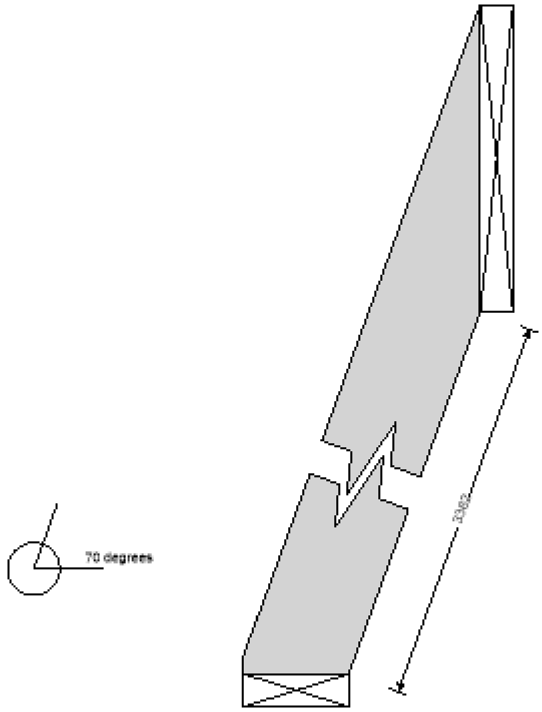
Permissible comp.stress; σ_{cp1_adm} = **3.256** N/mm^2; Applied comp.stress; σ_{cp1_max} = **0.294** N/mm^2

PASS - Applied compressive stress under long term loads is within permissible limits

Adopt external timber studwall 38x140mm C24 at 400mm c/c to support mansard roof and be supported off onto floor joists at third floor level.

TIMBER RAFTER DESIGN (BS5268-2:2002)

TEDDS calculation version 1.0.03



Rafter details

Breadth of timber sections;	b = **47** mm;	Depth of timber sections;	h = **150** mm
Rafter spacing;	s = **400** mm;	Rafter span;	**Single span**
Clear length of span on slope;	L_{cl} = **3362** mm;	Rafter slope;	α = **70.0** deg
Timber strength class;	**C24**		

Section properties

Cross sectional area of rafter;	A = **7050** mm^2;	Section modulus;	Z = **176250** mm^3
Radius of gyration;	r = **43** mm;	Second moment of area;	I = **13218750** mm^4

Loading details

Rafter self weight;	F_j = **0.02** kN/m;	Dead load on slope;	F_d = **1.20** kN/m^2
Imposed snow load on plan;	F_u = **0.75** kN/m^2;	Imposed point load;	F_p = **0.90** kN

Modification factors

Section depth factor;	K_7 = **1.08**;	Load sharing factor;	K_8 = **1.10**

Consider long term load condition

Load duration factor;	K_3 = **1.00**;	Total UDL perp. to rafter;	F = **0.172** kN/m
Notional bearing length;	L_b = **2** mm;	Effective span;	L_{eff} = **3365** mm

Check bending stress

Permissible bending stress;	σ_{m_adm} = **8.904** N/mm^2;	Applied bending stress;	σ_{m_max} = **1.385** N/mm^2

PASS - Applied bending stress within permissible limits

Check compressive stress parallel to grain

Permissible comp. stress;	σ_{c_adm} = **4.693** N/mm^2;	Applied compressive stress;	σ_{c_max} = **0.354** N/mm^2

PASS - Applied compressive stress within permissible limits

Check combined bending and compressive stress parallel to grain

Combined loading check; **0.235 < 1**

PASS - Combined compressive and bending stresses are within permissible limits

Check shear stress

Permissible shear stress; τ_{adm} = **0.781** N/mm^2; Applied shear stress; τ_{max} = **0.062** N/mm^2

PASS - Applied shear stress within permissible limits

Check deflection

Permissible deflection; δ_{adm} = **10.094** mm; Total deflection; δ_{max} = **2.077** mm

PASS - Total deflection within permissible limits

<u>Consider medium term load condition</u>

Load duration factor; K_3 = **1.25**; Total UDL perp. to rafter; F = **0.208** kN/m

Notional bearing length; L_b = **3** mm; Effective span; L_{eff} = **3365** mm

Check bending stress

Permissible bending stress; σ_{m_adm} = **11.130** N/mm^2; Applied bending stress; σ_{m_max} = **1.667** N/mm^2

PASS - Applied bending stress within permissible limits

Check compressive stress parallel to grain

Permissible comp. stress; σ_{c_adm} = **5.361** N/mm^2; Applied compressive stress; σ_{c_max} = **0.426** N/mm^2

PASS - Applied compressive stress within permissible limits

Check combined bending and compressive stress parallel to grain

Combined loading check; **0.233 < 1**

PASS - Combined compressive and bending stresses are within permissible limits

Check shear stress

Permissible shear stress; τ_{adm} = **0.976** N/mm^2; Applied shear stress; τ_{max} = **0.074** N/mm^2

PASS - Applied shear stress within permissible limits

Check deflection

Permissible deflection; δ_{adm} = **10.096** mm; Total deflection; δ_{max} = **2.502** mm

PASS - Total deflection within permissible limits

<u>Consider short term load condition</u>

Load duration factor; K_3 = **1.50**; Total UDL perp. to rafter; F = **0.172** kN/m

Notional bearing length; L_b = **4** mm; Effective span; L_{eff} = **3366** mm

Check bending stress

Permissible bending stress; σ_{m_adm} = **13.355** N/mm^2; Applied bending stress; σ_{m_max} = **2.855** N/mm^2

PASS - Applied bending stress within permissible limits

Check compressive stress parallel to grain

Permissible comp. stress; σ_{c_adm} = **5.874** N/mm^2; Applied compressive stress; σ_{c_max} = **0.474** N/mm^2

PASS - Applied compressive stress within permissible limits

Check combined bending and compressive stress parallel to grain

Combined loading check; **0.301 < 1**

PASS - Combined compressive and bending stresses are within permissible limits

Check shear stress

Permissible shear stress; τ_{adm} = **1.172** N/mm^2; Applied shear stress; τ_{max} = **0.127** N/mm^2

PASS - Applied shear stress within permissible limits

Check deflection

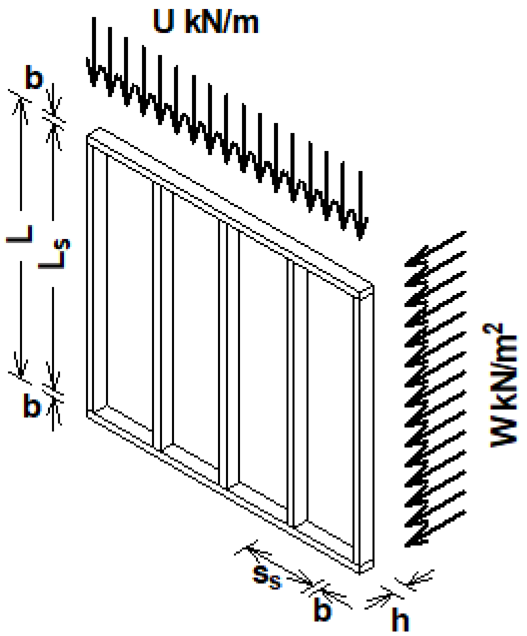
Permissible deflection; δ_{adm} = **10.098** mm; Total deflection; δ_{max} = **3.859** mm

PASS - Total deflection within permissible limits

Adopt timber rafter joists 47x150mm C24 @400mm c/c. Max span 1.15m. a=70deg

TIMBER STUD DESIGN (BS5268-2:2002)

TEDDS calculation version 1.0.05



Stud details

Stud breadth;	b = **38** mm;	Stud depth;	h = **140** mm
Number of studs;	N_s = **1**;	Stud grade;	**C24**

Section properties

Cross sectional area;	A = **5320** mm^2;	Section modulus;	Z = **124133** mm^3
Moment of inertia major axis;	I_x = **8689333** mm^4;	Moment of inertia minor axis;	I_y = **640173** mm^4
Radius of gyration major axis;	r_x = **40.4** mm;	Radius of gyration minor axis;	r_y = **11.0** mm

Panel details - Studs restrained by sheathing in the plane of the panel

Panel height;	L = **2400** mm;	Stud length;	L_s = **2324** mm
Standard stud spacing;	s_s = **400** mm;	Panel opening;	O = **0** mm
Loaded panel length;	s = **400** mm;	Effective length major axis;	L_{ex} = **1975** mm
		Slenderness ratio;	λ = **48.88**

Vertical loading details	**Dead loads**	**Imposed loads**
Wall UDL;	U_{w_d} = **1.01** kN/m	
Roof UDL;	U_{r_d} = **0.30** kN/m;	U_{r_i} = **0.38** kN/m
Floor UDL;	U_{f_d} = **0.60** kN/m;	U_{f_i} = **2.00** kN/m
Imposed floor load duration;	**Long term**	

Lateral loading

Wind loading;	W = **0.60** kN/m^2;	Wind load duration;	**Very short term**

Modification factors

Section depth factor; K_7 = **1.09**; Load sharing factor; K_8 = **1.10**

Check bending stress

Permissible bending stress; σ_{m_adm} = **15.700** N/mm^2; Applied bending stress; σ_{m_max} = **1.392** N/mm^2

PASS - Applied bending stress under very short term loads is within permissible limits

Check compressive stress on stud

Permissible comp.stress; σ_{c_adm} = **10.373** N/mm^2; Applied comp.stress; σ_{c_max} = **0.322** N/mm^2

PASS - Applied compressive stress under very short term loads is within permissible limits

Check compressive stress on rail

Permissible comp.stress; σ_{cp1_adm} = **5.698** N/mm^2; Applied comp.stress; σ_{cp1_max} = **0.322** N/mm^2

PASS - Applied compressive stress under very short term loads is within permissible limits

Check combined axial compression and bending

Combined axial compression and bending value; K = **0.121**; **< 1**

PASS - Combined compressive and bending stresses under very short term loads are within permissible limits

Check stud deflection

Permissible deflection; δ_{adm} = **4.300** mm; Actual deflection; δ_{max} = **0.971** mm

PASS - Deflection due to wind loading is less than permissible limit

Check compressive stress on stud

Permissible comp.stress; σ_{c_adm} = **7.860** N/mm^2; Applied comp.stress; σ_{c_max} = **0.322** N/mm^2

PASS - Applied compressive stress under medium term loads is within permissible limits

Check compressive stress on rail

Permissible comp.stress; σ_{cp1_adm} = **4.070** N/mm^2; Applied comp.stress; σ_{cp1_max} = **0.322** N/mm^2

PASS - Applied compressive stress under medium term loads is within permissible limits

Check compressive stress on stud

Permissible comp.stress; σ_{c_adm} = **6.451** N/mm^2; Applied comp.stress; σ_{c_max} = **0.294** N/mm^2

PASS - Applied compressive stress under long term loads is within permissible limits

Check compressive stress on rail

Permissible comp.stress; σ_{cp1_adm} = **3.256** N/mm^2; Applied comp.stress; σ_{cp1_max} = **0.294** N/mm^2

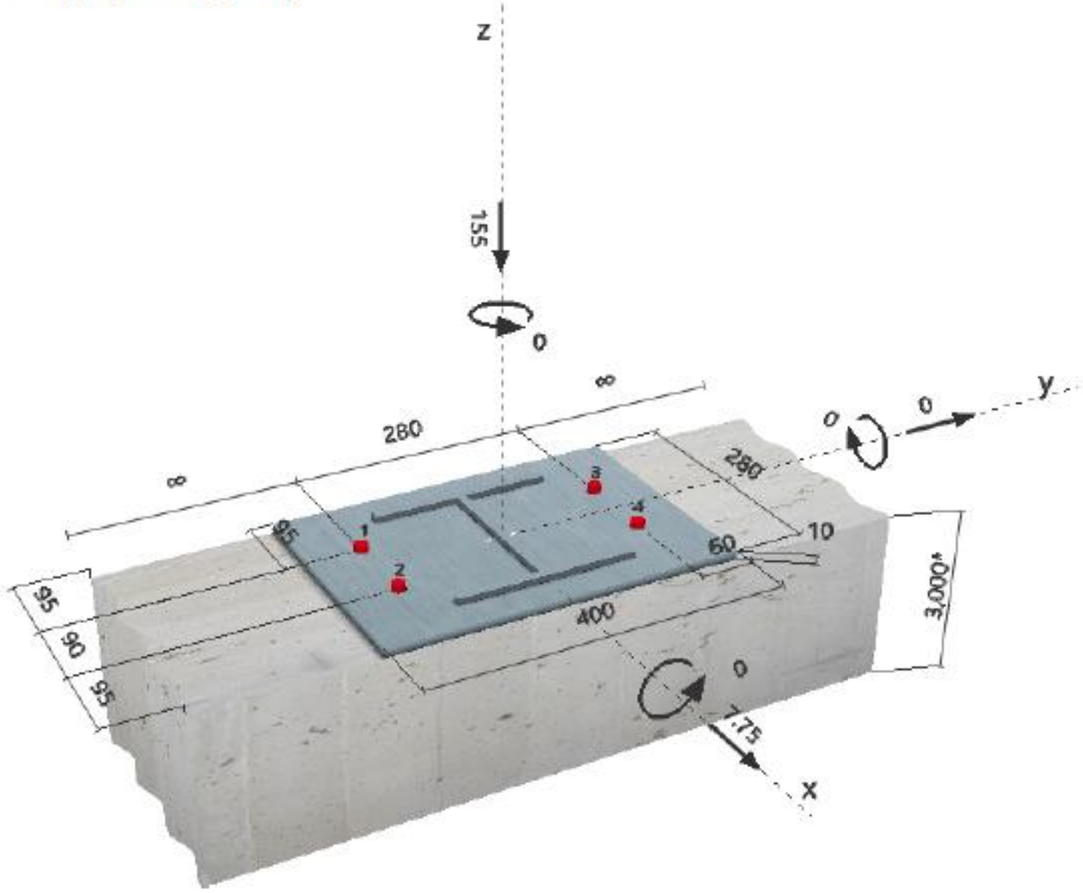
PASS - Applied compressive stress under long term loads is within permissible limits

Adopt external load bearing studwall minimum 38x140mm C24 at 400mm c/c to support masard roof construction and to be supported off onto floor joists below.

1 Input data

Anchor type and size:	**HIT-HY 200-A + HIT-V (8.8) M16**
Effective embedment depth:	$h_{ef,opti}$ = 80 mm ($h_{ef,limit}$ = 320 mm)
Material:	8.8
Approval No.:	ETA 11/0493
Issued I Valid:	03/02/2017 | -
Proof:	Design method ETAG BOND (EOTA TR 029)
Stand-off installation:	e_b = 0 mm (no stand-off); t = 10 mm
Baseplate:	l_x x l_y x t = 280 mm x 400 mm x 10 mm; (Recommended plate thickness: not calculated
Profile:	Advance UKC; (L x W x T x FT) = 203 mm x 204 mm x 11 mm x 11 mm
Base material:	cracked concrete, C30/37, $f_{c,cube}$ = 37.00 N/mm²; h = 3000 mm, Temp. short/long: 0/0 °C
Installation:	**hammer drilled hole, Installation condition: Dry**
Reinforcement:	No reinforcement or Reinforcement spacing >= 150 mm (any Ø) or >= 100 mm (Ø <= 10 mm) with longitudinal edge reinforcement d >= 12

Geometry [mm] & Loading [kN, kNm]



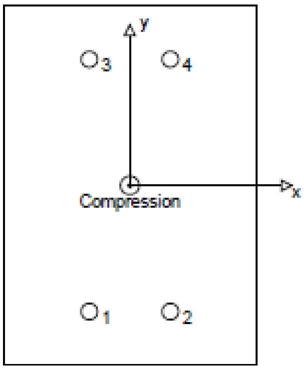
2 Load case/Resulting anchor forces

Load case: Design loads

Anchor reactions [kN]
Tension force: (+Tension, -Compression)

Anchor	Tension force	Shear force	Shear force x	Shear force y
1	0.000	1.938	1.938	0.000
2	0.000	1.938	1.938	0.000
3	0.000	1.938	1.938	0.000
4	0.000	1.938	1.938	0.000

max. concrete compressive strain: 0.05 [‰]
max. concrete compressive stress: 1.38 [N/mm²]
resulting tension force in (x/y)=(0/0): 0.000 [kN]
resulting compression force in (x/y)=(0/0): 155.000 [kN]



3 Tension load (EOTA TR 029, Section 5.2.2)

	Load [kN]	Capacity [kN]	Utilisation β_N [%]	Status
Steel failure*	N/A	N/A	N/A	N/A
Combined pullout-concrete cone failure**	N/A	N/A	N/A	N/A
Concrete cone failure**	N/A	N/A	N/A	N/A
Splitting failure**	N/A	N/A	N/A	N/A

* most unfavourable anchor **anchor group (anchors in tension)

4 Shear load (EOTA TR 029, Section 5.2.3)

	Load [kN]	Capacity [kN]	Utilisation β_V [%]	Status
Steel failure (without lever arm)*	1.938	50.400	4	OK
Steel failure (with lever arm)*	N/A	N/A	N/A	N/A
Pryout failure**	7.750	91.402	9	OK
Concrete edge failure in direction x+**	7.750	26.619	30	OK

* most unfavourable anchor **anchor group (relevant anchors)

4.1 Steel failure (without lever arm)

$V_{Rk,s}$ [kN]	$\gamma_{M,s}$	$V_{Rd,s}$ [kN]	V_{Sd} [kN]
63.000	1.250	50.400	1.938

4.2 Pryout failure (concrete cone relevant)

$A_{c,N}$ [mm²]	$A^0_{c,N}$ [mm²]	$c_{cr,N}$ [mm]	$s_{cr,N}$ [mm]	k-factor	k_1
134400	57600	120	240	2.000	7.200

$e_{c1,V}$ [mm]	$\psi_{ec1,N}$	$e_{c2,V}$ [mm]	$\psi_{ec2,N}$	$\psi_{s,N}$	$\psi_{re,N}$
0	1.000	0	1.000	0.938	1.000

$N^0_{Rk,c}$ [kN]	$\gamma_{M,c,p}$	$V_{Rd,cp}$ [kN]	V_{Sd} [kN]
31.338	1.500	91.402	7.750

4.3 Concrete edge failure in direction x+

h_{ef} [mm]	d_{nom} [mm]	k_1	α	β
80	16.0	1.700	0.092	0.070

c_1 [mm]	$A_{c,V}$ [mm²]	$A^0_{c,V}$ [mm²]
95	80513	40613

$\psi_{s,V}$	$\psi_{h,V}$	$\psi_{\alpha,V}$	$e_{c,V}$ [mm]	$\psi_{ec,V}$	$\psi_{re,V}$
1.000	1.000	1.000	0	1.000	1.200

$V^0_{Rk,c}$ [kN]	$\gamma_{M,c}$	$V_{Rd,c}$ [kN]	V_{Sd} [kN]
16.784	1.500	26.619	7.750

5 Displacements (highest loaded anchor)

Short term loading:

N_{Sk} = 0.000 [kN] δ_N = 0.000 [mm]
V_{Sk} = 2.870 [kN] δ_V = 0.115 [mm]
δ_{NV} = 0.115 [mm]

Long term loading:

N_{Sk} = 0.000 [kN] δ_N = 0.000 [mm]
V_{Sk} = 2.870 [kN] δ_V = 0.172 [mm]
δ_{NV} = 0.172 [mm]

Comments: Tension displacements are valid with half of the required installation torque moment for uncracked concrete! Shear displacements are valid without friction between the concrete and the baseplate! The gap due to the drilled hole and clearance hole tolerances are not included in this calculation!

The acceptable anchor displacements depend on the fastened construction and must be defined by the designer!

6 Warnings

- The anchor design methods in PROFIS Anchor require rigid anchor plates per current regulations (ETAG 001/Annex C, EOTA TR029, etc.). This means load re-distribution on the anchors due to elastic deformations of the anchor plate are not considered - the anchor plate is assumed to be sufficiently stiff, in order not to be deformed when subjected to the design loading. PROFIS Anchor calculates the minimum required anchor plate thickness with FEM to limit the stress of the anchor plate based on the assumptions explained above. The proof if the rigid base plate assumption is valid is not carried out by PROFIS Anchor. Input data and results must be checked for agreement with the existing conditions and for plausibility!
- Checking the transfer of loads into the base material is required in accordance with EOTA TR 029, Section 7!
- The design is only valid if the clearance hole in the fixture is not larger than the value given in Table 4.1 of EOTA TR029! For larger diameters of the clearance hole see Chapter 1.1. of EOTA TR029!
- The accessory list in this report is for the information of the user only. In any case, the instructions for use provided with the product have to be followed to ensure a proper installation.
- Drilled hole cleaning must be performed according to instructions for use (blow twice with oil-free compressed air (min. 6 bar), brush twice, blow twice with oil-free compressed air (min. 6 bar)).
- Characteristic bond resistances depend on short- and long-term temperatures.
- Please contact Hilti to check feasibility of HIT-V rod supply.
- Edge reinforcement is not required to avoid splitting failure

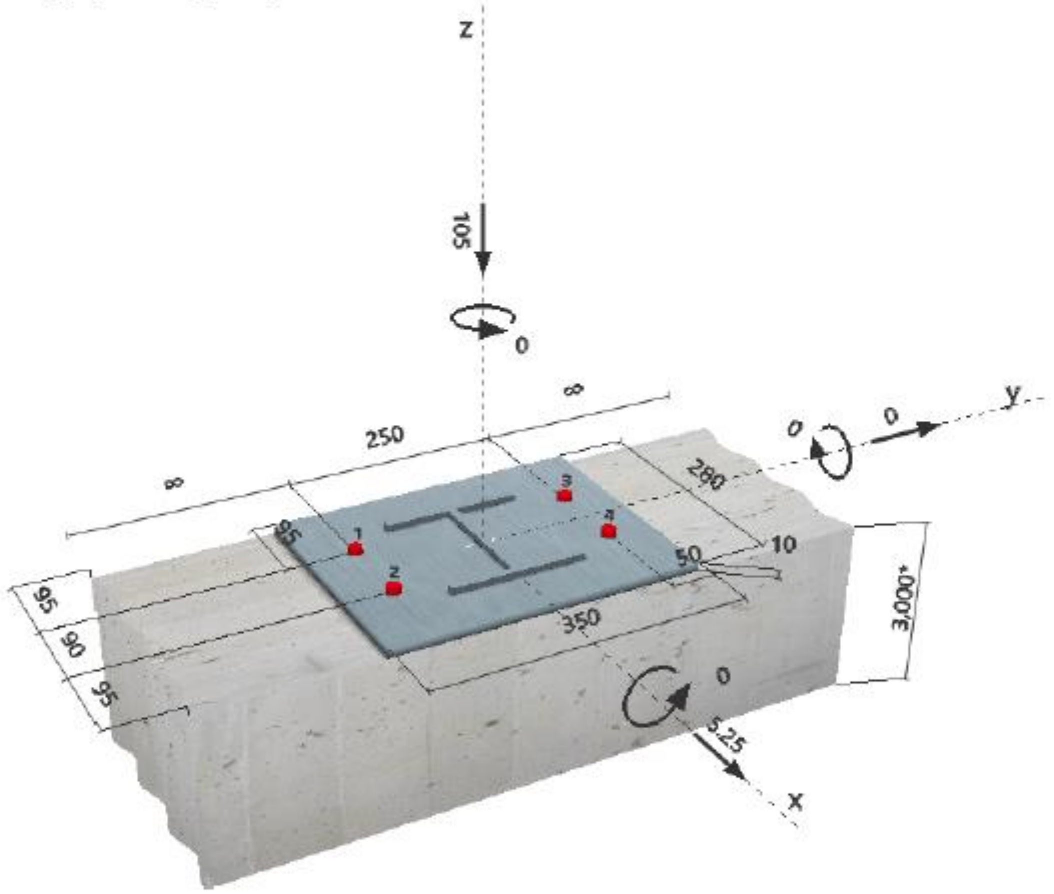
Fastening meets the design criteria!

Detail 1: connection detail for columns C101 to C104 onto RC retaining wall

1 Input data

Anchor type and size:	**HIT-HY 200-A + HIT-V (8.8) M16**
Effective embedment depth:	$h_{ef,opti}$ = 80 mm ($h_{ef,limit}$ = 320 mm)
Material:	8.8
Approval No.:	ETA 11/0493
Issued I Valid:	03/02/2017 | -
Proof:	Design method ETAG BOND (EOTA TR 029)
Stand-off installation:	e_b = 0 mm (no stand-off); t = 10 mm
Baseplate:	l_x x l_y x t = 280 mm x 350 mm x 10 mm; (Recommended plate thickness: not calculated
Profile:	Advance UKC; (L x W x T x FT) = 158 mm x 153 mm x 9 mm x 9 mm
Base material:	cracked concrete, C30/37, $f_{c,cube}$ = 37.00 N/mm²; h = 3000 mm, Temp. short/long: 0/0 °C
Installation:	**hammer drilled hole, Installation condition: Dry**
Reinforcement:	No reinforcement or Reinforcement spacing >= 150 mm (any Ø) or >= 100 mm (Ø <= 10 mm) with longitudinal edge reinforcement d >= 12

Geometry [mm] & Loading [kN, kNm]



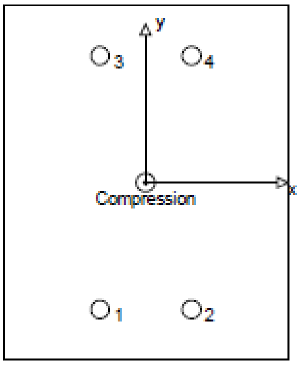
2 Load case/Resulting anchor forces

Load case: Design loads

Anchor reactions [kN]
Tension force: (+Tension, -Compression)

Anchor	Tension force	Shear force	Shear force x	Shear force y
1	0.000	1.313	1.313	0.000
2	0.000	1.313	1.313	0.000
3	0.000	1.313	1.313	0.000
4	0.000	1.313	1.313	0.000

max. concrete compressive strain: 0.04 [‰]
max. concrete compressive stress: 1.07 [N/mm²]
resulting tension force in (x/y)=(0/0): 0.000 [kN]
resulting compression force in (x/y)=(0/0): 105.000 [kN]



3 Tension load (EOTA TR 029, Section 5.2.2)

	Load [kN]	Capacity [kN]	Utilisation β_N [%]	Status
Steel failure*	N/A	N/A	N/A	N/A
Combined pullout-concrete cone failure**	N/A	N/A	N/A	N/A
Concrete cone failure**	N/A	N/A	N/A	N/A
Splitting failure**	N/A	N/A	N/A	N/A

* most unfavourable anchor **anchor group (anchors in tension)

4 Shear load (EOTA TR 029, Section 5.2.3)

	Load [kN]	Capacity [kN]	Utilisation β_V [%]	Status
Steel failure (without lever arm)*	1.313	50.400	3	OK
Steel failure (with lever arm)*	N/A	N/A	N/A	N/A
Pryout failure**	5.250	91.402	6	OK
Concrete edge failure in direction x+**	5.250	25.206	21	OK

* most unfavourable anchor **anchor group (relevant anchors)

4.1 Steel failure (without lever arm)

$V_{Rk,s}$ [kN]	$\gamma_{M,s}$	$V_{Rd,s}$ [kN]	V_{Sd} [kN]
63.000	1.250	50.400	1.313

4.2 Pryout failure (concrete cone relevant)

$A_{c,N}$ [mm²]	$A^0_{c,N}$ [mm²]	$c_{cr,N}$ [mm]	$s_{cr,N}$ [mm]	k-factor	k_1
134400	57600	120	240	2.000	7.200

$e_{c1,V}$ [mm]	$\psi_{ec1,N}$	$e_{c2,V}$ [mm]	$\psi_{ec2,N}$	$\psi_{s,N}$	$\psi_{re,N}$
0	1.000	0	1.000	0.938	1.000

$N^0_{Rk,c}$ [kN]	$\gamma_{M,c,p}$	$V_{Rd,cp}$ [kN]	V_{Sd} [kN]
31.338	1.500	91.402	5.250

4.3 Concrete edge failure in direction x+

h_{ef} [mm]	d_{nom} [mm]	k_1	α	β
80	16.0	1.700	0.092	0.070

c_1 [mm]	$A_{c,V}$ [mm²]	$A^0_{c,V}$ [mm²]
95	76238	40613

$\psi_{s,V}$	$\psi_{h,V}$	$\psi_{\alpha,V}$	$e_{c,V}$ [mm]	$\psi_{ec,V}$	$\psi_{re,V}$
1.000	1.000	1.000	0	1.000	1.200

$V^0_{Rk,c}$ [kN]	$\gamma_{M,c}$	$V_{Rd,c}$ [kN]	V_{Sd} [kN]
16.784	1.500	25.206	5.250

5 Displacements (highest loaded anchor)

Short term loading:

N_{Sk} = 0.000 [kN] δ_N = 0.000 [mm]
V_{Sk} = 1.944 [kN] δ_V = 0.078 [mm]
δ_{NV} = 0.078 [mm]

Long term loading:

N_{Sk} = 0.000 [kN] δ_N = 0.000 [mm]
V_{Sk} = 1.944 [kN] δ_V = 0.117 [mm]
δ_{NV} = 0.117 [mm]

Comments: Tension displacements are valid with half of the required installation torque moment for uncracked concrete! Shear displacements are valid without friction between the concrete and the baseplate! The gap due to the drilled hole and clearance hole tolerances are not included in this calculation!

The acceptable anchor displacements depend on the fastened construction and must be defined by the designer!

6 Warnings

- The anchor design methods in PROFIS Anchor require rigid anchor plates per current regulations (ETAG 001/Annex C, EOTA TR029, etc.). This means load re-distribution on the anchors due to elastic deformations of the anchor plate are not considered - the anchor plate is assumed to be sufficiently stiff, in order not to be deformed when subjected to the design loading. PROFIS Anchor calculates the minimum required anchor plate thickness with FEM to limit the stress of the anchor plate based on the assumptions explained above. The proof if the rigid base plate assumption is valid is not carried out by PROFIS Anchor. Input data and results must be checked for agreement with the existing conditions and for plausibility!
- Checking the transfer of loads into the base material is required in accordance with EOTA TR 029, Section 7!
- The design is only valid if the clearance hole in the fixture is not larger than the value given in Table 4.1 of EOTA TR029! For larger diameters of the clearance hole see Chapter 1.1. of EOTA TR029!
- The accessory list in this report is for the information of the user only. In any case, the instructions for use provided with the product have to be followed to ensure a proper installation.
- Drilled hole cleaning must be performed according to instructions for use (blow twice with oil-free compressed air (min. 6 bar), brush twice, blow twice with oil-free compressed air (min. 6 bar)).
- Characteristic bond resistances depend on short- and long-term temperatures.
- Please contact Hilti to check feasibility of HIT-V rod supply.
- Edge reinforcement is not required to avoid splitting failure

Fastening meets the design criteria!

Detail 2: connection detail for columns C105 & C106 onto RC retaining wall

Padstone Design – Eurocodes

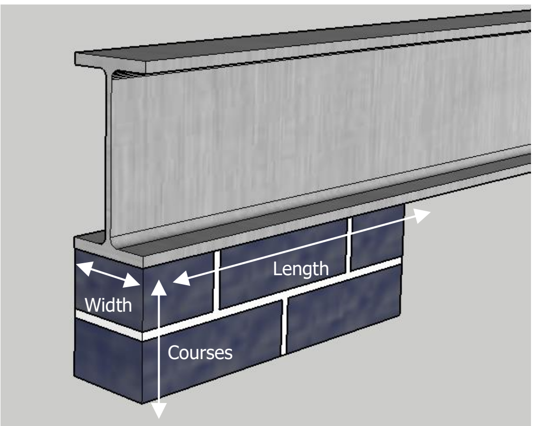
Assuming γ_m=3.0 f_k=(see table) N/mm^2 and Bearing Type 1
Required Bearing length = V_{ED}*γ_m/(1.25*f_k)/Padstone Width

100mm Blockwork/Brickwork Padstones

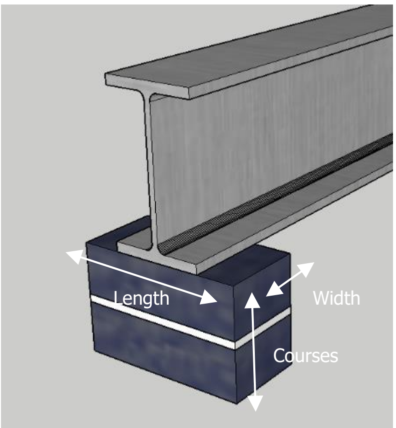
Maximum Allowable Load (kN)				Padstone Size		
Blockwork Strength = 3.6N/mm²	Blockwork Strength = 7.3N/mm²	Brickwork Strength = 10.4N/mm²	London Brick Strength = 3.6N/mm²	Length (mm)	Width (mm)	Courses
23	38	34	18	215	100	2
35	59	52	28	330	100	3
47	78	69	38	440	100	3
59	98	87	48	550	100	4
71	118	104	57	660	100	4
83	137	121	67	770	100	5
97	161	142	78	900	100	5

215mm Blockwork/Brickwork Padstones

Maximum Allowable Load (kN)				Padstone Size		
Blockwork Strength = 3.6N/mm²	Blockwork Strength = 7.3N/mm²	Brickwork Strength = 10.4N/mm²	London Brick Strength = 3.6N/mm²	Length (mm)	Width (mm)	Courses
36	63	73	40	215	215	2
56	97	112	62	330	215	3
74	130	149	82	440	215	3
93	162	187	103	550	215	4
112	195	224	124	660	215	4
131	227	262	144	770	215	5
153	266	306	169	900	215	5



Beam parallel to wall

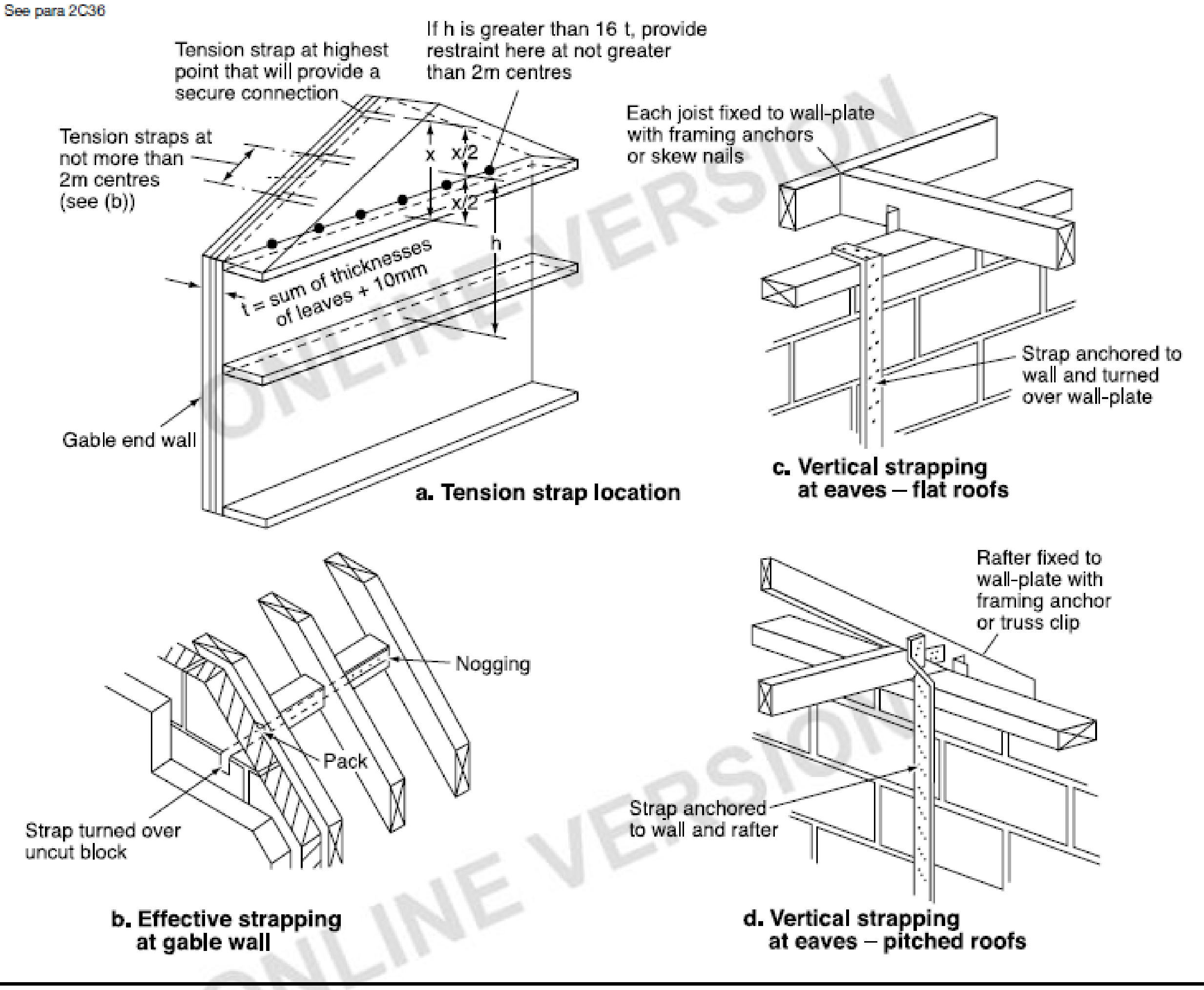


Beam perpendicular to wall

LATERAL RESTRAINT STRAPPING DETAILS:-

Diagram 16 **Lateral support at roof level**

See para 2C36



a. Tension strap location

c. Vertical strapping at eaves – flat roofs

b. Effective strapping at gable wall

d. Vertical strapping at eaves – pitched roofs

Diagram 15 **Lateral support by floors**

See para 2C35