Substructural Ltd
Suite LP20929
Lower Ground Floor
145-157 St John Street
London
EC1V 4PW

Tel: 02076 085740

www.substructural.co.uk

SUBSTRUCTURAL CALCULATIONS

(PREPARED BY JMS CONSULTING ENGINEERS LTD)

19 Rona Road, London, NW3 2HY

Project No: L15/088/04

Revision	Date	Changes
P1	16/07/15	Preliminary Issue for Planning Purposes
P2	24/07/15	Rear extension's steelwork frame amended to suit Architect's new layout
P3	30/07/15	Rear extension's steelwork frame now shown on key plans

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. JMS Engineers can act as Party Wall Surveyors in this instance and should be contacted accordingly.

- 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/m^2		Dead	Live
	Rafters, felt, insulation etc .	=	0.30 kN/m^2			
			0.95 kN/m^2/Cos 35 =	1.16 kN/m^2		
	Plasterboard	=		0.25 kN/m^2		
			TOTAL	=	**1.41 kN/m^2**	
	Attic	=		0.25		
	Roof snow loading	=	0.6*((60-35)/30)=	0.50 kN/m^2		
			TOTAL	=		**0.75 kN/m^2**
Roof :	Tiles	=	0.65 kN/m^2		Dead	Live
	Plasterboard	=	0.25 kN/m^2			
	Rafters, felt, insulation etc .	=	0.30 kN/m^2			
			1.20 kN/m^2/Cos 35 =	1.47 kN/m^2		
			TOTAL	=	**1.47 kN/m^2**	
	Attic	=		0.25		
	Roof snow loading	=	0.6*((60-35)/30)=	0.50 kN/m^2		
			TOTAL	=		**0.75 kN/m^2**
Roof:	Joists & boarding , finishes	=	0.35 kN/m^2			
(flat)	Plasterboard	=	0.25 kN/m^2			
			TOTAL	=	**0.60 kN/m^2**	
	Imposed	=	0.75 kN/m^2			
			TOTAL	=		**0.75 kN/m^2**
Floor:	PCU's	=	2.50 kN/m^2			
	Screed	=	1.80 kN/m^2			
	Plasterboard	=	0.25 kN/m^2			
			TOTAL	=	**4.55 kN/m^2**	
	Imposed loading		=		1.5	
	Partitions loading		=		1.0kN/m^2	
			TOTAL	=		**2.5 kN/m^2**
Floor:	Joists	=	0.15 kN/m^2			
	Boarding	=	0.1 kN/m^2			
	Plasterboard	=	0.25 kN/m^2			
			TOTAL	=	**0.5 kN/m^2**	
	Imposed loading		=		1.5	
	Partitions loading		=		0.5kN/m^2	
			TOTAL	=		**2.0 kN/m^2**
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, 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, 100mm brickwork	=	2.7*2.1	=5.67 kN/m		
	Plasterwork both sides	=	2.4*0.25*2	=1.2 kN/m		
			TOTAL	=	**6.87 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**	

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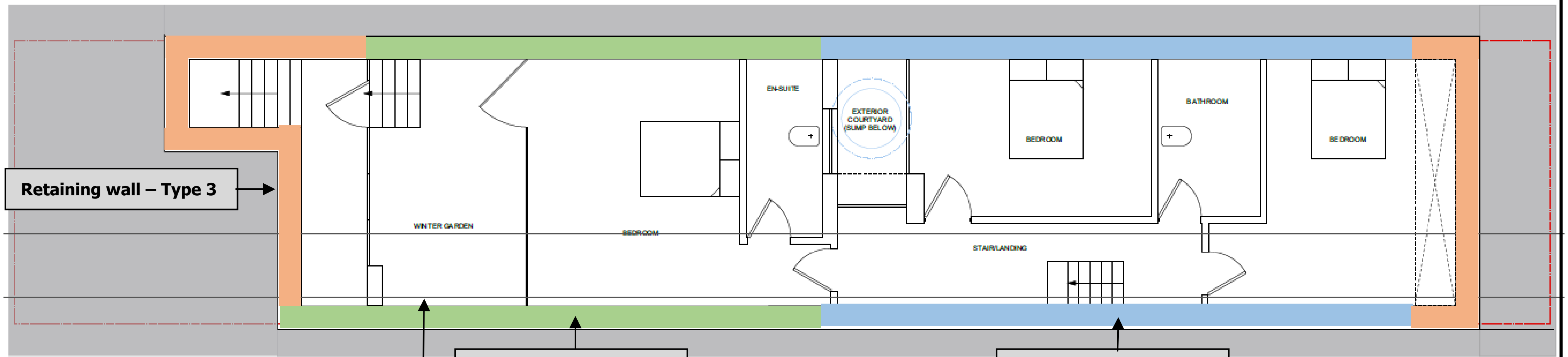
Job ref	: L15/088/04
Sheet	: **Structure / 4 -**
Made By	: Chris Papadakis
Date	: **July 2015**
Revision	: **P3**

Proposed Basement floor plan foundation layout



Retaining wall – Type 2

Retaining wall – Type 1

Concrete Bridge Above cantilevered from this wall.

Retaining wall types indicate different reinforcement schedule depending on the loadings acting on the retaining walls.

Provide 300mm thick Retaining wall with 300mm thick Base.

Note:-
Demolition of existing structure to occur in stages hence, temporary support is not proposed at this stage.

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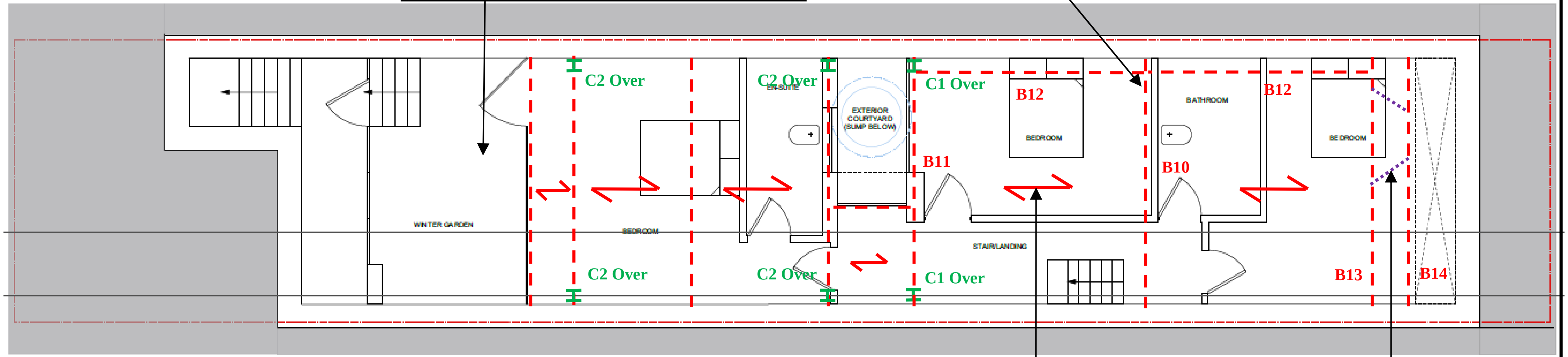
Proposed Basement floor plan showing structure over

Glass Extension:
Glass walls and roof to be designed by Specialist. Allow for area load of 1.5 kN/m^2 Live Load (Domestic Use)

Steel beams indicated thus. Refer to the table below for steel section.



Please refer to Pages 33-34 for the Isometric views of the steel frame.

Span of proposed floor joists indicated thus.
Provide 75x225mm deep C24 timber joists at 400mm Centres.

Prestressed Concrete Lintels to support bay window indicated thus.

STEEL SECTIONS	
Reference:-	**Size:-**
B10	203 x 203 UC 60
B11	203 x 203 UC 60
B12	203 x 203 UC 46
B13	254 x 254 UC 89
B14	203 x 203 UC 60

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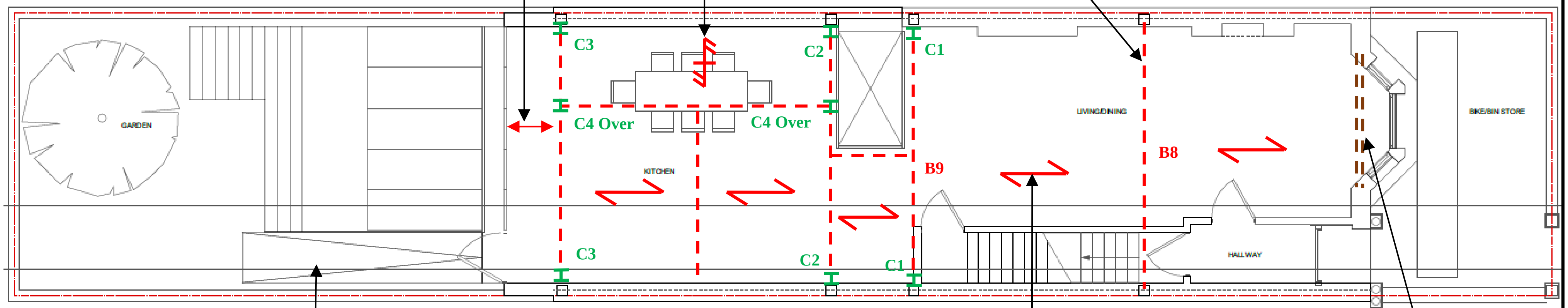
Job ref	: L15/088/04
Sheet	: **Structure / 6 -**
Made By	: Chris Papadakis
Date	: **July 2015**
Revision	: **P3**

Proposed Ground floor plan showing structure over

Span of proposed Glass Flat roof indicated thus.

Span of proposed Glass pitched roof indicated thus.

Steel beams indicated thus. Refer to the table below for steel section.



Concrete Bridge Above cantilevered from this wall.

Please refer to Pages 33-34 for the Isometric views of the steel frame.

Span of proposed floor joists indicated thus.
Provide 47x225mm deep C24 timber joists at 400mm Centres.

Multiple timber joists trimmers bolted together indicated thus.

STEEL SECTIONS	
Reference:-	**Size:-**
B8	203 x 203 UC 46
B9	254 x 254 UC 89
C1	203 x 203 UC 46

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Job ref	: L15/088/04
Sheet	: **Structure / 7 -**
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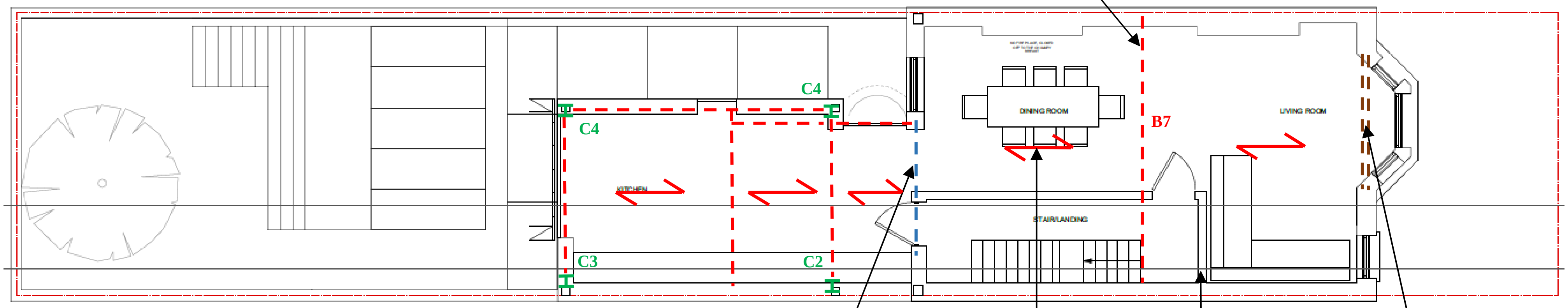
Proposed First floor plan showing structure over

Steel beam indicated thus. Refer to the table below for steel section.



Please refer to Pages 33-34 for the Isometric view of the steel frame.

Catnic lintel checked indicated thus.

Span of proposed floor joists indicated thus.
Provide 47x225mm deep C24 timber joists at 400mm Centres.

Multiple timber joists trimmers bolted together indicated thus.

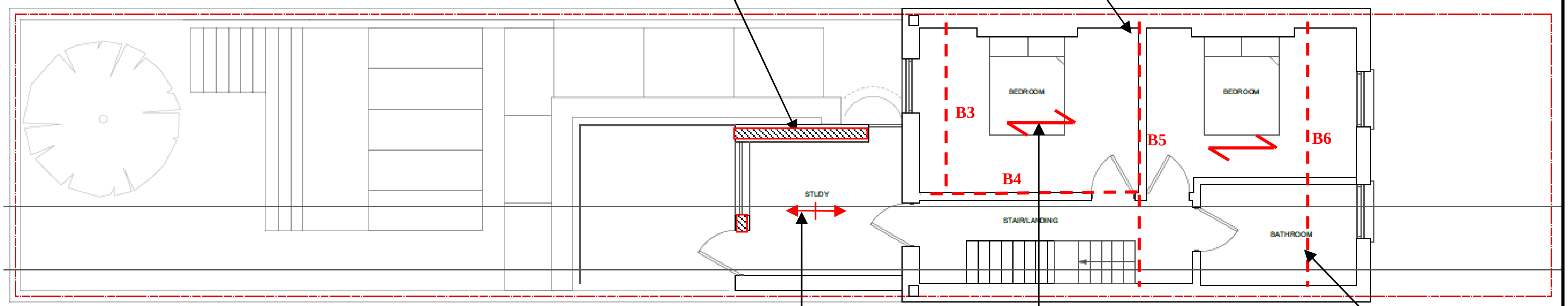
STEEL SECTIONS	
Reference:-	**Size:-**
B7	203 x 203 UC 46

Internal non load-bearing studwork indicated thus. Provide 38x89mm deep C24 Timber studs at 400mm Centres sheathed using 12mm OSB3 or Tructural Grade Ply.

Proposed Second floor plan showing structure over

External load-bearing studwork indicated thus. Provide 38x140mm deep C24 Timber studs at 400mm Centres sheathed using 12mm OSB3 or Tructural Grade Ply.

Internal non load-bearing studwork indicated thus. Provide 38x89mm deep C24 Timber studs at 400mm Centres sheathed using 12mm OSB3 or Tructural Grade Ply.



Please refer to Pages 33-34 for the Isometric view of the steel frame.

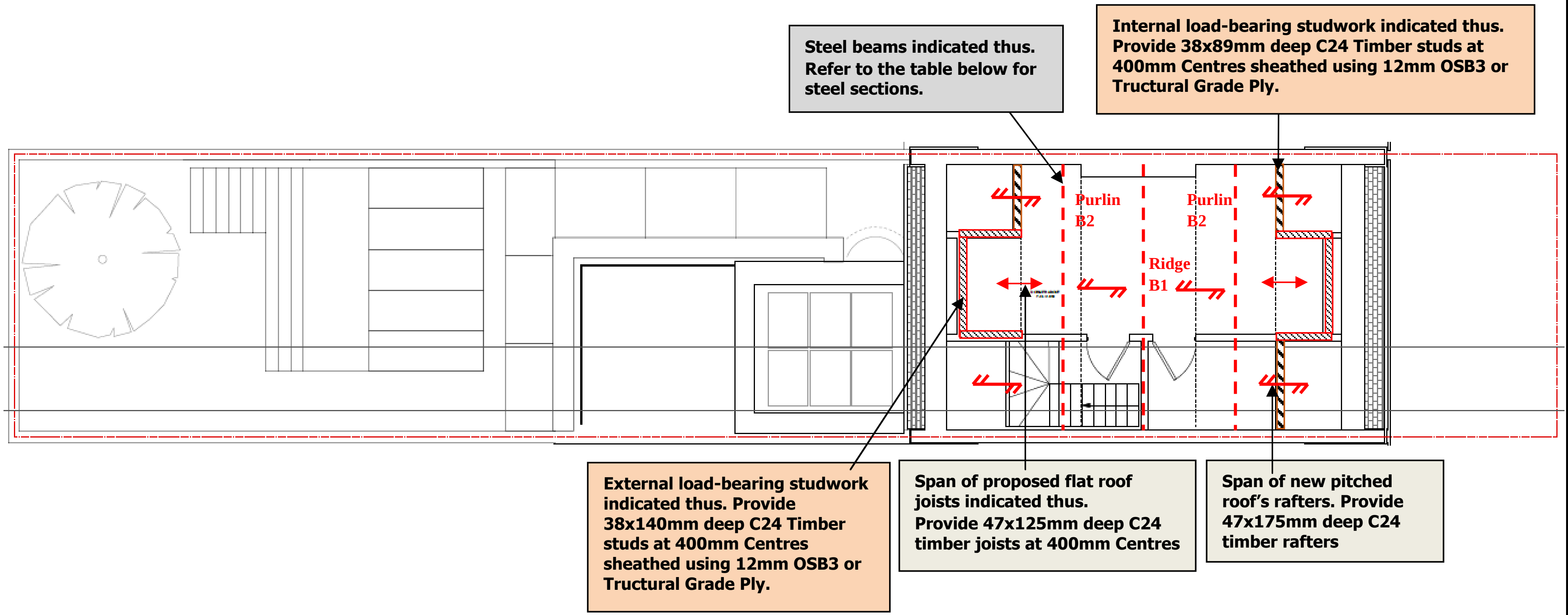
Span of proposed flat roof joists indicated thus.
Provide 47x175mm deep C24 timber joists at 400mm Centres

Span of proposed floor joists indicated thus.
Provide 47x225mm deep C24 timber joists at 400mm Centres.
Floor joists to support internal load-bearing studwork and roof above.
Double up joists under dormer cheeks.

Steel beams indicated thus. Refer to the table below for steel sections.

STEEL SECTIONS	
Reference:-	**Size:-**
B3	203 x 102 UB 23
B4	203 x 102 UB 23
B5	203 x 203 UC 46
B6	203 x 203 UC 46

Proposed Third floor plan showing structure over



STEEL SECTIONS	
Reference:-	**Size:-**
B1	203 x 102 UB 23
B2	152 x 152 UC 30

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Job ref	: L15/088/04
Sheet	: **Structure / 10 -**
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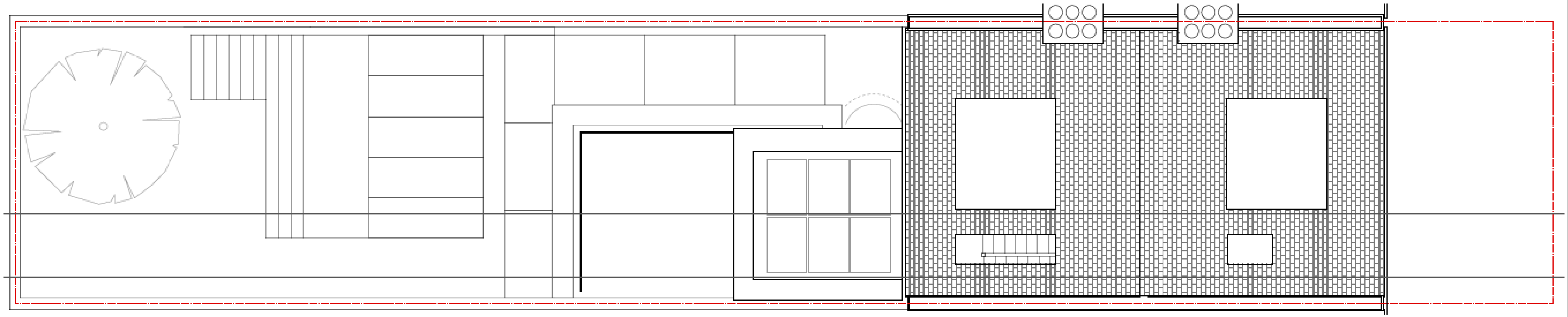
Date	: **July 2015**
Revision	: **P3**

Proposed Roof plan



Member Loading (in addition to the area loading as shown at appendix)

Load to B1

Main pitched roof	4/2*1.47=2.94	4/2*0.75=1.5	
Total	**2.94 kN/m Dead**	**1.50 kN/m Live**	**(Unfactored)**

Load to B2 (No dormer)

Main pitched roof	4.4/2*1.47=3.24	4.4/2*0.75=1.65	
Total	**3.24 kN/m Dead**	**1.65 kN/m Live**	**(Unfactored)**

Load to B2 (dormer)

Main pitched roof	2/2*1.47=1.47	2/2*0.75=0.75	
Flat roof	2.8/2*0.6=0.84	2.8/2*0.75=1.05	
Total	**2.31 kN/m Dead**	**1.80 kN/m Live**	**(Unfactored)**

Load to B3, B6

Flat roof	2.8/2*0.6=0.84	2.8/2*0.75=1.05	
External studwork	2.90		
Total	**3.74 kN/m Dead**	**1.05 kN/m Live**	**(Unfactored)**

Load to B4, B5

Internal studwork	1.01		
Total	**1.01 kN/m Dead**	**-**	**(Unfactored)**

Load to B7

Second floor	8.8/2*0.5=2.2	8.8/2*2=8.8	
Internal studwork	1.01		
Total	**3.21 kN/m Dead**	**8.80 kN/m Live**	**(Unfactored)**

Load to B8

First floor	8.8/2*0.5=2.2	8.3/2*2=8.8	
Total	**2.20 kN/m Dead**	**8.8 kN/m Live**	**(Unfactored)**

Load to B9

First floor	4.5/2*0.5=1.13	4.5/2*2=4.5	
Second floor	4.5/2*0.5=1.13	4.5/2*2=4.5	
Third floor	4.5/2*0.5=1.13	4.5/2*2=4.5	
Pitched roof	2.8/2*1.47=2.06	2.8/2*0.75=1.05	
External Wall	30.82		
Total	**36.27 kN/m Dead**	**14.55 kN/m Live**	**(Unfactored)**

Load to B10

Ground floor	8.8/2*2.6=11.44	8.8/2*1.5=6.6	
Total	**11.44 kN/m Dead**	**6.60 kN/m Live**	**(Unfactored)**

Load to B12

Chimney breast	43		
Total	**43.00 kN/m Dead**	**-**	**(Unfactored)**

AXIAL WITH MOMENTS (MEMBER)

B1

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

Member Loading and Member Forces

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Loading Combination : 1 UT + 1.35 D1 + 1.5 L1

D1 UDLY -002.940            ( kN/m )
L1 UDLY -001.500            ( kN/m )
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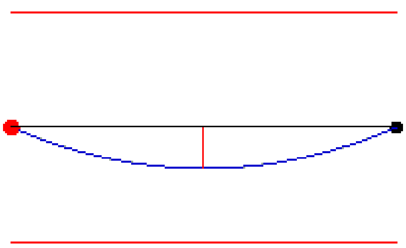

Member Forces in Load Case 1 and Maximum Deflection from Load Case 3						
Mem ber No.	**Node End1 End2**	**Axial Force (kN)**	**Shear Force (kN)**	**Bending Moment (kN.m)**	**Maximum Moment (kN.m @ m)**	**Maximum Deflection (mm @ m)**
1	1	0.087C	17.306	0.000	22.930	10.850
	2	0.087C	-17.306	0.000	@ 2.650	@ 2.650

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, 0.09, 22.93, 0	(Axial: Non-Slender)	Class 1
Auto Design Load Cases	1		

Local Capacity Check

$V_{y.Ed}/V_{pl.y.Rd}$	0.003 / 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	
$N_{pl.Rd} = A_g.f_y/\gamma_{M0}$	29.39 x 275/1 =	808.225 kN	
$n = N_{Ed}/N_{pl.Rd}$	0.087 / 808.225 =	0.000	OK
$W_{pl.N.y} = Fn(W_{pl.y}, A_{vy}, n)$	234.1, 12.371, 0	234.1 cm^3	
$M_{N.y.Rd} = W_{pl.N.y}.f_y/\gamma_{M0}$	234.1 x 275/1	64.378 kN.m	
$(M_{y.Ed}/M_{N.y.Rd})+(M_{z.Ed}/M_{N.z.Rd})$	$(22.925/64.378)^2+(0)^1=$	0.127	OK

Compression Resistance N.b.Rd

$\lambda y = \sqrt{A.fy/Ncr}$	$\sqrt{29.39x275/1553.76}$	0.722	
$N_{b.y.Rd} = Area.\chi.f_y/\gamma_{M1}$	29.39x0.837x275/10/1 =	676.770 kN	Curve a
$\lambda z = \sqrt{A.fy/Ncrz}$	$\sqrt{29.39x275/121.6}$	2.576	
$N_{b.z.Rd} = Area.\chi.f_y/\gamma_{M1}$	29.39x0.132x275/10/1 =	106.820 kN	Curve b
Let = Kt.Lx	1x5.3 =	5.3	
$\lambda T = \sqrt{A.fy/NcrT}$	$\sqrt{29.39x275/881.43}$	0.958	
$N_{b.T.Rd}= Area.\chi.fy/\gamma_{M1}$	29.39x0.624x275/10/1 =	504.438 kN	Curve b

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

C_1= fn(M_1, M_2, M_o, ψ, μ)	0.0, 0.0, 22.9, 0.882, 300.000	1.127	Uniform
$C_{mLT}=0.95+0.05\alpha_h$	M_h= 0.02, M_s= 22.93, ψ = 0.882, α_s= 0.001	0.95	Table B.3
C_{mz}=Max(0.6+0.4ψ, 0.4)	M = 0, ψ = 1.000	1	Table B.3
$C_{my}=0.95+0.05\alpha_h$	M_h= 0, M_s= 22.93, ψ = 1.000, α_s= 0.000	0.95	Table B.3

Lateral Buckling Check M.b.Rd

Le = 1.2L+2D	1.2 x 5.3 + 2 x 0.203 =	6.766 m	
M_{cr}= Fn($C_1, L_e, I_z, I_t, I_w, E$)	1.127, 6.766, 164.8, 7.019, 0.01537, 210000	24.559 kN.m	
$\lambda_{LT} = \sqrt{W.f_y/M_{cr}}$	$\sqrt{234.1 \times 275 / 24.559}$	1.619	
χ_{LT}= Fn(λ_{LT}, λ_{LT5950})	1.619, 1.687	0.380	Curve b
χ_{LT}.mod = Fn($\chi_{LT}, \lambda_{LT}, k_c, f$)	0.380, 1.619, 0.942, 1.000	0.380	6.3.2.3
$M_{b.Rd} = \chi W_{pl.y}.f_y \leq M_{c.y.Rd}$	0.380 x 234.1 x 275 ≤ 64.378 =	24.440 kN.m	

Buckling Resistance

$U_{N.y} = N_{Ed}/(\chi_y.N_{Rk}/\gamma_{M1})$	0.087 / 676.77	0.000	OK
$U_{N.z} = N_{Ed}/(\chi_z.N_{Rk}/\gamma_{M1})$	0.087 / 106.82	0.001	OK
$U_{M.y} = M_{y.Ed}/(\chi_{LT}.M_{y.Rk}/\gamma_{M1})$	22.925 / 24.44	0.938	OK
$U_{M.z} = M_{z.Ed}/(M_{z.Rk}/\gamma_{M1})$	0 / 13.695	0.000	OK

$k_{yy}=C_{my}\{1+(\lambda_y-0.2)U_{N.y}\}$		0.950	
$k_{zz}=C_{mz}\{1+1.4U_{N.z}\}$		1.001	
$k_{yz}=0.6\ k_{zz}$		0.601	
$k_{zy}=1-\{0.1\lambda_z/(C_{mLT}-0.25)\}U_{N.z}$		1.000	
$U_{Ny}+k_{yy}.U_{M.y}+k_{yz}.U_{M.z}$	0.000+0.950x0.938+0.601x0.000	0.891	OK
$U_{Nz}+k_{zy}.U_{M.y}+k_{zz}.U_{M.z}$	0.001+1.000x0.938+1.001x0.000	0.939	OK

Deflection Check - Load Case 3

Deflection Limits (Internal Beams)	δ ≤ 5300/360 = 14.7 mm Live (Case 2)	3.48 mm	OK
	δ ≤ 5300/250 = 21.2 mm D+L (Case 3)	10.85 mm	OK

Provide Section (23.07 kg/m) 203x102 UB 23 [S 275]

AXIAL WITH MOMENTS (MEMBER)

B2

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

Member Loading and Member Forces

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Loading Combination : 1 UT + 1.35 D1 + 1.5 L1

Part 1
D1 UDLY -003.240           ( kN/m )
L1 UDLY -001.650           ( kN/m )
Part 2
D1 UDLY -002.310           ( kN/m )
L1 UDLY -001.800           ( kN/m )
Part 3
D1 UDLY -003.240           ( kN/m )
L1 UDLY -001.650           ( kN/m )
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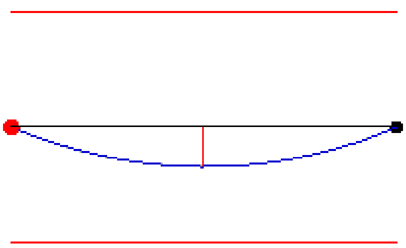


Member Forces in Load Case 1 and Maximum Deflection from Load Case 3						
Mem ber No.	**Node End1 End2**	**Axial Force (kN)**	**Shear Force (kN)**	**Bending Moment (kN.m)**	**Maximum Moment (kN.m @ m)**	**Maximum Deflection (mm @ m)**
	1	0.025C	18.061	0.000	23.194	13.243
	4	0.149C	-18.224	0.000	@ 2.660	@ 2.660

Classification and Effective Area (EN 1993: 2006)

Section (30.03 kg/m)	152x152 UC 30 [S 275]		
Class = Fn(b/T,d/t,f_y,N,M_y,M_z)	8.13, 19.02, 275, 0.15, 23.19, 0	(Axial: Non-Slender)	Class 1
Auto Design Load Cases	1		

Local Capacity Check

$V_{y.Ed}/V_{pl.y.Rd}$	0.124 / 183.454 =	0.001	Low Shear
$M_{c.y.Rd} = f_y.W_{pl.y}/\ \gamma_{M0}$	275 x 247.7/1	68.118 kN.m	
$N_{pl.Rd} = A_g.f_y/\ \gamma_{M0}$	38.26 x 275/1 =	1052.15 kN	
$n = N_{Ed}/N_{pl.Rd}$	0.025 / 1052.15 =	0.000	OK
$W_{pl.N.y} = Fn(W_{pl.y}, A_{vy}, n)$	247.7, 11.555, 0	247.7 cm³	
$M_{N.y.Rd} = W_{pl.N.y}.f_y/\ \gamma_{M0}$	247.7 x 275/1	68.118 kN.m	
$(M_{y.Ed}/M_{N.y.Rd})+(M_{z.Ed}/M_{N.z.Rd})$	$(23.192/68.118)^2+(0)^1=$	0.116	OK

Compression Resistance N.b.Rd

λy = √A.fy/Ncr	√38.26x275/1290.42	0.903	
$N_{b.y.Rd}$ = Area.χ.f_y/ γ_{M1}	38.26x0.659x275/10/1 =	693.565 kN	Curve b
λz = √A.fy/Ncrz	√38.26x275/414.23	1.594	
$N_{b.z.Rd}$ = Area.χ.f_y/ γ_{M1}	38.26x0.286x275/10/1 =	300.842 kN	Curve c
Let = Kt.Lx	1x5.3 =	5.3	
λT = √A.fy/NcrT	√38.26x275/1783.14	0.768	
$N_{b.T.Rd}$= Area.χ.fy/ γ_{M1}	38.26x0.682x275/10/1 =	717.731 kN	Curve c

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

C_1= fn(M_1, M_2, M_o, ψ,μ)	0.0, 0.0, 23.2, 0.941, 300.000	1.127	Uniform

$C_{mLT}=0.95+0.05\alpha_h$	M_h= 0.02, M_s= 23.19, ψ = 0.941, α_s= 0.001	0.95	Table B.3
$C_{mz}=Max(0.6+0.4\psi, 0.4)$	M = 0, ψ = 1.000	1	Table B.3
$C_{my}=0.95+0.05\alpha_h$	M_h= 0, M_s= 23.19, ψ = 1.000, α_s= 0.000	0.95	Table B.3

Lateral Buckling Check M.b.Rd

Le = 1.2L+2D	1.2 x 5.3 + 2 x 0.158 =	6.675 m	
M_{cr}= Fn(C_1,L_e,I_z,I_t,I_w,E)	1.127, 6.675, 561.4, 10.52, 0.03075, 210000	57.376 kN.m	
$\lambda_{LT}=\sqrt{W.f_y/M_{cr}}$	√ 247.7 x 275 / 57.376	1.090	
χ_{LT}= Fn(λ_{LT}, λ_{LT5950})	1.090, 1.049	0.645	Curve b
χ_{LT}.mod = Fn(χ_{LT},λ_{LT},k_c,f)	0.645, 1.090, 0.942, 0.976	0.661	6.3.2.3
$M_{b.Rd} = \chi\ W_{pl.y}.f_y \leq M_{c.y.Rd}$	0.661 x 247.7 x 275 ≤ 68.118 =	45.015 kN.m	

Buckling Resistance

$U_{N.y} = N_{Ed}/(\chi_y.N_{Rk}/\gamma_{M1})$	0.149 / 693.565	0.000	OK
$U_{N.z} = N_{Ed}/(\chi_z.N_{Rk}/\gamma_{M1})$	0.149 / 300.842	0.000	OK
$U_{M.y} = M_{y.Ed}/(\chi_{LT}.M_{y.Rk}/\gamma_{M1})$	23.192 / 45.015	0.515	OK
$U_{M.z} = M_{z.Ed}/(M_{z.Rk}/\gamma_{M1})$	0 / 30.69	0.000	OK
$k_{yy}=C_{my}\{1+(\lambda_y-0.2)U_{N.y}\}$		0.950	
$k_{zz}=C_{mz}\{1+1.4U_{N.z}\}$		1.001	
$k_{yz}=0.6\ k_{zz}$		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+0.950x0.515+0.600x0.000	0.490	OK
$U_{Nz}+k_{zy}.U_{M.y}+k_{zz}.U_{M.z}$	0.000+1.000x0.515+1.001x0.000	0.516	OK

Deflection Check - Load Case 3

Deflection Limits (Internal Beams)	δ ≤ 5300/360 = 14.7 mm Live (Case 2)	4.86 mm	OK
	δ ≤ 5300/250 = 21.2 mm D+L (Case 3)	13.24 mm	OK

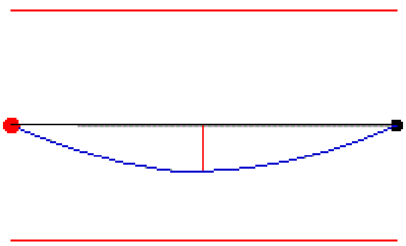
Provide Section (30.03 kg/m) **152x152 UC 30 [S 275]**

AXIAL WITH MOMENTS (MEMBER)

B3

Members 13-18 (N.2-N.3) @ Level 1 in Load Case 1

Member Loading and Member Forces



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

Part 1
D1 D      078.500           ( kN/m³)
Part 2
D1 D      078.500           ( kN/m³)
Part 3
D1 D      078.500           ( kN/m³)
Part 4
D1 D      078.500           ( kN/m³)
Part 5
D1 D      078.500           ( kN/m³)
D1 UDLY  -003.740           ( kN/m )
L1 UDLY  -001.050           ( kN/m )
Part 6
D1 D      078.500           ( kN/m³)
D1 UDLY  -003.740           ( kN/m )
L1 UDLY  -001.050           ( kN/m )
```

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

Member No.	Node End1 End2	Axial Force (kN)	Torque Moment (kN.m)	Shear Force (kN) y-y	Shear Force (kN) z-z	Bending Moment (kN.m) y-y	Bending Moment (kN.m) z-z	Maximum Moment (kN.m @ m) y-y	Maximum Moment (kN.m @ m) z-z	Maximum Deflection (mm @ m)
	2	0.34T	0.00	26.27	0.00	0.00	0.00	25.36	0.00	5.10
	14	0.11T	0.00	-27.20	0.00	0.00	0.00	@ 1.672	@ 0.000	@ 1.740

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, 25.36, 0	(Axial: Non-Slender)	Class 1
Auto Design Load Cases	1		

Local Capacity Check

$V_{y.Ed}/V_{pl.y.Rd}$	0.459 / 196.416 =	0.002	Low Shear
$M_{c.y.Rd} = f_y.W_{pl.y}/\gamma_{M0}$	275 x 234.1/1	64.378 kN.m	
$N_{pl.Rd} = Ag.fy/\gamma_{M0}$	29.39x275/1	808.225 kN	
$n = N_{Ed}/N_{pl.Rd}$	-0.343 / 808.225 =	0.000	OK
$W_{pl.N.y} = Fn(W_{pl.y}, A_{vy}, n)$	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	
$W_{pl.N.z} = Fn(W_{pl.z}, A_{vz}, n)$	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	
$(M_{y.Ed}/M_{N.y.Rd})+(M_{z.Ed}/M_{N.z.Rd})$	$(25.349/64.378)^2+(0.001/13.695)^1=$	0.155	OK

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

C_1= fn(M_1, M_2, M_o, ψ,μ)	0.0, 0.0, 25.3, 0.862, 300.000	1.127	Uniform
$C_{mLT}=0.95+0.05\alpha_h$	M_h= 0.03, M_s= 25.35, ψ = 0.862, α_s= 0.001	0.95	Table B.3
$C_{mz}=Max(0.6+0.4\psi, 0.4)$	M = 0, ψ = 0.000	0.6	Table B.3
$C_{my}=0.95+0.05\alpha_h$	M_h= 0, M_s= 25.35, ψ = 0.000, α_s= 0.000	0.95	Table B.3

Lateral Buckling Check M.b.Rd

Le = 1.2L+2D	1.2 x 3.5 + 2 x 0.203 =	4.606 m	
M_{cr}= Fn(C_1,L_e,I_z,I_t,I_w,E)	1.127, 4.606, 164.8, 7.019, 0.01537, 210000	38.290 kN.m	
$\lambda_{LT}= \sqrt{W.f_y/M_{cr}}$	√ 234.1 x 275 / 38.29	1.297	
χ_{LT}= Fn(λ_{LT}, λ_{LT5950})	1.297, 1.350	0.525	Curve b
χ_{LT}.mod = Fn(χ_{LT},λ_{LT},k_c,f)	0.525, 1.297, 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 234.1 x 275 ≤ 64.378 =	34.328 kN.m	

Buckling Resistance

$U_{N.y} = N_{Ed}/(\chi_y.N_{Rk}/\gamma_{M1})$	0 / 752.78	0.000	OK
$U_{N.z} = N_{Ed}/(\chi_z.N_{Rk}/\gamma_{M1})$	0 / 224.486	0.000	OK
$U_{M.y} = M_{y.Ed}/(\chi_{LT}.M_{y.Rk}/\gamma_{M1})$	25.349 / 34.328	0.738	OK
$U_{M.z} = M_{z.Ed}/(M_{z.Rk}/\gamma_{M1})$	0.001 / 13.695	0.000	OK
$k_{yy}=C_{my}\{1+(\lambda_y-0.2)U_{N.y}\}$		0.950	
$k_{zz}=C_{mz}\{1+1.4U_{N.z}\}$		0.600	
$k_{yz}=0.6\ k_{zz}$		0.360	
$k_{zy}= 1- \{0.1\lambda_z/(C_{mLT}-0.25)\}U_{N.z}$		1.000	
$U_{Ny}+k_{yy}.U_{M.y}+k_{yz}.U_{M.z}$	0.000+0.950x0.738+0.360x0.000	0.702	OK
$U_{Nz}+k_{zy}.U_{M.y}+k_{zz}.U_{M.z}$	0.000+1.000x0.738+0.600x0.000	0.738	OK

Deflection Check - Load Case 3

Deflection Limits (Internal Beams)	δ ≤ 3500/360 = 9.7 mm Live (Case 2)	2.27 mm	OK
	δ ≤ 3500/250 = 14 mm D+L (Case 3)	5.1 mm	OK

Provide Section (23.07 kg/m) **203x102 UB 23 [S 275]**

AXIAL WITH MOMENTS (MEMBER)

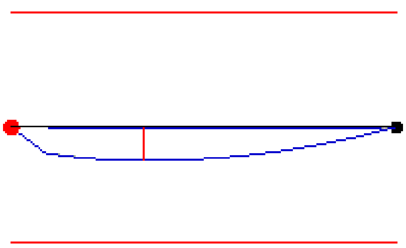
B4

Members 34-36 (N.6-N.14) @ 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 -000.450            ( kN/m )
L1 UDLY -001.350            ( kN/m )
D1 D     078.500            ( kN/m³)
Part 2
D1 UDLY -000.450            ( kN/m )
L1 UDLY -001.350            ( kN/m )
D1 D     078.500            ( kN/m³)
D1 UDLY -002.900            ( kN/m )
Part 3
D1 UDLY -000.450            ( kN/m )
L1 UDLY -001.350            ( kN/m )
D1 D     078.500            ( kN/m³)
D1 UDLY -001.010            ( kN/m )
```



Member Forces in Load Case 1 and Maximum Deflection from Load Case 3										
Mem ber No.	Node End1 End2	Axial Force (kN)	Torque Moment (kN.m)	Shear Force (kN) y-y	Shear Force (kN) z-z	Bending Moment (kN.m) y-y	Bending Moment (kN.m) z-z	Maximum Moment (kN.m @ m) y-y	Maximum Moment (kN.m @ m) z-z	Maximum Deflection (mm @ m)
	6	0.00C	0.00	36.12	0.00	0.00	0.00	18.60	0.05	6.32
	23	0.00T	0.00	-12.72	-0.02	-0.18	0.00	@ 1.530	@ 1.500	@ 2.130

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, 18.6, 0.05	(Axial: Non-Slender)	Class 1
Auto Design Load Cases	1		

Local Capacity Check

$V_{y.Ed}/V_{pl.y.Rd}$	0.04 / 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	
$V_{z.Ed}/V_{pl.z.Rd}$	0.015 / 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	808.225 kN	
$n = N_{Ed}/N_{pl.Rd}$	-0.001 / 808.225 =	0.000	OK
$W_{pl.N.y} = Fn(W_{pl.y}, A_{vy}, n)$	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	
$W_{pl.N.z} = Fn(W_{pl.z}, A_{vz}, n)$	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	
$(M_{y.Ed}/M_{N.y.Rd})+(M_{z.Ed}/M_{N.z.Rd})$	$(18.596/64.378)^2+(0.046/13.695)^1=$	0.087	OK

Compression Resistance N.b.Rd

$\lambda y = \sqrt{A.fy/Ncr}$	$\sqrt{29.39x275/2155.32}$	0.613	
$N_{b.y.Rd} = Area.\chi.f_y/\gamma_{M1}$	29.39x0.885x275/10/1 =	715.373 kN	Curve a
$\lambda z = \sqrt{A.fy/Ncrz}$	$\sqrt{29.39x275/168.68}$	2.187	
$N_{b.z.Rd} = Area.\chi.f_y/\gamma_{M1}$	29.39x0.178x275/10/1 =	144.177 kN	Curve b
Let = Kt.Lx	1x4.5 =	4.5	
$\lambda T = \sqrt{A.fy/NcrT}$	$\sqrt{29.39x275/938.31}$	0.928	
$N_{b.T.Rd}= Area.\chi.fy/\gamma_{M1}$	29.39x0.643x275/10/1 =	519.767 kN	Curve b

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

$C_1= fn(M_1, M_2, M_o, \psi, \mu)$	0.0, -0.2, 18.0, -0.205, -108.717	1.147	Uniform
$C_{mLT}=0.95+0.05\alpha_h(1+2\psi)$	M_h= -0.17, M_s= 18.03, ψ = -0.205, α_s= -0.009	0.95	Table B.3
$C_{mz}=Max(0.6+0.4\psi, 0.4)$	M = 0, ψ = 0.000	0.6	Table B.3
$C_{my}=0.95+0.05\alpha_h$	M_h= -0.18, M_s= 17.53, ψ = 0.000, α_s= -0.010	0.949	Table B.3

Lateral Buckling Check M.b.Rd

Le = 1.2L+2D	1.2 x 4.5 + 2 x 0.203 =	5.806 m	

M_{cr}= Fn(C_1,L_e,I_z,I_t,I_w,E)	1.147, 5.806, 164.8, 7.019, 0.01537, 210000	29.689 kN.m	
λ_{LT}= √ W.f_y/M_{cr}	√ 234.1 x 275 / 29.689	1.473	
χ_{LT}= Fn(λ_{LT}, λ_{LT5950})	1.473, 1.547	0.439	Curve b
χ_{LT}.mod = Fn(χ_{LT},λ_{LT},k_c,f)	0.439, 1.473, 0.934, 0.997	0.441	6.3.2.3
$M_{b.Rd}$ = χ $W_{pl.y}$.f_y≤ $M_{c.y.Rd}$	0.441 x 234.1 x 275 ≤ 64.378 =	28.368 kN.m	

Buckling Resistance

$U_{N.y}$ = N_{Ed}/(χ_y.N_{Rk}/γ_{M1})	0 / 715.373	0.000	OK
$U_{N.z}$ = N_{Ed}/(χ_z.N_{Rk}/γ_{M1})	0 / 144.177	0.000	OK
$U_{M.y}$ = $M_{y.Ed}$/(χ_{LT}.$M_{y.Rk}$/γ_{M1})	18.596 / 28.368	0.656	OK
$U_{M.z}$ = $M_{z.Ed}$/($M_{z.Rk}$/γ_{M1})	0.046 / 13.695	0.003	OK
k_{yy}=C_{my}{1+(λ_y-0.2)$U_{N.y}$}		0.949	
k_{zz}=C_{mz}{1+1.4$U_{N.z}$}		0.600	
k_{yz}=0.6 k_{zz}		0.360	
k_{zy}= 1- {0.1λ_z/(C_{mLT}-0.25)}$U_{N.z}$		1.000	
U_{Ny}+k_{yy}.$U_{M.y}$+k_{yz}.$U_{M.z}$	0.000+0.949x0.656+0.360x0.003	0.624	OK
U_{Nz}+k_{zy}.$U_{M.y}$+k_{zz}.$U_{M.z}$	0.000+1.000x0.656+0.600x0.003	0.658	OK

Deflection Check - Load Case 3

Deflection Limits (Internal Beams)	δ ≤ 4500/360 = 12.5 mm Live (Case 2)	2.65 mm	OK
	δ ≤ 4500/250 = 18 mm D+L (Case 3)	6.32 mm	OK

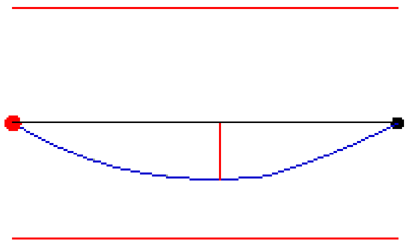
Provide Section (23.07 kg/m) **203x102 UB 23 [S 275]**

AXIAL WITH MOMENTS (MEMBER)

B5

Members 1-12 (N.15-N.16) @ Level 1 in Load Case 1

Member Loading and Member Forces



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

Part 1
D1 D      078.500           ( kN/m³)
Part 2
D1 D      078.500           ( kN/m³)
Part 3
D1 D      078.500           ( kN/m³)
Part 4
D1 D      078.500           ( kN/m³)
Part 5
D1 D      078.500           ( kN/m³)
Part 6
D1 D      078.500           ( kN/m³)
Part 7
D1 D      078.500           ( kN/m³)
Part 8
D1 D      078.500           ( kN/m³)
D1 UDLY -001.010            ( kN/m )
Part 9
D1 D      078.500           ( kN/m³)
D1 UDLY -001.010            ( kN/m )
Part 10
D1 D      078.500           ( kN/m³)
D1 UDLY -001.010            ( kN/m )
Part 11
D1 D      078.500           ( kN/m³)
D1 UDLY -001.010            ( kN/m )
Part 12
D1 D      078.500           ( kN/m³)
D1 UDLY -001.010            ( kN/m )
```

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

Mem ber No.	Node End1 End2	Axial Force (kN)	Torque Moment (kN.m)	Shear Force (kN) y-y	Shear Force (kN) z-z	Bending Moment (kN.m) y-y	Bending Moment (kN.m) z-z	Maximum Moment (kN.m @ m) y-y	Maximum Moment (kN.m @ m) z-z	Maximum Deflection (mm @ m)
	15	0.01C	0.00	49.37	0.00	0.00	0.00	66.08	0.00	14.03
	33	0.43C	0.00	-43.69	0.00	0.00	0.00	@ 2.775	@ 0.000	@ 2.658

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, 0.43, 66.08, 0	(Axial: Non-Slender)	Class 1
Auto Design Load Cases	1		

Local Capacity Check

$V_{y.Ed}/V_{pl.y.Rd}$	1.049 / 269.498 =	0.004	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	
$n = N_{Ed}/N_{pl.Rd}$	0.007 / 1615.075 =	0.000	OK
$W_{pl.N.y} = Fn(W_{pl.y}, A_{vy}, n)$	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	
$(M_{y.Ed}/M_{N.y.Rd})+(M_{z.Ed}/M_{N.z.Rd})$	$(66.039/136.785)^2+(0)^1=$	0.233	OK

Compression Resistance N.b.Rd

$\lambda y = \sqrt{A.fy/Ncr}$	√58.73x275/3372.49	0.692	
$N_{b.y.Rd} = Area.\chi.f_y/\ \gamma_{M1}$	58.73x0.788x275/10/1 =	1272.861 kN	Curve b
$\lambda z = \sqrt{A.fy/Ncrz}$	√58.73x275/1144.48	1.188	
$N_{b.z.Rd} = Area.\chi.f_y/\ \gamma_{M1}$	58.73x0.44x275/10/1 =	710.090 kN	Curve c
Let = Kt.Lx	1x5.3 =	5.3	

$\lambda T = \sqrt{A.fy/NcrT}$	$\sqrt{58.73x275/2728.77}$	0.769	
$N_{b.T.Rd}$= Area.χ.fy/ γ_{M1}	58.73x0.681x275/10/1 =	1100.539 kN	Curve c

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

C_1= fn(M_1, M_2, M_o, ψ,μ)	0.0, 0.0, 65.9, 0.854, 300.000	1.127	Uniform
C_{mLT}=0.95+0.05α_h	M_h= 0.05, M_s= 65.95, ψ = 0.854, α_s= 0.001	0.95	Table B.3
C_{mz}=Max(0.6+0.4ψ, 0.4)	M = 0, ψ = 1.000	1	Table B.3
C_{my}=0.95+0.05α_h	M_h= 0, M_s= 65.96, ψ = 1.000, α_s= 0.000	0.95	Table B.3

Lateral Buckling Check M.b.Rd

Le = 1.2L+2D	1.2 x 5.3 + 2 x 0.203 =	6.766 m	
M_{cr}= Fn(C_1,L_e,I_z,I_t,I_w,E)	1.127, 6.766, 1551, 22.15, 0.1429, 210000	147.402 kN.m	
λ_{LT}= $\sqrt{}$ W.f_y/M_{cr}	$\sqrt{}$ 497.4 x 275 / 147.402	0.963	
χ_{LT}= Fn(λ_{LT}, λ_{LT5950})	0.963, 0.922	0.722	Curve b
χ_{LT}.mod = Fn(χ_{LT},λ_{LT},k_c,f)	0.722, 0.963, 0.942, 0.973	0.742	6.3.2.3
$M_{b.Rd}$ = χ $W_{pl.y}$.$f_y$$\leq$ $M_{c.y.Rd}$	0.742 x 497.4 x 275 $\leq$ 136.785 =	101.552 kN.m	

Buckling Resistance

$U_{N.y}$ = N_{Ed}/(χ_y.N_{Rk}/γ_{M1})	0.426 / 1272.861	0.000	OK
$U_{N.z}$ = N_{Ed}/(χ_z.N_{Rk}/γ_{M1})	0.426 / 710.09	0.001	OK
$U_{M.y}$ = $M_{y.Ed}$/(χ_{LT}.$M_{y.Rk}$/γ_{M1})	66.039 / 101.552	0.650	OK
$U_{M.z}$ = $M_{z.Ed}$/($M_{z.Rk}$/γ_{M1})	0 / 63.498	0.000	OK
k_{yy}=C_{my}{1+(λ_y-0.2)$U_{N.y}$}		0.950	
k_{zz}=C_{mz}{1+1.4$U_{N.z}$}		1.001	
k_{yz}=0.6 k_{zz}		0.601	
k_{zy}= 1- {0.1λ_z/(C_{mLT}-0.25)}$U_{N.z}$		1.000	
U_{Ny}+k_{yy}.$U_{M.y}$+k_{yz}.$U_{M.z}$	0.000+0.950x0.650+0.601x0.000	0.618	OK
U_{Nz}+k_{zy}.$U_{M.y}$+k_{zz}.$U_{M.z}$	0.001+1.000x0.650+1.001x0.000	0.651	OK

Deflection Check - Load Case 3

Deflection Limits (Internal Beams)	$\delta \leq$ 5300/360 = 14.7 mm Live (Case 2)	8.33 mm	OK
	$\delta \leq$ 5300/250 = 21.2 mm D+L (Case 3)	14.03 mm	OK

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

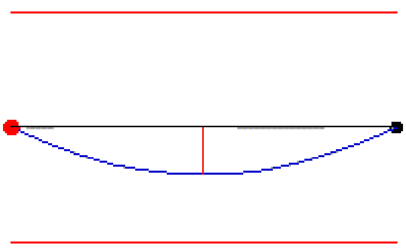
Axial with Moments (Member)

B6

Members 21-32 (N.34-N.36) @ 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 -000.180 (kN/m)
L1 UDLY -000.113 (kN/m)
D1 D 078.500 (kN/m^3)
Part 2
D1 UDLY -000.180 (kN/m)
L1 UDLY -000.113 (kN/m)
D1 D 078.500 (kN/m^3)
Part 3
D1 UDLY -000.180 (kN/m)
L1 UDLY -000.113 (kN/m)
D1 D 078.500 (kN/m^3)
Part 4
D1 UDLY -000.180 (kN/m)
L1 UDLY -000.113 (kN/m)
D1 D 078.500 (kN/m^3)
Part 5
D1 UDLY -000.180 (kN/m)
L1 UDLY -000.113 (kN/m)
D1 D 078.500 (kN/m^3)
D1 UDLY -003.740 (kN/m)
L1 UDLY -001.050 (kN/m)
Part 6
D1 UDLY -000.180 (kN/m)
L1 UDLY -000.113 (kN/m)
D1 D 078.500 (kN/m^3)
D1 UDLY -003.740 (kN/m)
L1 UDLY -001.050 (kN/m)
Part 7
D1 UDLY -000.180 (kN/m)
L1 UDLY -000.113 (kN/m)
D1 D 078.500 (kN/m^3)
D1 UDLY -003.740 (kN/m)
L1 UDLY -001.050 (kN/m)
Part 8
D1 UDLY -000.180 (kN/m)
L1 UDLY -000.113 (kN/m)
D1 D 078.500 (kN/m^3)
Part 9
D1 UDLY -000.180 (kN/m)
L1 UDLY -000.113 (kN/m)
D1 D 078.500 (kN/m^3)
Part 10
D1 UDLY -000.180 (kN/m)
L1 UDLY -000.113 (kN/m)
D1 D 078.500 (kN/m^3)
Part 11
D1 UDLY -000.180 (kN/m)
L1 UDLY -000.113 (kN/m)
D1 D 078.500 (kN/m^3)
Part 12
D1 UDLY -000.180 (kN/m)
L1 UDLY -000.113 (kN/m)
D1 D 078.500 (kN/m^3)

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

Mem ber No.	Node End1 End2	Axial Force (kN)	Torque Moment (kN.m)	Shear Force (kN)		Bending Moment (kN.m)		Maximum Moment (kN.m @ m)		Maximum Deflection (mm @ m)
				y-y	z-z	y-y	z-z	y-y	z-z	
	34	0.00C	0.00	39.65	0.00	0.00	0.00	55.55	0.00	12.00
	51	0.41C	0.00	-38.71	0.00	0.00	0.00	@ 2.658	@ 0.000	@ 2.645

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, 0.41, 55.54, 0	(Axial: Non-Slender)	Class 1
Auto Design Load Cases	1		

Local Capacity Check

$V_{y.Ed}/V_{pl.y.Rd}$	0.193 / 269.498 =	0.001	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	
$n = N_{Ed}/N_{pl.Rd}$	0.004 / 1615.075 =	0.000	OK
$W_{pl.N.y}$ = Fn($W_{pl.y}$, A_{vy}, n)	497.4, 16.974, 0	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	
$(M_{y.Ed}/M_{N.y.Rd})+(M_{z.Ed}/M_{N.z.Rd})$	$(55.544/136.785)^2+(0)^1=$	0.165	OK

Compression Resistance N.b.Rd

λy = √A.fy/Ncr	√58.73x275/3372.49	0.692	
$N_{b.y.Rd}$ = Area.χ.f_y/γ_{M1}	58.73x0.788x275/10/1 =	1272.861 kN	Curve b
λz = √A.fy/Ncrz	√58.73x275/1144.48	1.188	
$N_{b.z.Rd}$ = Area.χ.f_y/γ_{M1}	58.73x0.44x275/10/1 =	710.090 kN	Curve c
Let = Kt.Lx	1x5.3 =	5.3	
λT = √A.fy/NcrT	√58.73x275/2728.77	0.769	
$N_{b.T.Rd}$= Area.χ.fy/ γ_{M1}	58.73x0.681x275/10/1 =	1100.539 kN	Curve c

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

C_1= fn(M_1, M_2, M_o, ψ,μ)	0.0, 0.0, 55.5, 0.946, 300.000	1.127	Uniform
$C_{mLT}=0.95+0.05\alpha_h$	M_h= 0.04, M_s= 55.54, ψ = 0.946, α_s= 0.001	0.95	Table B.3
C_{mz}=Max(0.6+0.4ψ, 0.4)	M = 0, ψ = 1.000	1	Table B.3
$C_{my}=0.95+0.05\alpha_h$	M_h= 0, M_s= 55.55, ψ = 1.000, α_s= 0.000	0.95	Table B.3

Lateral Buckling Check M.b.Rd

Le = 1.2L+2D	1.2 x 5.3 + 2 x 0.203 =	6.766 m	
M_{cr}= Fn(C_1,L_e,I_z,I_t,I_w,E)	1.127, 6.766, 1551, 22.15, 0.1429, 210000	147.402 kN.m	
λ_{LT}= √ W.f_y/M_{cr}	√ 497.4 x 275 / 147.402	0.963	
χ_{LT}= Fn(λ_{LT}, λ_{LT5950})	0.963, 0.922	0.722	Curve b
χ_{LT}.mod = Fn(χ_{LT},λ_{LT},k_c,f)	0.722, 0.963, 0.942, 0.973	0.742	6.3.2.3
$M_{b.Rd} = \chi\ W_{pl.y}.f_y \leq M_{c.y.Rd}$	0.742 x 497.4 x 275 ≤ 136.785 =	101.552 kN.m	

Buckling Resistance

$U_{N.y} = N_{Ed}/(\chi_y.N_{Rk}/\gamma_{M1})$	0.408 / 1272.861	0.000	OK
$U_{N.z} = N_{Ed}/(\chi_z.N_{Rk}/\gamma_{M1})$	0.408 / 710.09	0.001	OK
$U_{M.y} = M_{y.Ed}/(\chi_{LT}.M_{y.Rk}/\gamma_{M1})$	55.544 / 101.552	0.547	OK
$U_{M.z} = M_{z.Ed}/(M_{z.Rk}/\gamma_{M1})$	0 / 63.498	0.000	OK
$k_{yy}=C_{my}\{1+(\lambda_y-0.2)U_{N.y}\}$		0.950	
$k_{zz}=C_{mz}\{1+1.4U_{N.z}\}$		1.001	
$k_{yz}=0.6\ k_{zz}$		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+0.950x0.547+0.600x0.000	0.520	OK
$U_{Nz}+k_{zy}.U_{M.y}+k_{zz}.U_{M.z}$	0.001+1.000x0.547+1.001x0.000	0.547	OK

Deflection Check - Load Case 3

Deflection Limits (Internal Beams)	δ ≤ 5300/360 = 14.7 mm Live (Case 2)	4.99 mm	OK
	δ ≤ 5300/250 = 21.2 mm D+L (Case 3)	12 mm	OK

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

AXIAL WITH MOMENTS (MEMBER)

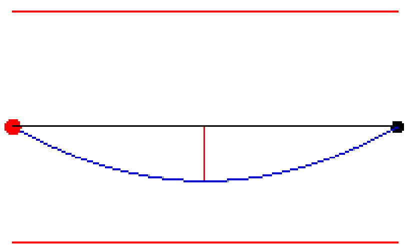
B7

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 -003.210            ( kN/m )
L1 UDLY -008.800            ( 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	0.241C	48.113	0.000	63.750	13.347
	2	0.241C	-48.113	0.000	@ 2.650	@ 2.650

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, 0.24, 63.75, 0	(Axial: Non-Slender)	Class 1
Auto Design Load Cases	1		

Local Capacity Check

$V_{y.Ed}/V_{pl.y.Rd}$	0.003 / 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	
$N_{pl.Rd} = A_g.f_y/\gamma_{M0}$	58.73 x 275/1 =	1615.075 kN	
$n = N_{Ed}/N_{pl.Rd}$	0.241 / 1615.075 =	0.000	OK
$W_{pl.N.y}$ = Fn($W_{pl.y}, A_{vy}, n$)	497.4, 16.974, 0	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	
$(M_{y.Ed}/M_{N.y.Rd})+(M_{z.Ed}/M_{N.z.Rd})$	$(63.747/136.785)^2+(0)^1=$	0.217	OK

Compression Resistance N.b.Rd

$\lambda y = \sqrt{A.fy/Ncr}$	$\sqrt{58.73x275/3372.49}$	0.692	
$N_{b.y.Rd} = Area.\chi.f_y/\gamma_{M1}$	58.73x0.788x275/10/1 =	1272.861 kN	Curve b
$\lambda z = \sqrt{A.fy/Ncrz}$	$\sqrt{58.73x275/1144.48}$	1.188	
$N_{b.z.Rd} = Area.\chi.f_y/\gamma_{M1}$	58.73x0.44x275/10/1 =	710.090 kN	Curve c
Let = Kt.Lx	1x5.3 =	5.3	
$\lambda T = \sqrt{A.fy/NcrT}$	$\sqrt{58.73x275/2728.77}$	0.769	
$N_{b.T.Rd} = Area.\chi.fy/\gamma_{M1}$	58.73x0.681x275/10/1 =	1100.539 kN	Curve c

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

C_1= fn(M_1, M_2, M_o, ψ, μ)	0.0, 0.0, 63.7, 0.958, 300.000	1.127	Uniform
$C_{mLT}=0.95+0.05\alpha_h$	M_h= 0.05, M_s= 63.75, ψ = 0.958, α_s= 0.001	0.95	Table B.3
C_{mz}=Max(0.6+0.4ψ, 0.4)	M = 0, ψ = 1.000	1	Table B.3
$C_{my}=0.95+0.05\alpha_h$	M_h= 0, M_s= 63.75, ψ = 1.000, α_s= 0.000	0.95	Table B.3

Lateral Buckling Check M.b.Rd

Le = 1.2L+2D	1.2 x 5.3 + 2 x 0.203 =	6.766 m	
M_{cr}= Fn($C_1, L_e, I_z, I_t, I_w, E$)	1.127, 6.766, 1551, 22.15, 0.1429, 210000	147.402 kN.m	
$\lambda_{LT} = \sqrt{W.f_y/M_{cr}}$	$\sqrt{497.4 \times 275 / 147.402}$	0.963	
χ_{LT}= Fn(λ_{LT}, λ_{LT5950})	0.963, 0.922	0.722	Curve b
χ_{LT}.mod = Fn($\chi_{LT}, \lambda_{LT}, k_c, f$)	0.722, 0.963, 0.942, 0.973	0.742	6.3.2.3
$M_{b.Rd} = \chi\ W_{pl.y}.f_y \leq M_{c.y.Rd}$	0.742 x 497.4 x 275 ≤ 136.785 =	101.552 kN.m	

Buckling Resistance

$U_{N.y} = N_{Ed}/(\chi_y.N_{Rk}/\gamma_{M1})$	0.241 / 1272.861	0.000	OK
$U_{N.z} = N_{Ed}/(\chi_z.N_{Rk}/\gamma_{M1})$	0.241 / 710.09	0.000	OK
$U_{M.y} = M_{y.Ed}/(\chi_{LT}.M_{y.Rk}/\gamma_{M1})$	63.747 / 101.552	0.628	OK
$U_{M.z} = M_{z.Ed}/(M_{z.Rk}/\gamma_{M1})$	0 / 63.498	0.000	OK
$k_{yy}=C_{my}\{1+(\lambda_y-0.2)U_{N.y}\}$		0.950	

$k_{zz}=C_{mz}\{1+1.4U_{N.z}\}$		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+0.950x0.628+0.600x0.000	0.597	OK
$U_{Nz}+k_{zy}.U_{M.y}+k_{zz}.U_{M.z}$	0.000+1.000x0.628+1.000x0.000	0.628	OK

Deflection Check - Load Case 2

Deflection Limits (Internal Beams)	δ ≤ 5300/360 = 14.7 mm Live (Case 2)	9.42 mm	OK
	δ ≤ 5300/250 = 21.2 mm D+L (Case 3)	13.35 mm	OK

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

AXIAL WITH MOMENTS (MEMBER)

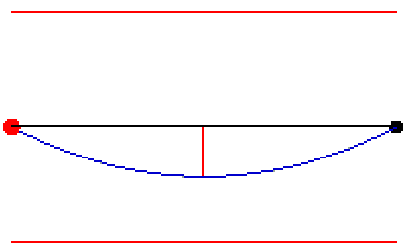
B8

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 -002.200          ( kN/m )
L1 UDLY -008.800          ( 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	0.222C	44.500	0.000	58.962	12.266
	2	0.222C	-44.500	0.000	@ 2.650	@ 2.650

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, 0.22, 58.96, 0	(Axial: Non-Slender)	Class 1
Auto Design Load Cases	1		

Local Capacity Check

$V_{y.Ed}/V_{pl.y.Rd}$	0.004 / 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	
$N_{pl.Rd} = A_g.f_y/\ \gamma_{M0}$	58.73 x 275/1 =	1615.075 kN	
$n = N_{Ed}/N_{pl.Rd}$	0.222 / 1615.075 =	0.000	OK
$W_{pl.N.y}$ = Fn($W_{pl.y}$, A_{vy}, n)	497.4, 16.974, 0	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	
$(M_{y.Ed}/M_{N.y.Rd})+(M_{z.Ed}/M_{N.z.Rd})$	$(58.957/136.785)^2+(0)^1=$	0.186	OK

Compression Resistance N.b.Rd

λy = √A.fy/Ncr	√58.73x275/3372.49	0.692	
$N_{b.y.Rd}$ = Area.χ.f_y/ γ_{M1}	58.73x0.788x275/10/1 =	1272.861 kN	Curve b
λz = √A.fy/Ncrz	√58.73x275/1144.48	1.188	
$N_{b.z.Rd}$ = Area.χ.f_y/ γ_{M1}	58.73x0.44x275/10/1 =	710.090 kN	Curve c
Let = Kt.Lx	1x5.3 =	5.3	
λT = √A.fy/NcrT	√58.73x275/2728.77	0.769	
$N_{b.T.Rd}$= Area.χ.fy/ γ_{M1}	58.73x0.681x275/10/1 =	1100.539 kN	Curve c

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

C_1= fn(M_1, M_2, M_o, ψ,μ)	0.0, 0.0, 58.9, 0.933, 300.000	1.127	Uniform
C_{mLT}=0.95+0.05α_h	M_h= 0.05, M_s= 58.96, ψ = 0.933, α_s= 0.001	0.95	Table B.3
C_{mz}=Max(0.6+0.4ψ, 0.4)	M = 0, ψ = 1.000	1	Table B.3

C_{my}=0.95+0.05α_h	M_h= 0, M_s= 58.96, ψ = 1.000, α_s= 0.000	0.95	Table B.3

Lateral Buckling Check M.b.Rd

Le = 1.2L+2D	1.2 x 5.3 + 2 x 0.203 =	6.766 m	
M_{cr}= Fn(C_1,L_e,I_z,I_t,I_w,E)	1.127, 6.766, 1551, 22.15, 0.1429, 210000	147.402 kN.m	
λ_{LT}= $\sqrt{}$ W.f_y/M_{cr}	$\sqrt{}$ 497.4 x 275 / 147.402	0.963	
χ_{LT}= Fn(λ_{LT}, λ_{LT5950})	0.963, 0.922	0.722	Curve b
χ_{LT}.mod = Fn(χ_{LT},λ_{LT},k_c,f)	0.722, 0.963, 0.942, 0.973	0.742	6.3.2.3
$M_{b.Rd}$ = χ $W_{pl.y}$.f_y≤ $M_{c.y.Rd}$	0.742 x 497.4 x 275 ≤ 136.785 =	101.552 kN.m	

Buckling Resistance

$U_{N.y}$ = N_{Ed}/(χ_y.N_{Rk}/γ_{M1})	0.222 / 1272.861	0.000	OK
$U_{N.z}$ = N_{Ed}/(χ_z.N_{Rk}/γ_{M1})	0.222 / 710.09	0.000	OK
$U_{M.y}$ = $M_{y.Ed}$/(χ_{LT}.$M_{y.Rk}$/γ_{M1})	58.957 / 101.552	0.581	OK
$U_{M.z}$ = $M_{z.Ed}$/($M_{z.Rk}$/γ_{M1})	0 / 63.498	0.000	OK
k_{yy}=C_{my}{1+(λ_y-0.2)$U_{N.y}$}		0.950	
k_{zz}=C_{mz}{1+1.4$U_{N.z}$}		1.000	
k_{yz}=0.6 k_{zz}		0.600	
k_{zy}= 1- {0.1λ_z/(C_{mLT}-0.25)}$U_{N.z}$		1.000	
U_{Ny}+k_{yy}.$U_{M.y}$+k_{yz}.$U_{M.z}$	0.000+0.950x0.581+0.600x0.000	0.552	OK
U_{Nz}+k_{zy}.$U_{M.y}$+k_{zz}.$U_{M.z}$	0.000+1.000x0.581+1.000x0.000	0.581	OK

Deflection Check - Load Case 2

Deflection Limits (Internal Beams)	δ ≤ 5300/360 = 14.7 mm Live (Case 2)	9.42 mm	OK
	δ ≤ 5300/250 = 21.2 mm D+L (Case 3)	12.27 mm	OK

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

Axial with Moments (Member)

B9

Member 12 (N.27-N.28) @ Level 1 in Load Case 1

Member Loading and Member Forces

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

D1 D      078.500            ( kN/m³)
D1 UDLY  -036.270            ( kN/m  )
L1 UDLY  -014.550            ( kN/m  )
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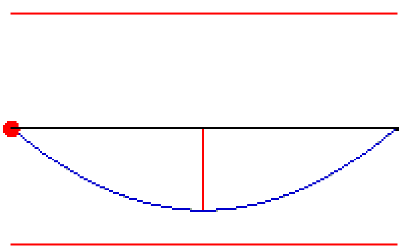


Member Forces in Load Case 1 and Maximum Deflection from Load Case 3										
Mem ber No.	Node End1 End2	Axial Force (kN)	Torque Moment (kN.m)	Shear Force (kN)		Bending Moment (kN.m)		Maximum Moment (kN.m @ m)		Maximum Deflection (mm @ m)
				y-y	z-z	y-y	z-z	y-y	z-z	
12	27	0.00C	0.00	182.13	0.00	-3.00	0.00	227.40	0.00	14.49
	28	0.00C	0.00	-182.14	0.00	-3.01	0.00	@ 2.530	@ 0.000	@ 2.530

Classification and Effective Area (EN 1993: 2006)

Section (88.95 kg/m)	254x254 UC 89 [S 275]		
Class = Fn(b/T,d/t,f_y,N,M_y,M_z)	7.41, 19.45, 265, 0, 227.39, 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 / 471.33 =	0	Low Shear
$M_{c.y.Rd}$ = f_y.$W_{pl.y}$/ γ_{M0}	265 x 1223.9/1	324.334 kN.m	
$M_{y.Ed}$/$M_{c.y.Rd}$	227.392 / 324.334 =	0.701	OK

Equivalent Uniform Moment Factor C1

C_1= fn(M_1, M_2, M_o, ψ,μ)	-2.8, -2.8, 230.2, 0.997, -81.463	1.202	Uniform

Lateral Buckling Check M.b.Rd

Le = 1.2L+2D	1.2 x 5.06 + 2 x 0.260 =	6.593 m	
M_{cr}= Fn(C_1,L_e,I_z,I_t,I_w,E)	1.202, 6.593, 4864, 102.3, 0.7166, 210000	625.963 kN.m	
λ_{LT}= √ W.f_y/M_{cr}	√ 1223.9 x 265 / 625.963	0.720	
χ_{LT}= Fn(λ_{LT}, λ_{LT5950})	0.720, 0.712	0.860	Curve b
χ_{LT}.mod = Fn(χ_{LT},λ_{LT},k_c,f)	0.860, 0.720, 0.912, 0.957	0.899	6.3.2.3
$M_{b.Rd}$ = χ $W_{pl.y}$.f_y≤ $M_{c.y.Rd}$	0.899 x 1224 x 265 ≤ 324.334 =	291.474 kN.m	
$M_{y.Ed}$/$M_{b.Rd}$	227.392 / 291.474	0.780	OK

Deflection Check - Load Case 3

Deflection Limits (Internal Beams)	δ ≤ 5060/360 = 14.1 mm Live (Case 2)	4.08 mm	OK
	δ ≤ 5060/250 = 20.2 mm D+L (Case 3)	14.49 mm	OK

Provide Section (88.95 kg/m) 254x254 UC 89 [S 275]

AXIAL WITH MOMENTS (MEMBER)

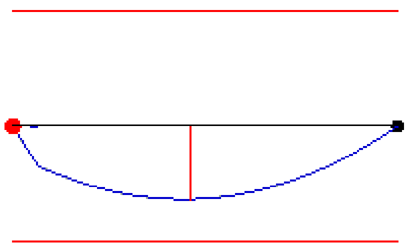
B10

Members 1-2 (N.11-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 UDLY -011.440            ( kN/m )
L1 UDLY -006.600            ( kN/m )
D1 D     078.500            ( kN/m³)
Part 2
D1 UDLY -011.440            ( kN/m )
L1 UDLY -006.600            ( kN/m )
D1 D     078.500            ( kN/m³)
```



Member Forces in Load Case 1 and Maximum Deflection from Load Case 3										
Mem ber No.	Node End1 End2	Axial Force (kN)	Torque Moment (kN.m)	Shear Force (kN)		Bending Moment (kN.m)		Maximum Moment (kN.m @ m)		Maximum Deflection (mm @ m)
				y-y	z-z	y-y	z-z	y-y	z-z	
	11	1.26T	0.00	181.20	0.02	0.00	0.00	113.97	0.01	19.11
	13	0.32T	0.00	-77.21	0.00	0.00	0.00	@ 2.330	@ 0.336	@ 2.578

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, 113.96, 0.01	(Axial: Non-Slender)	Class 1
Auto Design Load Cases	1		

Local Capacity Check

$V_{y.Ed}$/$V_{pl.y.Rd}$	2.575 / 351.748 =	0.007	Low Shear
$M_{c.y.Rd}$ = f_y.$W_{pl.y}$/ γ_{M0}	275 x 656.1/1	180.428 kN.m	
$N_{pl.Rd}$ = Ag.fy/γ_{M0}	76.37x275/1	2100.175 kN	
n = N_{Ed}/$N_{pl.Rd}$	-1.265 / 2100.175 =	0.001	OK
$W_{pl.N.y}$ = Fn($W_{pl.y}$, A_{vy}, n)	656.1, 22.154, 0.001	656.1 cm³	
$M_{N.y.Rd}$ = $W_{pl.N.y}$.f_y/ γ_{M0}	656.1 x 275/1	180.428 kN.m	
($M_{y.Ed}$/$M_{N.y.Rd}$)+($M_{z.Ed}$/$M_{N.z.Rd}$)	$(113.843/180.428)^2+(0)^1$=	0.398	OK

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

C_1= fn(M_1, M_2, M_o, ψ,μ)	0.2, 0.1, 112.6, 0.425, 300.000	1.127	Uniform
C_{mLT}=0.95+0.05α_h	M_h= 0.18, M_s= 112.78, ψ = 0.425, α_s= 0.002	0.95	Table B.3
C_{mz}=Max(0.6+0.4ψ, 0.4)	M = 0, ψ = 1.000	1	Table B.3
C_{my}=0.95+0.05α_h	M_h= 0, M_s= 112.78, ψ = 1.000, α_s= 0.000	0.95	Table B.3

Lateral Buckling Check M.b.Rd

Le = 1.2L+2D	1.2 x 5.3 + 2 x 0.210 =	6.779 m

M_{cr}= Fn(C_1,L_e,I_z,I_t,I_w,E)	1.127, 6.779, 2068, 47.23, 0.1969, 210000	235.996 kN.m	
$\lambda_{LT}= \sqrt{W.f_y/M_{cr}}$	$\sqrt{656.1 \times 275 / 235.996}$	0.874	
χ_{LT}= Fn(λ_{LT}, λ_{LT5950})	0.874, 0.837	0.775	Curve b
χ_{LT}.mod = Fn(χ_{LT},λ_{LT},k_c,f)	0.775, 0.874, 0.942, 0.971	0.798	6.3.2.3
$M_{b.Rd} = \chi\ W_{pl.y}.f_y \leq M_{c.y.Rd}$	0.798 x 656.1 x 275 ≤ 180.428 =	143.965 kN.m	

Buckling Resistance

$U_{N.y} = N_{Ed}/(\chi_y.N_{Rk}/\gamma_{M1})$	0 / 1667.819	0.000	OK
$U_{N.z} = N_{Ed}/(\chi_z.N_{Rk}/\gamma_{M1})$	0 / 937.412	0.000	OK
$U_{M.y} = M_{y.Ed}/(\chi_{LT}.M_{y.Rk}/\gamma_{M1})$	113.843 / 143.965	0.791	OK
$U_{M.z} = M_{z.Ed}/(M_{z.Rk}/\gamma_{M1})$	0 / 83.958	0.000	OK
$k_{yy}=C_{my}\{1+(\lambda_y-0.2)U_{N.y}\}$		0.950	
$k_{zz}=C_{mz}\{1+1.4U_{N.z}\}$		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+0.950x0.791+0.600x0.000	0.751	OK
$U_{Nz}+k_{zy}.U_{M.y}+k_{zz}.U_{M.z}$	0.000+1.000x0.791+1.000x0.000	0.791	OK

Deflection Check - Load Case 3

Deflection Limits (Internal Beams)	δ ≤ 5300/360 = 14.7 mm Live (Case 2)	5.27 mm	OK
	δ ≤ 5300/250 = 21.2 mm D+L (Case 3)	19.11 mm	OK

Provide Section (59.95 kg/m) 203x203 UC 60 [S 275]

Axial with Moments (Member)

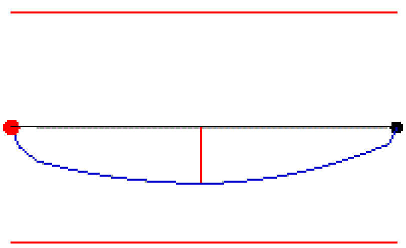
B11

Members 3-7 (N.3-N.4) @ 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 -005.720          ( kN/m )
L1 UDLY -003.300          ( kN/m )
D1 D     078.500          ( kN/m³)
Part 2
D1 UDLY -005.720          ( kN/m )
L1 UDLY -003.300          ( kN/m )
D1 D     078.500          ( kN/m³)
Part 3
D1 UDLY -005.720          ( kN/m )
L1 UDLY -003.300          ( kN/m )
D1 D     078.500          ( kN/m³)
Part 4
D1 UDLY -005.720          ( kN/m )
L1 UDLY -003.300          ( kN/m )
D1 D     078.500          ( kN/m³)
Part 5
D1 UDLY -005.720          ( kN/m )
L1 UDLY -003.300          ( kN/m )
D1 D     078.500          ( kN/m³)
```



Member Forces in Load Case 1 and Maximum Deflection from Load Case 3											
Mem ber No.	Node End1 End2	Axial Force (kN)	Torque Moment (kN.m)	Shear Force (kN)		Bending Moment (kN.m)		Maximum Moment (kN.m @ m)		Maximum Deflection (mm @ m)
				y-y	z-z	y-y	z-z	y-y	z-z	
	3	0.70T	0.00	277.80	-0.30	0.00	0.00	86.79	-0.13	15.17
	9	0.01T	0.00	-232.87	0.00	0.00	0.00	@ 2.600	@ 0.348	@ 2.626

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, 86.79, 0.13	(Axial: Non-Slender)	Class 1
Auto Design Load Cases	1		

Local Capacity Check

$V_{y.Ed}/V_{pl.y.Rd}$	0.027 / 351.748 =	0	Low Shear
$M_{c.y.Rd} = f_y.W_{pl.y}/\gamma_{M0}$	275 x 656.1/1	180.428 kN.m	
$V_{z.Ed}/V_{pl.z.Rd}$	0.025 / 927.974 =	0	Low Shear
$M_{c.z.Rd} = f_y.W_{pl.z}/\gamma_{M0}$	275 x 305.3/1	83.958 kN.m	
$N_{pl.Rd} = Ag.fy/\gamma_{M0}$	76.37x275/1	2100.175 kN	
$n = N_{Ed}/N_{pl.Rd}$	-2.137 / 2100.175 =	0.001	OK
$W_{pl.N.y} = Fn(W_{pl.y}, A_{vy}, n)$	656.1, 22.154, 0.001	656.1 cm³	
$M_{N.y.Rd} = W_{pl.N.y}.f_y/\gamma_{M0}$	656.1 x 275/1	180.428 kN.m	
$W_{pl.N.z} = Fn(W_{pl.z}, A_{vz}, n)$	305.3, 58.447, 0.001	305.3 cm³	
$M_{N.z.Rd} = W_{pl.N.z}.f_y/\gamma_{M0}$	305.3 x 275/1	83.958 kN.m	
$(M_{y.Ed}/M_{N.y.Rd})+(M_{z.Ed}/M_{N.z.Rd})$	$(86.792/180.428)^2+(0.065/83.958)^1=$	0.232	OK

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

C_1= fn(M_1, M_2, M_o, ψ,μ)	0.3, 0.2, 86.5, 0.855, 300.000	1.127	Uniform
$C_{mLT}=0.95+0.05\alpha_h$	M_h= 0.28, M_s= 86.76, ψ = 0.855, α_s= 0.003	0.95	Table B.3
C_{mz}=Max(0.6+0.4ψ, 0.4)	M = 0, ψ = 0.000	0.6	Table B.3
$C_{my}=0.95+0.05\alpha_h$	M_h= 0, M_s= 86.77, ψ = 1.000, α_s= 0.000	0.95	Table B.3

Lateral Buckling Check M.b.Rd

Le = 1.2L+2D	1.2 x 5.3 + 2 x 0.210 =	6.779 m	
M_{cr}= Fn(C_1,L_e,I_z,I_t,I_w,E)	1.127, 6.779, 2068, 47.23, 0.1969, 210000	235.996 kN.m	
$\lambda_{LT}= \sqrt{W.f_y/M_{cr}}$	$\sqrt{656.1 \times 275 / 235.996}$	0.874	
χ_{LT}= Fn(λ_{LT}, λ_{LT5950})	0.874, 0.837	0.775	Curve b
χ_{LT}.mod = Fn($\chi_{LT},\lambda_{LT},k_c,f$)	0.775, 0.874, 0.942, 0.971	0.798	6.3.2.3
$M_{b.Rd} = \chi\ W_{pl.y}.f_y \le M_{c.y.Rd}$	0.798 x 656.1 x 275 ≤ 180.428 =	143.965 kN.m	

Buckling Resistance

$U_{N.y} = N_{Ed}/(\chi_y.N_{Rk}/\gamma_{M1})$	0 / 1667.819	0.000	OK
$U_{N.z} = N_{Ed}/(\chi_z.N_{Rk}/\gamma_{M1})$	0 / 937.412	0.000	OK
$U_{M.y} = M_{y.Ed}/(\chi_{LT}.M_{y.Rk}/\gamma_{M1})$	86.792 / 143.965	0.603	OK
$U_{M.z} = M_{z.Ed}/(M_{z.Rk}/\gamma_{M1})$	0.065 / 83.958	0.001	OK
$k_{yy}=C_{my}\{1+(\lambda_y-0.2)U_{N.y}\}$		0.950	
$k_{zz}=C_{mz}\{1+1.4U_{N.z}\}$		0.600	
$k_{yz}=0.6\ k_{zz}$		0.360	
$k_{zy}= 1- \{0.1\lambda_z/(C_{mLT}-0.25)\}U_{N.z}$		1.000	
$U_{Ny}+k_{yy}.U_{M.y}+k_{yz}.U_{M.z}$	0.000+0.950x0.603+0.360x0.001	0.573	OK
$U_{Nz}+k_{zy}.U_{M.y}+k_{zz}.U_{M.z}$	0.000+1.000x0.603+0.600x0.001	0.603	OK

Deflection Check - Load Case 3

Deflection Limits (Internal Beams)	δ ≤ 5300/360 = 14.7 mm Live (Case 2)	4.26 mm	OK
	δ ≤ 5300/250 = 21.2 mm D+L (Case 3)	15.17 mm	OK

Provide Section (59.95 kg/m) **203x203 UC 60 [S 275]**

AXIAL WITH MOMENTS (MEMBER)

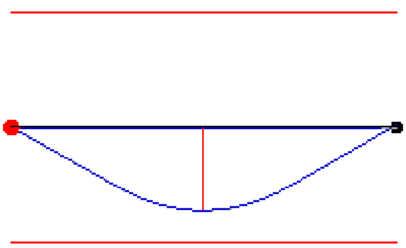
B12

Members 18-20 (N.5-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 D      078.500          ( kN/m³)
Part 2
D1 D      078.500          ( kN/m³)
D1 UDLY -043.000           ( kN/m )
Part 3
D1 D      078.500          ( kN/m³)
```



Member Forces in Load Case 1 and Maximum Deflection from Load Case 3										
Mem ber No.	Node End1 End2	Axial Force (kN)	Torque Moment (kN.m)	Shear Force (kN)		Bending Moment (kN.m)		Maximum Moment (kN.m @ m)		Maximum Deflection (mm @ m)
				y-y	z-z	y-y	z-z	y-y	z-z	
	5	0.44T	0.01	58.77	0.29	-0.43	-0.01	98.34	0.34	14.32
	12	0.44T	-0.01	-60.07	-0.30	-3.29	-0.02	@ 2.180	@ 1.200	@ 2.200

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, 98.33, 0.34	(Axial: Non-Slender)	Class 1
Auto Design Load Cases	1		

Local Capacity Check

$V_{y.Ed}/V_{pl.y.Rd}$	0.657 / 269.498 =	0.002	Low Shear
$M_{c.y.Rd} = f_y.W_{pl.y}/\ \gamma_{M0}$	275 x 497.4/1	136.785 kN.m	
$V_{z.Ed}/V_{pl.z.Rd}$	0.002 / 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	1615.075 kN	
$n = N_{Ed}/N_{pl.Rd}$	-0.442 / 1615.075 =	0.000	OK
$W_{pl.N.y} = Fn(W_{pl.y}, A_{vy}, n)$	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	
$W_{pl.N.z} = Fn(W_{pl.z}, A_{vz}, n)$	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	
$(M_{y.Ed}/M_{N.y.Rd})+(M_{z.Ed}/M_{N.z.Rd})$	$(98.329/136.785)^2+(0.341/63.498)^1=$	0.522	OK

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

C_1= fn(M_1, M_2, M_o, ψ, μ)	-0.4, -3.2, 100.1, 0.113, -31.009	1.167	Uniform
$C_{mLT}=0.95+0.05\alpha_h$	M_h= -3.23, M_s= 98.33, ψ = 0.113, α_s= -0.033	0.948	Table B.3
$C_{mz}=0.95+0.05\alpha_h$	M_h= -0.02, M_s= 0.34, ψ = 0.562, α_s= -0.047	0.948	Table B.3
$C_{my}=0.95+0.05\alpha_h$	M_h= -3.29, M_s= 98.33, ψ = 0.129, α_s= -0.033	0.948	Table B.3

Lateral Buckling Check M.b.Rd

Le = 1.2L+2D	1.2 x 4.4 + 2 x 0.203 =	5.686 m	
M_{cr}= Fn($C_1, L_e, I_z, I_t, I_w, E$)	1.167, 5.686, 1551, 22.15, 0.1429, 210000	191.344 kN.m	
$\lambda_{LT}= \sqrt{W.f_y/M_{cr}}$	√ 497.4 x 275 / 191.344	0.845	
χ_{LT}= Fn(λ_{LT}, λ_{LT5950})	0.845, 0.823	0.792	Curve b
χ_{LT}.mod = Fn($\chi_{LT}, \lambda_{LT}, k_c, f$)	0.792, 0.845, 0.926, 0.963	0.822	6.3.2.3
$M_{b.Rd} = \chi\ W_{pl.y}.f_y \leq M_{c.y.Rd}$	0.822 x 497.4 x 275 ≤ 136.785 =	112.441 kN.m	

Buckling Resistance

$U_{N.y} = N_{Ed}/(\chi_y.N_{Rk}/\gamma_{M1})$	0 / 1372.147	0.000	OK
$U_{N.z} = N_{Ed}/(\chi_z.N_{Rk}/\gamma_{M1})$	0 / 885.187	0.000	OK
$U_{M.y} = M_{y.Ed}/(\chi_{LT}.M_{y.Rk}/\gamma_{M1})$	98.329 / 112.441	0.874	OK
$U_{M.z} = M_{z.Ed}/(M_{z.Rk}/\gamma_{M1})$	0.341 / 63.498	0.005	OK
$k_{yy}=C_{my}\{1+(\lambda_y-0.2)U_{N.y}\}$		0.948	
$k_{zz}=C_{mz}\{1+(2\lambda_z-0.6)U_{N.z}\}$		0.948	
$k_{yz}=0.6\ k_{zz}$		0.569	
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+0.948x0.874+0.569x0.005	0.832	OK

$U_{Nz}+k_{zy}.U_{M.y}+k_{zz}.U_{M.z}$	0.000+1.000x0.874+0.948x0.005	0.880	OK

Deflection Check - Load Case 3

Deflection Limits (Internal Beams)	$\delta \leq$ 4400/250 = 17.6 mm D+L (Case 3)	14.32 mm	OK

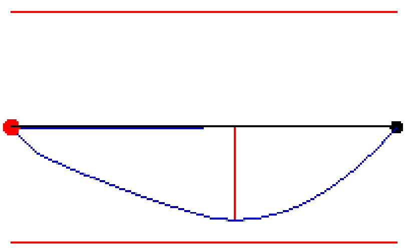
Provide Section (46.1 kg/m) **203x203 UC 46 [S 275]**

AXIAL WITH MOMENTS (MEMBER)

B13

Members 8-11 (N.16-N.17) @ 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 -007.295           ( kN/m )
L1 UDLY -003.825           ( kN/m )
D1 D     078.500           ( kN/m³)
D1 UDLY -049.550           ( kN/m )
L1 UDLY -014.550           ( kN/m )
Part 2
D1 UDLY -007.295           ( kN/m )
L1 UDLY -003.825           ( kN/m )
D1 D     078.500           ( kN/m³)
D1 UDLY -049.550           ( kN/m )
L1 UDLY -014.550           ( kN/m )
Part 3
D1 UDLY -007.295           ( kN/m )
L1 UDLY -003.825           ( kN/m )
D1 D     078.500           ( kN/m³)
Part 4
D1 UDLY -007.295           ( kN/m )
L1 UDLY -003.825           ( kN/m )
D1 D     078.500           ( kN/m³)
D1 UDLY -049.550           ( kN/m )
L1 UDLY -014.550           ( kN/m )
```

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

Mem ber No.	Node End1 End2	Axial Force (kN)	Torque Moment (kN.m)	Shear Force (kN) y-y	Shear Force (kN) z-z	Bending Moment (kN.m) y-y	Bending Moment (kN.m) z-z	Maximum Moment (kN.m @ m) y-y	Maximum Moment (kN.m @ m) z-z	Maximum Deflection (mm @ m)
	16	2.00T	0.00	222.74	0.52	0.00	0.00	260.60	0.18	17.70
	20	0.67T	0.00	-234.50	0.00	0.00	0.00	@ 3.082	@ 0.347	@ 2.750

Classification and Effective Area (EN 1993: 2006)

Section (88.95 kg/m)	254x254 UC 89 [S 275]		
Class = Fn(b/T,d/t,f_y,N,M_y,M_z)	7.41, 19.45, 265, 0, 260.55, 0.18	(Axial: Non-Slender)	Class 1
Auto Design Load Cases	1		

Local Capacity Check

$V_{y.Ed}/V_{pl.y.Rd}$	2.097 / 471.33 =	0.004	Low Shear
$M_{c.y.Rd} = f_y.W_{pl.y}/\gamma_{M0}$	265 x 1223.9/1	324.334 kN.m	
$N_{pl.Rd} = Ag.fy/\gamma_{M0}$	113.31x265/1	3002.715 kN	
$n = N_{Ed}/N_{pl.Rd}$	-1.996 / 3002.715 =	0.001	OK
$W_{pl.N.y}$ = Fn($W_{pl.y}$, A_{vy}, n)	1223.9, 30.806, 0.001	1223.9 cm^3	
$M_{N.y.Rd} = W_{pl.N.y}.f_y/\gamma_{M0}$	1223.9 x 265/1	324.334 kN.m	
$(M_{y.Ed}/M_{N.y.Rd})+(M_{z.Ed}/M_{N.z.Rd})$	$(260.578/324.334)^2+(0)^1=$	0.645	OK

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

C_1= fn(M_1, M_2, M_o, ψ,μ)	0.2, 0.2, 248.6, 0.944, 300.000	1.127	Uniform
$C_{mLT}=0.95+0.05\alpha_h$	M_h= 0.23, M_s= 248.83, ψ = 0.944, α_s= 0.001	0.95	Table B.3
$C_{mz}=0.95+0.05\alpha_h$	M_h= 0, M_s= 0.01, ψ = 0.000, α_s= 0.111	0.956	Table B.3

C_{my}=0.95+0.05α_h	M_h= 0, M_s= 248.83, ψ = 1.000, α_s= 0.000	0.95	Table B.3

Lateral Buckling Check M.b.Rd

Le = 1.2L+2D	1.2 x 5.3 + 2 x 0.260 =	6.881 m	
M_{cr}= Fn(C_1,L_e,I_z,I_t,I_w,E)	1.127, 6.881, 4864, 102.3, 0.7166, 210000	555.377 kN.m	
λ_{LT}= √ W.f_y/M_{cr}	√ 1223.9 x 265 / 555.377	0.764	
χ_{LT}= Fn(λ_{LT}, λ_{LT5950})	0.764, 0.732	0.837	Curve b
χ_{LT}.mod = Fn(χ_{LT},λ_{LT},k_c,f)	0.837, 0.764, 0.942, 0.971	0.861	6.3.2.3
$M_{b.Rd}$ = χ $W_{pl.y}$.f_y≤ $M_{c.y.Rd}$	0.861 x 1224 x 265 ≤ 324.334 =	279.397 kN.m	

Buckling Resistance

$U_{N.y}$ = N_{Ed}/(χ_y.N_{Rk}/γ_{M1})	0 / 2608.696	0.000	OK
$U_{N.z}$ = N_{Ed}/(χ_z.N_{Rk}/γ_{M1})	0 / 1773.616	0.000	OK
$U_{M.y}$ = $M_{y.Ed}$/(χ_{LT}.$M_{y.Rk}$/γ_{M1})	260.578 / 279.397	0.933	OK
$U_{M.z}$ = $M_{z.Ed}$/($M_{z.Rk}$/γ_{M1})	0 / 152.455	0.000	OK
k_{yy}=C_{my}{1+(λ_y-0.2)$U_{N.y}$}		0.950	
k_{zz}=C_{mz}{1+(2λ_z-0.6)$U_{N.z}$}		0.956	
k_{yz}=0.6 k_{zz}		0.573	
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+0.950x0.933+0.573x0.000	0.886	OK
U_{Nz}+k_{zy}.$U_{M.y}$+k_{zz}.$U_{M.z}$	0.000+1.000x0.933+0.956x0.000	0.933	OK

Deflection Check - Load Case 3

Deflection Limits (Internal Beams)	δ ≤ 5300/360 = 14.7 mm Live (Case 2)	3.87 mm	OK
	δ ≤ 5300/250 = 21.2 mm D+L (Case 3)	17.7 mm	OK

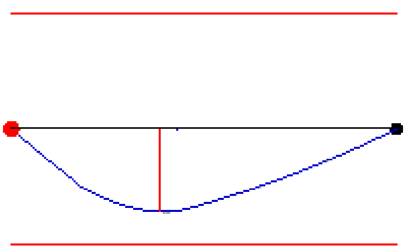
Provide Section (88.95 kg/m) **254x254 UC 89 [S 275]**

AXIAL WITH MOMENTS (MEMBER)

B14

Members 13-15 (N.21-N.22) @ 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 -002.700             ( kN/m )
L1 UDLY -000.900             ( kN/m )
D1 TRY  -001.063 0.000 0.450 (kN,m,m)
L1 TRY  -000.354 0.000 0.450 (kN,m,m)
D1 D     078.500             ( kN/m³)
Part 2
D1 UDLY -002.700             ( kN/m )
L1 UDLY -000.900             ( kN/m )
D1 TRY  -001.063 0.000 0.450 (kN,m,m)
L1 TRY  -000.354 0.000 0.450 (kN,m,m)
D1 D     078.500             ( kN/m³)
D1 UDLY -033.420             ( kN/m )
L1 UDLY -002.640             ( kN/m )
Part 3
D1 UDLY -002.700             ( kN/m )
L1 UDLY -000.900             ( kN/m )
D1 TRY  -001.063 0.000 0.450 (kN,m,m)
L1 TRY  -000.354 0.000 0.450 (kN,m,m)
D1 D     078.500             ( kN/m³)
```

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

Mem ber No.	Node End1 End2	Axial Force (kN)	Torque Moment (kN.m)	Shear Force (kN) y-y	Shear Force (kN) z-z	Bending Moment (kN.m) y-y	Bending Moment (kN.m) z-z	Maximum Moment (kN.m @ m) y-y	Maximum Moment (kN.m @ m) z-z	Maximum Deflection (mm @ m)
	21	0.94T	0.00	97.11	-0.10	0.00	0.00	129.15	-0.10	19.63
	25	0.07T	0.00	-55.65	0.00	0.00	0.00	@ 2.146	@ 0.900	@ 2.484

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, 129.14, 0.1	(Axial: Non-Slender)	Class 1
Auto Design Load Cases	1		

Local Capacity Check

$V_{y.Ed}/V_{pl.y.Rd}$	5.779 / 351.748 =	0.016	Low Shear
$M_{c.y.Rd} = f_y.W_{pl.y}/\ \gamma_{M0}$	275 x 656.1/1	180.428 kN.m	
$V_{z.Ed}/V_{pl.z.Rd}$	0.073 / 927.974 =	0	Low Shear
$M_{c.z.Rd} = f_y.W_{pl.z}/\ \gamma_{M0}$	275 x 305.3/1	83.958 kN.m	
$N_{pl.Rd} = Ag.fy/\gamma_{M0}$	76.37x275/1	2100.175 kN	
$n = N_{Ed}/N_{pl.Rd}$	-0.945 / 2100.175 =	0.000	OK
$W_{pl.N.y} = Fn(W_{pl.y}, A_{vy}, n)$	656.1, 22.154, 0	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	
$W_{pl.N.z} = Fn(W_{pl.z}, A_{vz}, n)$	305.3, 58.447, 0	305.3 cm^3	
$M_{N.z.Rd} = W_{pl.N.z}.f_y/\ \gamma_{M0}$	305.3 x 275/1	83.958 kN.m	
$(M_{y.Ed}/M_{N.y.Rd})+(M_{z.Ed}/M_{N.z.Rd})$	$(128.855/180.428)^2+(0.012/83.958)^1=$	0.51	OK

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

C_1= fn(M_1, M_2, M_o, ψ, μ)	0.1, 0.1, 116.8, 0.613, 300.000	1.127	Uniform
$C_{mLT}=0.95+0.05\alpha_h$	M_h= 0.09, M_s= 116.86, ψ = 0.613, α_s= 0.001	0.95	Table B.3
C_{mz}=Max(0.6+0.4ψ, 0.4)	M = 0, ψ = 0.000	0.6	Table B.3
$C_{my}=0.95+0.05\alpha_h$	M_h= 0, M_s= 116.86, ψ = 1.000, α_s= 0.000	0.95	Table B.3

Lateral Buckling Check M.b.Rd

Le = 1.2L+2D	1.2 x 5.3 + 2 x 0.210 =	6.779 m	
M_{cr}= Fn($C_1, L_e, I_z, I_t, I_w, E$)	1.127, 6.779, 2068, 47.23, 0.1969, 210000	235.996 kN.m	
$\lambda_{LT} = \sqrt{W.f_y/M_{cr}}$	√ 656.1 x 275 / 235.996	0.874	
χ_{LT}= Fn(λ_{LT}, λ_{LT5950})	0.874, 0.837	0.775	Curve b
χ_{LT}.mod = Fn($\chi_{LT}, \lambda_{LT}, k_c, f$)	0.775, 0.874, 0.942, 0.971	0.798	6.3.2.3
$M_{b.Rd} = \chi\ W_{pl.y}.f_y \leq M_{c.y.Rd}$	0.798 x 656.1 x 275 ≤ 180.428 =	143.965 kN.m	

Buckling Resistance

$U_{N.y} = N_{Ed}/(\chi_y.N_{Rk}/\gamma_{M1})$	0 / 1667.819	0.000	OK
$U_{N.z} = N_{Ed}/(\chi_z.N_{Rk}/\gamma_{M1})$	0 / 937.412	0.000	OK
$U_{M.y} = M_{y.Ed}/(\chi_{LT}.M_{y.Rk}/\gamma_{M1})$	128.855 / 143.965	0.895	OK
$U_{M.z} = M_{z.Ed}/(M_{z.Rk}/\gamma_{M1})$	0.012 / 83.958	0.000	OK
$k_{yy}=C_{my}\{1+(\lambda_y-0.2)U_{N.y}\}$		0.950	
$k_{zz}=C_{mz}\{1+1.4U_{N.z}\}$		0.600	
$k_{yz}=0.6\ k_{zz}$		0.360	
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+0.950x0.895+0.360x0.000	0.850	OK
$U_{Nz}+k_{zy}.U_{M.y}+k_{zz}.U_{M.z}$	0.000+1.000x0.895+0.600x0.000	0.895	OK

Deflection Check - Load Case 3

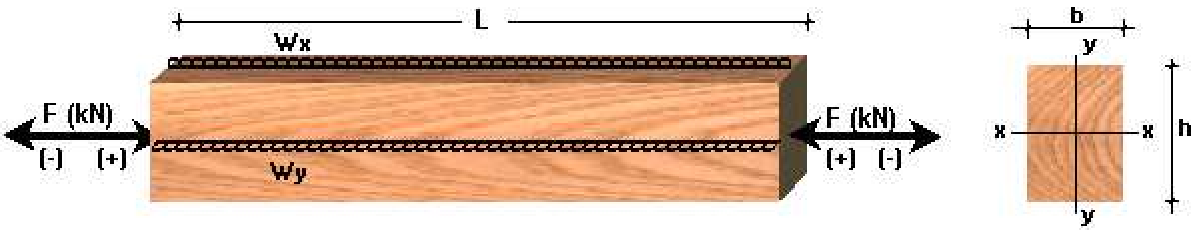
Deflection Limits (Internal Beams)	δ ≤ 5300/360 = 14.7 mm Live (Case 2)	2.27 mm	OK
	δ ≤ 5300/250 = 21.2 mm D+L (Case 3)	19.63 mm	OK

Provide Section (59.95 kg/m) 203x203 UC 60 [S 275]

MASTERKEY : TIMBER DESIGN

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

Third Floor's Joists



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 225 Regularized Section in Strength Class C24
Section Properties (cm²,cm³,cm)	Area 105.8, $W_{el.y}$ 396.6, $W_{el.z}$ 82.8, i_y 6.5, i_z 1.36
Specification	1 : Internal use in continuously heated building
	Long Term loading
Integrated Design	Critical Case 1
Member Details	N_{Ed} = 0.0 kN, L_y = 4.1 m, L_z = 4.1 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 1.26 x 1.00 x 24.00/1.3	16.30 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}	Instantaneous Deflection	11000 N/mm²	Deflection

Axial Load with Moments Check

Critical Design Location	X = 1.610		
$\sigma_{m.y.d} = M_y/W_{el.y}$	4.889 / 396.56 ≤ 12.92	12.33 N/mm²	OK
$\sigma_{m.z.d} = M_z/W_{el.z}$	0.022 / 82.84 ≤ 16.30	0.27 N/mm²	OK
$U_t = \sigma_{c.0.d}/f_{t.0.d}$	0.000/7.538	0.000	OK
$U_{m.y} = \sigma_{m.y.d}/f_{m.y.d}$	12.330/12.923	0.954	OK
$U_{m.z} = \sigma_{m.z.d}/f_{m.z.d}$	0.270/16.299	0.017	OK
$U_t+U_{m.y}+k_m.U_{m.z}$	0.000+0.954+0.7x0.017	0.966	OK
$U_t+k_m.U_{m.y}+U_{m.z}$	0.000+0.7x0.954+0.017	0.684	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 √(5.155²+0.025²) / 106 / 0.67 ≤ 2.15	1.09 N/mm²	OK
$\sigma_{c\perp ax} = V_{y.Ed}$ / $(b.l_y)$	5.155 / (47 x 75) ≤ 2.02	1.46 N/mm²	OK
$\sigma_{c\perp ay} = V_{z.Ed}$ / $(h.l_z)$	0.025 / (225 x 75) ≤ 2.02	0.00 N/mm²	OK

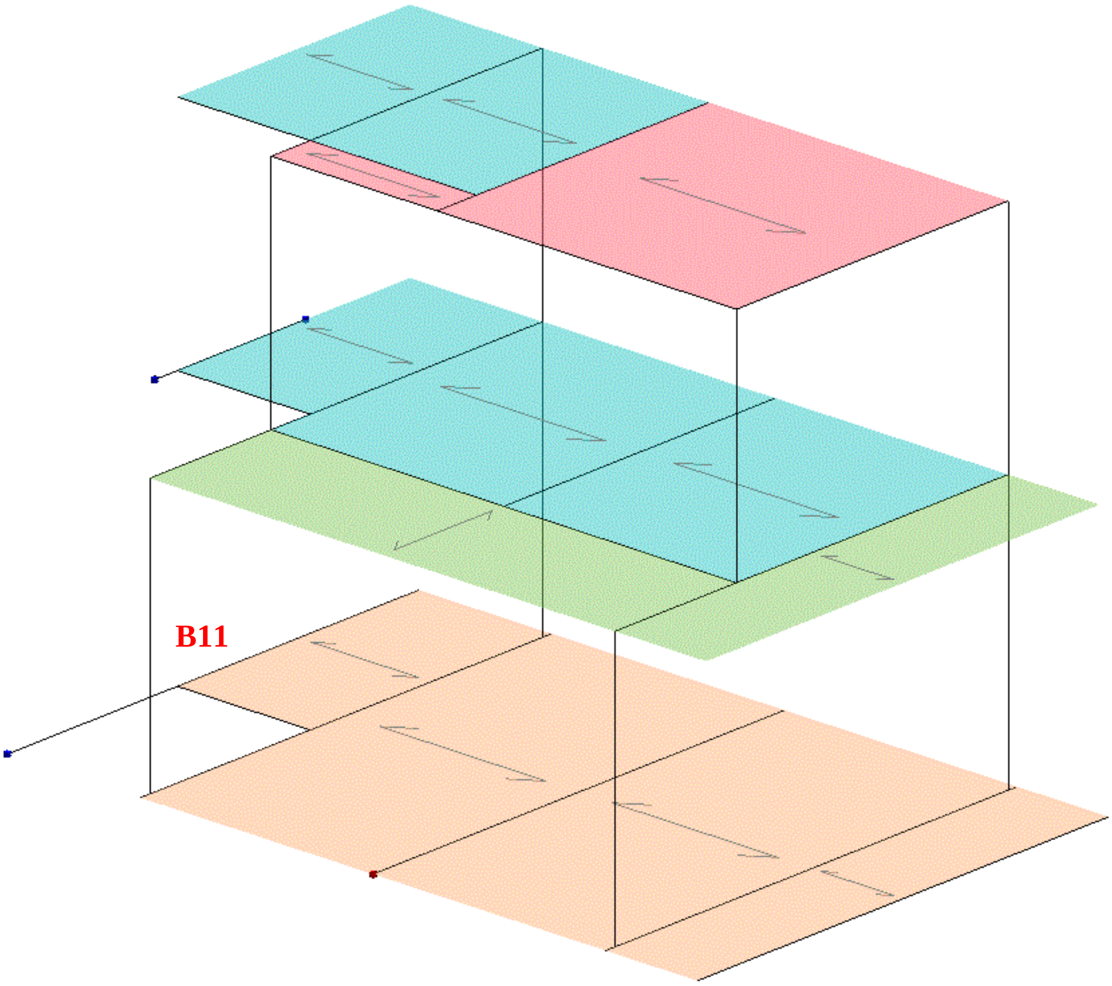
Deflection Check (Shear Deflection NOT Included)

Critical Load Case 003 : Dead Plus Live (Serviceability)

$\delta = \delta_m$	12.06 ≤ L/250	12.06 mm	OK

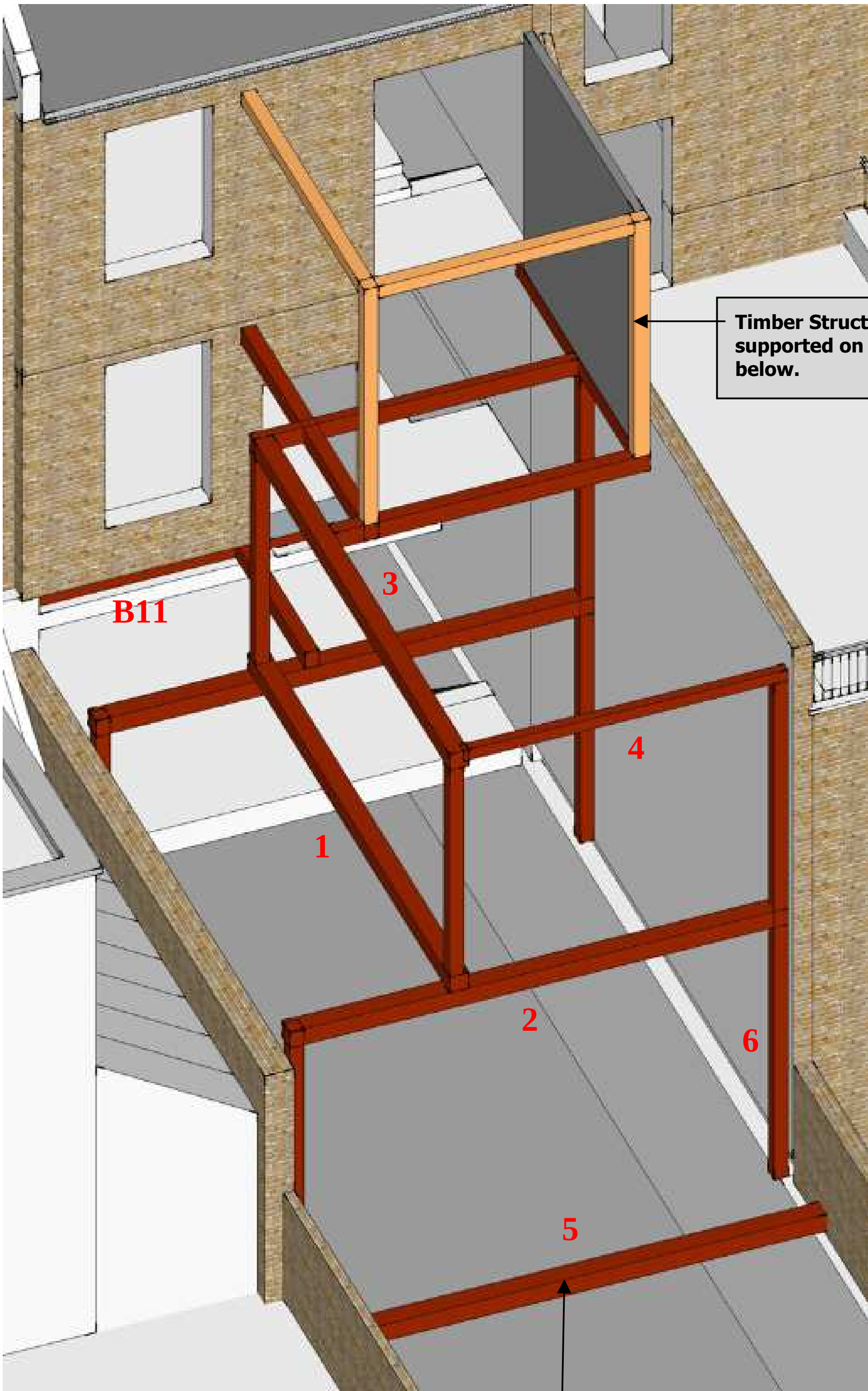
Provide **C24 Timber 47x225mm deep floor joists at 400mm centres**

Isometric view of the Rear Extension's frame showing Area Loadings



Dead D	Live L
2.600	1.500
0.500	1.500
1.000	1.500
0.600	0.750

Isometric view of the Rear Extension's frame showing Steel Sections



Numbered members' detailed designs are shown at the following pages.

AXIAL WITH MOMENTS (MEMBER)

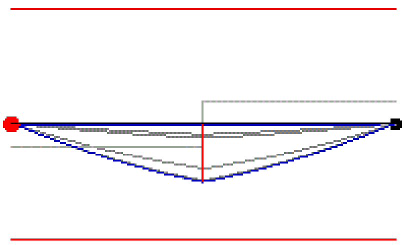
1

Members 42-43 (N.23-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 -000.604          ( kN/m )
L1 UDLY -000.604          ( kN/m )
D1 D     078.500          ( kN/m³)
L1 UDLY -002.900          ( kN/m )
Part 2
D1 UDLY -000.604          ( kN/m )
L1 UDLY -000.604          ( kN/m )
D1 D     078.500          ( kN/m³)
L1 UDLY -002.900          ( kN/m )
```



Member Forces in Load Case 1 and Maximum Deflection from Load Case 9										
Mem ber No.	Node End1 End2	Axial Force (kN)	Torque Moment (kN.m)	Shear Force (kN) y-y	z-z	Bending Moment (kN.m) y-y	z-z	Maximum Moment (kN.m @ m) y-y	z-z	Maximum Deflection (mm @ m)
	23	0.88T	27.54	34.11	0.14	-1.46	-0.01	67.80	0.38	13.69
	27	0.88T	27.54	-33.98	-0.13	-1.11	0.00	@ 2.800	@ 2.800	@ 2.800

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, 67.8, 0.38	(Axial: Non-Slender)	Class 1
Auto Design Load Cases	1 and 4-7		

Local Capacity Check

$V_{y.Ed}/V_{pl.y.Rd}$	15.366 / 269.498 =	0.057	Low Shear
$M_{c.y.Rd} = f_y.W_{pl.y}/\gamma_{M0}$	275 x 497.4/1	136.785 kN.m	
$V_{z.Ed}/V_{pl.z.Rd}$	0.137 / 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	1615.075 kN	
$n = N_{Ed}/N_{pl.Rd}$	-0.877 / 1615.075 =	0.001	OK
$W_{pl.N.y} = Fn(W_{pl.y}, A_{vy}, n)$	497.4, 16.974, 0.001	497.4 cm³	
$M_{N.y.Rd} = W_{pl.N.y}.f_y/\gamma_{M0}$	497.4 x 275/1	136.785 kN.m	
$W_{pl.N.z} = Fn(W_{pl.z}, A_{vz}, n)$	230.9, 44.792, 0.001	230.9 cm³	
$M_{N.z.Rd} = W_{pl.N.z}.f_y/\gamma_{M0}$	230.9 x 275/1	63.498 kN.m	
$(M_{y.Ed}/M_{N.y.Rd})+(M_{z.Ed}/M_{N.z.Rd})$	$(67.804/136.785)^2+(0.376/63.498)^1=$	0.252	OK

Lateral Buckling Check M.b.Rd

$M_{b.Rd} = M_{c.y.Rd}$	Fully Restrained	136.785 kN.m	

Buckling Resistance

$U_{N.y} = N_{Ed}/(\chi_y.N_{Rk}/\gamma_{M1})$	0 / 1236.682	0.000	OK
$U_{N.z} = N_{Ed}/(\chi_z.N_{Rk}/\gamma_{M1})$	0 / 1615.075	0.000	OK
$U_{M.y} = M_{y.Ed}/(\chi_{LT}.M_{y.Rk}/\gamma_{M1})$	67.804 / 136.785	0.496	OK
$U_{M.z} = M_{z.Ed}/(M_{z.Rk}/\gamma_{M1})$	0.376 / 63.498	0.006	OK
$k_{yy}=C_{my}\{1+(\lambda_y-0.2)U_{N.y}\}$		0.898	
$k_{zz}=C_{mz}\{1+(2\lambda_z-0.6)U_{N.z}\}$		0.901	
$k_{yz}=0.6\ k_{zz}$		0.541	
$k_{zy}=0.6\ k_{yy}$		0.539	
$U_{Ny}+k_{yy}.U_{M.y}+k_{yz}.U_{M.z}$	0.000+0.898x0.496+0.541x0.006	0.448	OK
$U_{Nz}+k_{zy}.U_{M.y}+k_{zz}.U_{M.z}$	0.000+0.539x0.496+0.901x0.006	0.272	OK

Deflection Check - Load Case 2

Deflection Limits (Internal Beams)	$\delta \leq 5600/360 = 15.6$ mm Live (Case 2)	10.14 mm	OK
	$\delta \leq 5600/250 = 22.4$ mm D+L (Case 3)	13.69 mm	OK

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

AXIAL WITH MOMENTS (MEMBER)

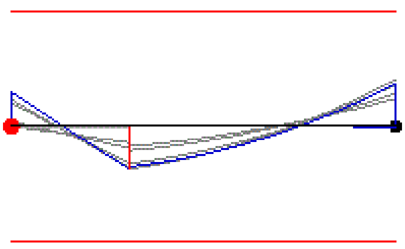
2

Members 15-16 (N.20-N.23) @ 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 -000.413            ( kN/m )
L1 UDLY -000.413            ( kN/m )
D1 D     078.500            ( kN/m³)
Part 2
D1 UDLY -000.413            ( kN/m )
L1 UDLY -000.413            ( kN/m )
D1 D     078.500            ( kN/m³)
D1 UDLY -002.900            ( kN/m )
```



Member Forces in Load Case 1 and Maximum Deflection from Load Case 9											
Mem ber No.	Node End1 End2	Axial Force (kN)	Torque Moment (kN.m)	Shear Force (kN)		Bending Moment (kN.m)		Maximum Moment (kN.m @ m)		Maximum Deflection (mm @ m)
				y-y	z-z	y-y	z-z	y-y	z-z	
	20	23.90C	0.02	58.43	-0.01	-40.81	0.00	50.03	-0.02	5.80
	24	11.43C	0.00	-44.88	0.00	-50.63	0.01	@ 1.594	@ 1.530	@ 2.222

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, 23.9, 50.93, 0.02	(Axial: Non-Slender)	Class 1
Auto Design Load Cases	1 and 4-7		

Local Capacity Check

$V_{y.Ed}/V_{pl.y.Rd}$	55.532 / 269.498 =	0.206	Low Shear
$M_{c.y.Rd} = f_y.W_{pl.y}/\gamma_{M0}$	275 x 497.4/1	136.785 kN.m	
$V_{z.Ed}/V_{pl.z.Rd}$	0.007 / 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	
$N_{pl.Rd} = A_g.f_y/\gamma_{M0}$	58.73 x 275/1 =	1615.075 kN	
$n = N_{Ed}/N_{pl.Rd}$	11.432 / 1615.075 =	0.007	OK
$W_{pl.N.y} = Fn(W_{pl.y}, A_{vy}, n)$	497.4, 16.974, 0.007	497.4 cm³	
$M_{N.y.Rd} = W_{pl.N.y}.f_y/\gamma_{M0}$	497.4 x 275/1	136.785 kN.m	
$W_{pl.N.z} = Fn(W_{pl.z}, A_{vz}, n)$	230.9, 44.792, 0.007	230.9 cm³	
$M_{N.z.Rd} = W_{pl.N.z}.f_y/\gamma_{M0}$	230.9 x 275/1	63.498 kN.m	
$(M_{y.Ed}/M_{N.y.Rd})+(M_{z.Ed}/M_{N.z.Rd})$	$(50.927/136.785)^2+(0.016/63.498)^1=$	0.139	OK

Compression Resistance N.b.Rd

$\lambda y = \sqrt{A.fy/Ncr}$	$\sqrt{58.73x275/3490.01}$	0.68	
$N_{b.y.Rd}$ = Area.$\chi.f_y/\gamma_{M1}$	58.73x0.795x275/10/1 =	1283.423 kN	Curve b
$\lambda z = \sqrt{A.fy/Ncrz}$	$\sqrt{58.73x275/1184.36}$	1.168	
$N_{b.z.Rd}$ = Area.$\chi.f_y/\gamma_{M1}$	58.73x0.45x275/10/1 =	726.039 kN	Curve c
Let = Kt.Lx	1x5.21 =	5.21	
$\lambda T = \sqrt{A.fy/NcrT}$	$\sqrt{58.73x275/2764.03}$	0.764	
$N_{b.T.Rd}$= Area.χ.fy/ γ_{M1}	58.73x0.685x275/10/1 =	1105.527 kN	Curve c

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

C_1= fn(M_1, M_2, M_o, ψ, μ)	-40.8, -50.6, 78.7, 0.806, -1.555	1.943	Uniform
C_{mLT}=Max(0.1-0.8α_s, 0.4)	M_h= -50.59, M_s= 33, ψ = 0.806, α_s= -0.652	0.622	Table B.3
C_{mz}=Max(0.6+0.4ψ, 0.4)	M = 0.01, ψ = -0.714	0.4	Table B.3
C_{my}=Max(0.1-0.8α_s, 0.4)	M_h= -50.63, M_s= 33, ψ = 0.806, α_s= -0.652	0.621	Table B.3

Lateral Buckling Check M.b.Rd

Le = 1.2L+2D	1.2 x 5.21 + 2 x 0.203 =	6.658 m	
M_{cr}= Fn($C_1, L_e, I_z, I_t, I_w, E$)	1.943, 6.658, 1551, 22.15, 0.1429, 210000	259.392 kN.m	
$\lambda_{LT}= \sqrt{W.f_y/M_{cr}}$	$\sqrt{497.4 \times 275 / 259.392}$	0.726	
χ_{LT}= Fn(λ_{LT}, λ_{LT5950})	0.726, 0.913	0.856	Curve b
χ_{LT}.mod = Fn($\chi_{LT}, \lambda_{LT}, k_c, f$)	0.856, 0.726, 0.717, 0.860	0.996	6.3.2.3
$M_{b.Rd} = \chi\ W_{pl.y}.f_y \leq M_{c.y.Rd}$	0.996 x 497.4 x 275 ≤ 136.785 =	136.174 kN.m	

Buckling Resistance

$U_{N.y} = N_{Ed}/(\chi_y.N_{Rk}/\gamma_{M1})$	23.898 / 1283.423	0.019	OK
$U_{N.z} = N_{Ed}/(\chi_z.N_{Rk}/\gamma_{M1})$	23.898 / 726.039	0.033	OK
$U_{M.y} = M_{y.Ed}/(\chi_{LT}.M_{y.Rk}/\gamma_{M1})$	50.927 / 136.174	0.374	OK
$U_{M.z} = M_{z.Ed}/(M_{z.Rk}/\gamma_{M1})$	0.016 / 63.498	0.000	OK
$k_{yy}=C_{my}\{1+(\lambda_y-0.2)U_{N.y}\}$		0.627	
$k_{zz}=C_{mz}\{1+1.4U_{N.z}\}$		0.418	
$k_{yz}=0.6\ k_{zz}$		0.251	
$k_{zy}=1-\{0.1\lambda_z/(C_{mLT}-0.25)\}U_{N.z}$		0.990	
$U_{Ny}+k_{yy}.U_{M.y}+k_{yz}.U_{M.z}$	0.019+0.627x0.374+0.251x0.000	0.253	OK
$U_{Nz}+k_{zy}.U_{M.y}+k_{zz}.U_{M.z}$	0.033+0.990x0.374+0.418x0.000	0.403	OK

Deflection Check - Load Case 3

Deflection Limits (Internal Beams)	$\delta \leq 5210/360$ = 14.5 mm Live (Case 2)	2.93 mm	OK
	$\delta \leq 5210/250$ = 20.8 mm D+L (Case 3)	5.59 mm	OK

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

AXIAL WITH MOMENTS (MEMBER)

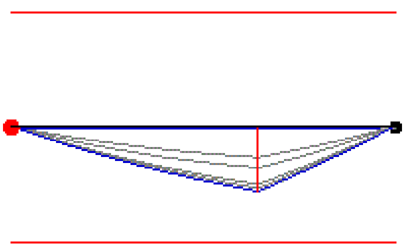
3

Members 39-40 (N.34-N.36) @ Level 1 in Load Case 1

Member Loading and Member Forces

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

Part 1
D1 D      078.500              ( kN/m³)
D1 UDLY -001.500               ( kN/m )
Part 2
D1 D      078.500              ( kN/m³)
D1 UDLY -001.500               ( kN/m )
```



Member Forces in Load Case 1 and Maximum Deflection from Load Case 9										
Mem ber No.	Node End1 End2	Axial Force (kN)	Torque Moment (kN.m)	Shear Force (kN)		Bending Moment (kN.m)		Maximum Moment (kN.m @ m)		Maximum Deflection (mm @ m)
				y-y	z-z	y-y	z-z	y-y	z-z	
	34	0.87C	0.00	17.87	0.12	-0.77	-0.01	47.81	0.45	15.17
	37	0.84C	0.00	-26.88	-0.23	-1.08	-0.01	@ 3.600	@ 3.600	@ 2.952

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, 0.87, 47.82, 0.45	(Axial: Non-Slender)	Class 1
Auto Design Load Cases	1 and 4-7		

Local Capacity Check

$V_{y.Ed}/V_{pl.y.Rd}$	22.016 / 231.406 =	0.095	Low Shear
$M_{c.y.Rd} = f_y.W_{pl.y}/\ \gamma_{M0}$	275 x 314.4/1	86.46 kN.m	
$V_{z.Ed}/V_{pl.z.Rd}$	0.229 / 408.182 =	0.001	Low Shear
$M_{c.z.Rd} = f_y.W_{pl.z}/\ \gamma_{M0}$	275 x 88.2/1	24.255 kN.m	
$N_{pl.Rd} = A_g.f_y/\ \gamma_{M0}$	38.21 x 275/1 =	1050.775 kN	
$n = N_{Ed}/N_{pl.Rd}$	0.842 / 1050.775 =	0.001	OK
$W_{pl.N.y}$ = Fn($W_{pl.y}$, A_{vy}, n)	314.4, 14.575, 0.001	314.4 cm³	
$M_{N.y.Rd} = W_{pl.N.y}.f_y/\ \gamma_{M0}$	314.4 x 275/1	86.46 kN.m	
$W_{pl.N.z}$ = Fn($W_{pl.z}$, A_{vz}, n)	88.2, 25.709, 0.001	88.2 cm³	
$M_{N.z.Rd} = W_{pl.N.z}.f_y/\ \gamma_{M0}$	88.2 x 275/1	24.255 kN.m	
$(M_{y.Ed}/M_{N.y.Rd})+(M_{z.Ed}/M_{N.z.Rd})$	$(47.809/86.46)^2+(0.448/24.255)^1=$	0.324	OK

Compression Resistance N.b.Rd

$\lambda y = \sqrt{A.fy/Ncr}$	√38.21x275/1914.33	0.741	
$N_{b.y.Rd} = Area.\chi.f_y/\ \gamma_{M1}$	38.21x0.828x275/10/1 =	869.891 kN	Curve a
$\lambda z = \sqrt{A.fy/Ncrz}$	√38.21x275/254.78	2.028	
$N_{b.z.Rd} = Area.\chi.f_y/\ \gamma_{M1}$	38.21x0.204x275/10/1 =	214.622 kN	Curve b
Let = Kt.Lx	1x5.6 =	5.6	
$\lambda T = \sqrt{A.fy/NcrT}$	√38.21x275/1255.13	0.915	
$N_{b.T.Rd} = Area.\chi.fy/\ \gamma_{M1}$	38.21x0.652x275/10/1 =	684.626 kN	Curve b

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

C_1= fn(M_1, M_2, M_o, ψ,μ)	-0.7, -1.1, 40.6, 0.708, -38.493	1.196	Uniform
$C_{mLT}=0.95+0.05\alpha_h$	M_h= -1.06, M_s= 39.75, ψ = 0.708, α_s= -0.027	0.949	Table B.3
$C_{mz}=0.95+0.05\alpha_h$	M_h= -0.01, M_s= 0.34, ψ = 0.700, α_s= -0.030	0.949	Table B.3
$C_{my}=0.95+0.05\alpha_h$	M_h= -1.08, M_s= 39.75, ψ = 0.707, α_s= -0.027	0.949	Table B.3

Lateral Buckling Check M.b.Rd

Le = 1.00 L	1 x 5.6 =	5.6 m	
M_{cr}= Fn(C_1,L_e,I_z,I_t,I_w,E)	1.196, 5.600, 385.5, 10.3, 0.03734, 210000	62.687 kN.m	
$\lambda_{LT} = \sqrt{W.f_y/M_{cr}}$	√ 314.4 x 275 / 62.687	1.174	
χ_{LT}= Fn(λ_{LT}, λ_{LT5950})	1.174, 1.237	0.594	Curve b
χ_{LT}.mod = Fn(χ_{LT},λ_{LT},k_c,f)	0.594, 1.174, 0.915, 0.969	0.613	6.3.2.3
$M_{b.Rd} = \chi\ W_{pl.y}.f_y \leq M_{c.y.Rd}$	0.613 x 314.4 x 275 ≤ 86.460 =	52.997 kN.m	

Buckling Resistance

$U_{N.y} = N_{Ed}/(\chi_y.N_{Rk}/\gamma_{M1})$	0.868 / 869.891	0.001	OK

$U_{N.z} = N_{Ed}/(\chi_z.N_{Rk}/\gamma_{M1})$	0.868 / 214.622	0.004	OK
$U_{M.y} = M_{y.Ed}/(\chi_{LT}.M_{y.Rk}/\gamma_{M1})$	47.809 / 52.997	0.902	OK
$U_{M.z} = M_{z.Ed}/(M_{z.Rk}/\gamma_{M1})$	0.448 / 24.255	0.018	OK
$k_{yy}=C_{my}\{1+(\lambda_y-0.2)U_{N.y}\}$		0.949	
$k_{zz}=C_{mz}\{1+1.4U_{N.z}\}$		0.954	
$k_{yz}=0.6\ k_{zz}$		0.572	
$k_{zy}= 1- \{0.1\lambda_z/(C_{mLT}-0.25)\}U_{N.z}$		0.999	
$U_{Ny}+k_{yy}.U_{M.y}+k_{yz}.U_{M.z}$	0.001+0.949x0.902+0.572x0.018	0.868	OK
$U_{Nz}+k_{zy}.U_{M.y}+k_{zz}.U_{M.z}$	0.004+0.999x0.902+0.954x0.018	0.923	OK

Deflection Check - Load Case 3

Deflection Limits (Internal Beams)	$\delta \leq 5600/360 = 15.6$ mm Live (Case 2)	4.88 mm	OK
	$\delta \leq 5600/250 = 22.4$ mm D+L (Case 3)	15.17 mm	OK

Provide Section (29.99 kg/m) **203x133 UB 30 [S 275]**

AXIAL WITH MOMENTS (MEMBER)

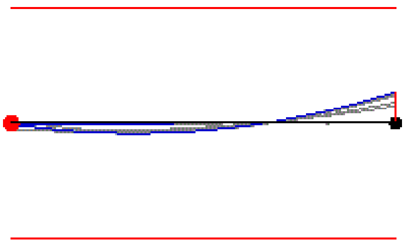
4

Member 19 (N.34-N.35) @ Level 1 in Load Case 6

Member Loading and Member Forces

```
Loading Combination : 1 UT + 1.25 D1 + 1.5 L1 + 0.75 W1

D1 UDLY -001.800            ( kN/m )
L1 UDLY -002.700            ( kN/m )
D1 D     078.500            ( kN/m³)
D1 UDLY -001.500            ( kN/m )
```



Member Forces in Load Case 6 and Maximum Deflection from Load Case 9										
Mem ber No.	Node End1 End2	Axial Force (kN)	Torque Moment (kN.m)	Shear Force (kN)		Bending Moment (kN.m)		Maximum Moment (kN.m @ m)		Maximum Deflection (mm @ m)
				y-y	z-z	y-y	z-z	y-y	z-z	
19	34	12.83C	0.00	9.82	0.00	0.50	0.01	6.18		0.64
	35	12.83C	0.00	-20.74	0.00	-19.15	0.00	@ 1.152		@ 1.332

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, 12.83, 19.15, 0.01	(Axial: Non-Slender)	Class 1
Auto Design Load Cases	1 and 4-7		

Local Capacity Check

$V_{y.Ed}/V_{pl.y.Rd}$	20.738 / 203.402 =	0.102	Low Shear
$M_{c.y.Rd} = f_y.W_{pl.y}/\gamma_{M0}$	275 x 257.7/1	70.868 kN.m	
$V_{z.Ed}/V_{pl.z.Rd}$	0.003 / 329.914 =	0	Low Shear
$M_{c.z.Rd} = f_y.W_{pl.z}/\gamma_{M0}$	275 x 70.9/1	19.498 kN.m	
$N_{pl.Rd} = A_g.f_y/\gamma_{M0}$	31.96 x 275/1 =	878.9 kN	
$n = N_{Ed}/N_{pl.Rd}$	12.827 / 878.9 =	0.015	OK
$W_{pl.N.y}$ = Fn($W_{pl.y}$, A_{vy}, n)	257.7, 12.811, 0.015	257.7 cm³	
$M_{N.y.Rd} = W_{pl.N.y}.f_y/\gamma_{M0}$	257.7 x 275/1	70.868 kN.m	
$W_{pl.N.z}$ = Fn($W_{pl.z}$, A_{vz}, n)	70.9, 20.779, 0.015	70.9 cm³	
$M_{N.z.Rd} = W_{pl.N.z}.f_y/\gamma_{M0}$	70.9 x 275/1	19.498 kN.m	
$(M_{y.Ed}/M_{N.y.Rd})+(M_{z.Ed}/M_{N.z.Rd})$	$(19.15/70.868)^2+(0.003/19.498)^1=$	0.073	OK

Compression Resistance N.b.Rd

$\lambda y = \sqrt{A.fy/Ncr}$	√31.96x275/3743.98	0.484	
$N_{b.y.Rd} = Area.\chi.f_y/\gamma_{M1}$	31.96x0.929x275/10/1 =	816.529 kN	Curve a
$\lambda z = \sqrt{A.fy/Ncrz}$	√31.96x275/493.37	1.333	
$N_{b.z.Rd} = Area.\chi.f_y/\gamma_{M1}$	31.96x0.411x275/10/1 =	361.352 kN	Curve b
Let = Kt.Lx	1x3.6 =	3.6	
$\lambda T = \sqrt{A.fy/NcrT}$	√31.96x275/1146.18	0.876	
$N_{b.T.Rd} = Area.\chi.fy/\gamma_{M1}$	31.96x0.677x275/10/1 =	594.828 kN	Curve b

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

C_1= fn(M_1, M_2, M_o, ψ,μ)	0.5, -19.1, 13.7, -0.027, -0.718	3.737	Uniform
C_{mLT}=Max(0.1(1-ψ) -0.8α_s, 0.4)	M_h= -19.13, M_s= 4.42, ψ = -0.027, α_s= -0.231	0.4	Table B.3
C_{mz}=Max(0.6+0.4ψ, 0.4)	M = 0.01, ψ = -0.500	0.4	Table B.3
C_{my}=Max(0.1(1-ψ) -0.8α_s, 0.4)	M_h= -19.15, M_s= 4.42, ψ = -0.026, α_s= -0.231	0.4	Table B.3

Lateral Buckling Check M.b.Rd

Le = 1.2L+2D	1.2 x 3.6 + 2 x 0.203 =	4.726 m	
M_{cr}= Fn(C_1,L_e,I_z,I_t,I_w,E)	3.737, 4.726, 308.5, 5.964, 0.02933, 210000	173.568 kN.m	
$\lambda_{LT} = \sqrt{W.f_y/M_{cr}}$	√ 257.7 x 275 / 173.568	0.639	
χ_{LT}= Fn(λ_{LT}, λ_{LT5950})	0.639, 1.189	0.899	Curve b
χ_{LT}.mod = Fn(χ_{LT},λ_{LT},k_c,f)	0.899, 0.639, 0.517, 0.771	1.000	6.3.2.3
$M_{b.Rd} = \chi\ W_{pl.y}.f_y \leq M_{c.y.Rd}$	1.000 x 257.7 x 275 ≤ 70.868 =	70.868 kN.m	

Buckling Resistance

$U_{N.y} = N_{Ed}/(\chi_y.N_{Rk}/\gamma_{M1})$	12.827 / 816.529	0.016	OK

$U_{N.z} = N_{Ed}/(\chi_z.N_{Rk}/\gamma_{M1})$	12.827 / 361.352	0.035	OK
$U_{M.y} = M_{y.Ed}/(\chi_{LT}.M_{y.Rk}/\gamma_{M1})$	19.15 / 70.868	0.270	OK
$U_{M.z} = M_{z.Ed}/(M_{z.Rk}/\gamma_{M1})$	0.003 / 19.498	0.000	OK
$k_{yy}=C_{my}\{1+(\lambda_y-0.2)U_{N.y}\}$		0.402	
$k_{zz}=C_{mz}\{1+1.4U_{N.z}\}$		0.420	
$k_{yz}=0.6\ k_{zz}$		0.252	
$k_{zy}=0.6\ k_{yy}$		0.241	
$U_{Ny}+k_{yy}.U_{M.y}+k_{yz}.U_{M.z}$	0.016+0.402x0.270+0.252x0.000	0.124	OK
$U_{Nz}+k_{zy}.U_{M.y}+k_{zz}.U_{M.z}$	0.035+0.241x0.270+0.420x0.000	0.101	OK

Deflection Check - Load Case 3

Deflection Limits (Internal Beams)	$\delta \leq 3600/360 = 10$ mm Live (Case 2)	0.19 mm	OK
	$\delta \leq 3600/250 = 14.4$ mm D+L (Case 3)	0.54 mm	OK

Provide Section (25.09 kg/m) **203x133 UB 25 [S 275]**

Axial with Moments (Member)

5

Member 27 (N.3-N.4) @ Level 1 in Load Case 1

Member Loading and Member Forces

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

D1 UDLY -001.430            ( kN/m )
L1 UDLY -000.825            ( kN/m )
D1 D     078.500            ( kN/m³)
D1 UDLY -002.950            ( kN/m )
L1 UDLY -000.750            ( kN/m )
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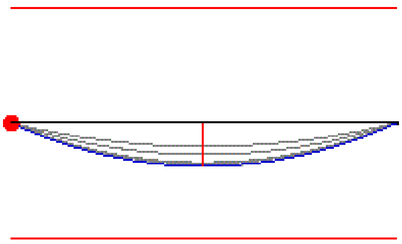


Member Forces in Load Case 1 and Maximum Deflection from Load Case 3										
Mem ber No.	Node End1 End2	Axial Force (kN)	Torque Moment (kN.m)	Shear Force (kN)		Bending Moment (kN.m)		Maximum Moment (kN.m @ m)		Maximum Deflection (mm @ m)
				y-y	z-z	y-y	z-z	y-y	z-z	
27	3	0.00C	0.00	23.65	0.00	0.00	0.00	32.23		11.81
	4	0.00C	0.00	-23.65	0.00	0.00	0.00	@ 2.725		@ 2.725

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, 32.22, 0	(Axial: Non-Slender)	Class 1
Auto Design Load Cases	1 and 4-7		

Moment Capacity Check M.c.y.Rd

$V_{y.Ed}/V_{pl.y.Rd}$	0.003 / 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	
$M_{y.Ed}/M_{c.y.Rd}$	32.224 / 86.46 =	0.373	OK

Equivalent Uniform Moment Factor C1

C_1= fn(M_1, M_2, M_o, ψ,μ)	0.0, 0.0, 32.2, 0.875, 300.000	1.127	Uniform

Lateral Buckling Check M.b.Rd

Le = 1.2L+2D	1.2 x 5.45 + 2 x 0.207 =	6.954 m	
M_{cr}= Fn(C_1,L_e,I_z,I_t,I_w,E)	1.127, 6.954, 385.5, 10.3, 0.03734, 210000	45.638 kN.m	
$\lambda_{LT} = \sqrt{W.f_y/M_{cr}}$	$\sqrt{}$ 314.4 x 275 / 45.638	1.376	
χ_{LT}= Fn(λ_{LT}, λ_{LT5950})	1.376, 1.408	0.484	Curve b
χ_{LT}.mod = Fn(χ_{LT},λ_{LT},k_c,f)	0.484, 1.376, 0.942, 0.990	0.489	6.3.2.3
$M_{b.Rd} = \chi\ W_{pl.y}.f_y \le M_{c.y.Rd}$	0.489 x 314.4 x 275 ≤ 86.460 =	42.288 kN.m	
$M_{y.Ed}/M_{b.Rd}$	32.224 / 42.288	0.762	OK

Deflection Check - Load Case 3

Deflection Limits (Internal Beams)	$\delta \le$ 5450/360 = 15.1 mm Live (Case 2)	2.97 mm	OK
	$\delta \le$ 5450/250 = 21.8 mm D+L (Case 3)	11.81 mm	OK

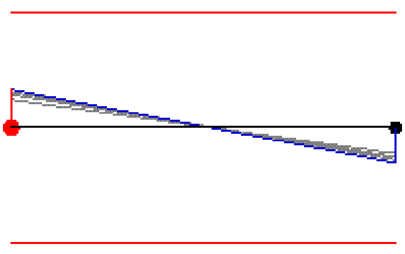
Provide Section (29.99 kg/m) 203x133 UB 30 [S 275]

AXIAL WITH MOMENTS (MEMBER)
6
Member 59 (N.7-N.24) @ Level 1 in Load Case 6

Member Loading and Member Forces

```
Loading Combination : 1 UT + 1.25 D1 + 1.5 L1 + 0.75 W1

D1 D      078.500              ( kN/m³)
```



Member Forces in Load Case 6 and Maximum Deflection from Load Case 9										
Mem ber No.	Node End1 End2	Axial Force (kN)	Torque Moment (kN.m)	Shear Force (kN)		Bending Moment (kN.m)		Maximum Moment (kN.m @ m)		Maximum Deflection (mm @ m)
				y-y	z-z	y-y	z-z	y-y	z-z	
59	7	70.27C	0.00	28.78	0.00	-44.83	0.00		0.00	0.59
	24	68.55C	0.00	28.78	0.00	41.52	-0.01		@ 0.000	@ 0.720

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, 70.27, 44.83, 0.01	(Axial: Non-Slender)	Class 1
Auto Design Load Cases	1 and 4-7		

Local Capacity Check

$V_{y.Ed}/V_{pl.y.Rd}$	28.784 / 269.498 =	0.107	Low Shear
$M_{c.y.Rd} = f_y.W_{pl.y}/\ \gamma_{M0}$	275 x 497.4/1	136.785 kN.m	
$V_{z.Ed}/V_{pl.z.Rd}$	0.001 / 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	
$N_{pl.Rd} = A_g.f_y/\ \gamma_{M0}$	58.73 x 275/1 =	1615.075 kN	
$n = N_{Ed}/N_{pl.Rd}$	68.546 / 1615.075 =	0.042	OK
$W_{pl.N.y} = Fn(W_{pl.y}, A_{vy}, n)$	497.4, 16.974, 0.042	497.4 cm³	
$M_{N.y.Rd} = W_{pl.N.y}.f_y/\ \gamma_{M0}$	497.4 x 275/1	136.785 kN.m	
$W_{pl.N.z} = Fn(W_{pl.z}, A_{vz}, n)$	230.9, 44.792, 0.042	230.9 cm³	
$M_{N.z.Rd} = W_{pl.N.z}.f_y/\ \gamma_{M0}$	230.9 x 275/1	63.498 kN.m	
$(M_{y.Ed}/M_{N.y.Rd})+(M_{z.Ed}/M_{N.z.Rd})$	$(44.834/136.785)^2+(0.004/63.498)^1=$	0.107	OK

Compression Resistance N.b.Rd

$\lambda y = \sqrt{A.fy/Ncr}$	$\sqrt{58.73x275/10525.9}$	0.392	
$N_{b.y.Rd} = Area.\chi.f_y/\ \gamma_{M1}$	58.73x0.929x275/10/1 =	1500.914 kN	Curve b
$\lambda z = \sqrt{A.fy/Ncrz}$	$\sqrt{58.73x275/3572.04}$	0.672	
$N_{b.z.Rd} = Area.\chi.f_y/\ \gamma_{M1}$	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	
$N_{b.T.Rd}= Area.\chi.fy/\ \gamma_{M1}$	58.73x0.8x275/10/1 =	1291.661 kN	Curve c

Equivalent Uniform Moment Factor C1

$C_1= fn(M_1, M_2, \psi)$	-44.8, 41.5, -0.926	2.550	Not Loaded
$C_{mLT}=Max(0.6+0.4\psi, 0.4)$	M = -44.81, ψ = -0.926	0.4	Table B.3
$C_{mz}=Max(0.6+0.4\psi, 0.4)$	M = -0.01, ψ = 0.500	0.8	Table B.3
$C_{my}=Max(0.6+0.4\psi, 0.4)$	M = -44.83, ψ = -0.926	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	
$\chi_{LT}= Fn(\lambda_{LT}, \lambda_{LT5950})$	0.446, 0.642	0.982	Curve b
$\chi_{LT}.mod = Fn(\chi_{LT},\lambda_{LT},k_c,f)$	0.982, 0.446, 0.626, 0.860	1.000	6.3.2.3
$M_{b.Rd} = \chi\ W_{pl.y}.f_y \leq M_{c.y.Rd}$	1.000 x 497.4 x 275 ≤ 136.785 =	136.785 kN.m	

Buckling Resistance

$U_{N.y} = N_{Ed}/(\chi_y.N_{Rk}/\gamma_{M1})$	70.275 / 1500.914	0.047	OK

$U_{N.z} = N_{Ed}/(\chi_z.N_{Rk}/\gamma_{M1})$	70.275 / 1197.991	0.059	OK
$U_{M.y} = M_{y.Ed}/(\chi_{LT}.M_{y.Rk}/\gamma_{M1})$	44.834 / 136.785	0.328	OK
$U_{M.z} = M_{z.Ed}/(M_{z.Rk}/\gamma_{M1})$	0.004 / 63.498	0.000	OK
$k_{yy}=C_{my}\{1+(\lambda_y-0.2)U_{N.y}\}$		0.404	
$k_{zz}=C_{mz}\{1+(2\lambda_z-0.6)U_{N.z}\}$		0.835	
$k_{yz}=0.6\ k_{zz}$		0.501	
$k_{zy}=0.6\ k_{yy}$		0.242	
$U_{Ny}+k_{yy}.U_{M.y}+k_{yz}.U_{M.z}$	0.047+0.404x0.328+0.501x0.000	0.179	OK
$U_{Nz}+k_{zy}.U_{M.y}+k_{zz}.U_{M.z}$	0.059+0.242x0.328+0.835x0.000	0.138	OK

Deflection Check - Load Case 3

Deflection Limits (Internal Beams)	$\delta \leq 3000/360 = 8.3$ mm Live (Case 2)	0.18 mm	OK
	$\delta \leq 3000/250 = 12$ mm D+L (Case 3)	0.53 mm	OK

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