BEAM DESIGNS

QUICK-SIMPLE-EFFICIENT

Structural Calculations

BD

Revision	Date	Changes
-		Initial Issue

STRUCTURAL ENGINEERING ALLIANCE GROUP

SEAG

EST. 2024

Project No : BD220419
Sheet : **Structure /1**
Made By : JS
Checked By : CP
Date : April 2022

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. Unless noted otherwise, 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 2015 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, including fabricated structural steelwork, should be UKCA marked in accordance with current legislation and to achieve Execution Class 2 (EXC2), unless noted otherwise.
11. 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 consider:
    a. Increased wind loading by providing additional laps and fixings to roof coverings
    b. Consider foundation depth on shrinkable clays and 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 consider any local trees, then this should be checked and agreed with the Building Inspector on site.

Party Wall etc. Act 1996

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

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

								Dead		Live	
Roof (Flat Ceiling)		Tiles	=	0.65 kN/m^2							
	35	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			
								1.41 kN/m^2			
		Attic	=						=	0.25 kN/m^2	
		Roof Snow Loading	=	0.6	x ((60 - 35)/30)				=	0.50 kN/m^2	
										0.75 kN/m^2	
Roof (Sloped Ceiling)		Tiles	=	0.65 kN/m^2							
	35	Plasterboard	=	0.25 kN/m^2							
		Rafters, felt, insulation etc	=	0.30 kN/m^2							
				1.20 kN/m^2	/COS 35		=	1.46 kN/m^2			
								1.46 kN/m^2			
		Attic							=	0 kN/m^2	
		Roof Snow Loading	=	0.6	x ((60 - 35)/30)				=	0.50 kN/m^2	
										0.5 kN/m^2	
Roof (<15 deg.)		Joists, boarding and finishes					=	0.35 kN/m^2			
		Plasterboard					=	0.25 kN/m^2			
								0.60 kN/m^2			
		Imposed							=	0.75 kN/m^2	
										0.75 kN/m^2	
Floor (PCU)		PCU's					=	3.00 kN/m^2			
		Screed					=	1.80 kN/m^2			
		Plasterboard					=	0.25 kN/m^2			
								5.05 kN/m^2			
		Imposed loading							=	1.50 kN/m^2	
		Partition Loading < 1.0 kN/m							=	0.50 kN/m^2	
										2.00 kN/m^2	
Floor (timber)		Joists					=	0.15 kN/m^2			
		Boards					=	0.20 kN/m^2			
		Plasterboard					=	0.25 kN/m^2			
								0.60 kN/m^2			
		Imposed loading							=	1.50 kN/m^2	
		Partition Loading < 1.0 kN/m							=	0.50 kN/m^2	
										2.00 kN/m^2	
Walls (Height m)		100mm blockwork		2.70	x	1.40	=	3.78 kN/m			**Unit weights**
	2.7	Plasterwork both sides	=	2.70	x	0.25 x 2	=	1.35 kN/m			(x by wall hgt)
								5.13 kN/m			1.90 kN/m^2
	2.7	102.5 mm brickwork	=	2.70	x	2.10	=	5.67 kN/m			
		Plasterwork both sides	=	2.70	x	0.25 x 2	=	1.35 kN/m			(x by wall hgt)
							=	7.02 kN/m			2.60 kN/m^2
	2.4	2.4m high studwork	=	2.40	x	0.12	=	0.29 kN/m			
		Plasterwork both sides	=	2.40	x	0.15 x 2	=	0.72 kN/m			(x by wall hgt)
							=	1.01 kN/m			0.42 kN/m^2
	2.7	Cavity wall Bwk+Block	=	2.70 ‹		(2.1 + 1.4)	=	9.45 kN/m			
		Plasterwork both sides	=	2.70	x	0.15 x 2	=	0.81 kN/m			(x by wall hgt)
							=	10.26 kN/m			3.80 kN/m^2

Disproportionate collapse classification

Check the disproportionate collapse classification for the propsoed building

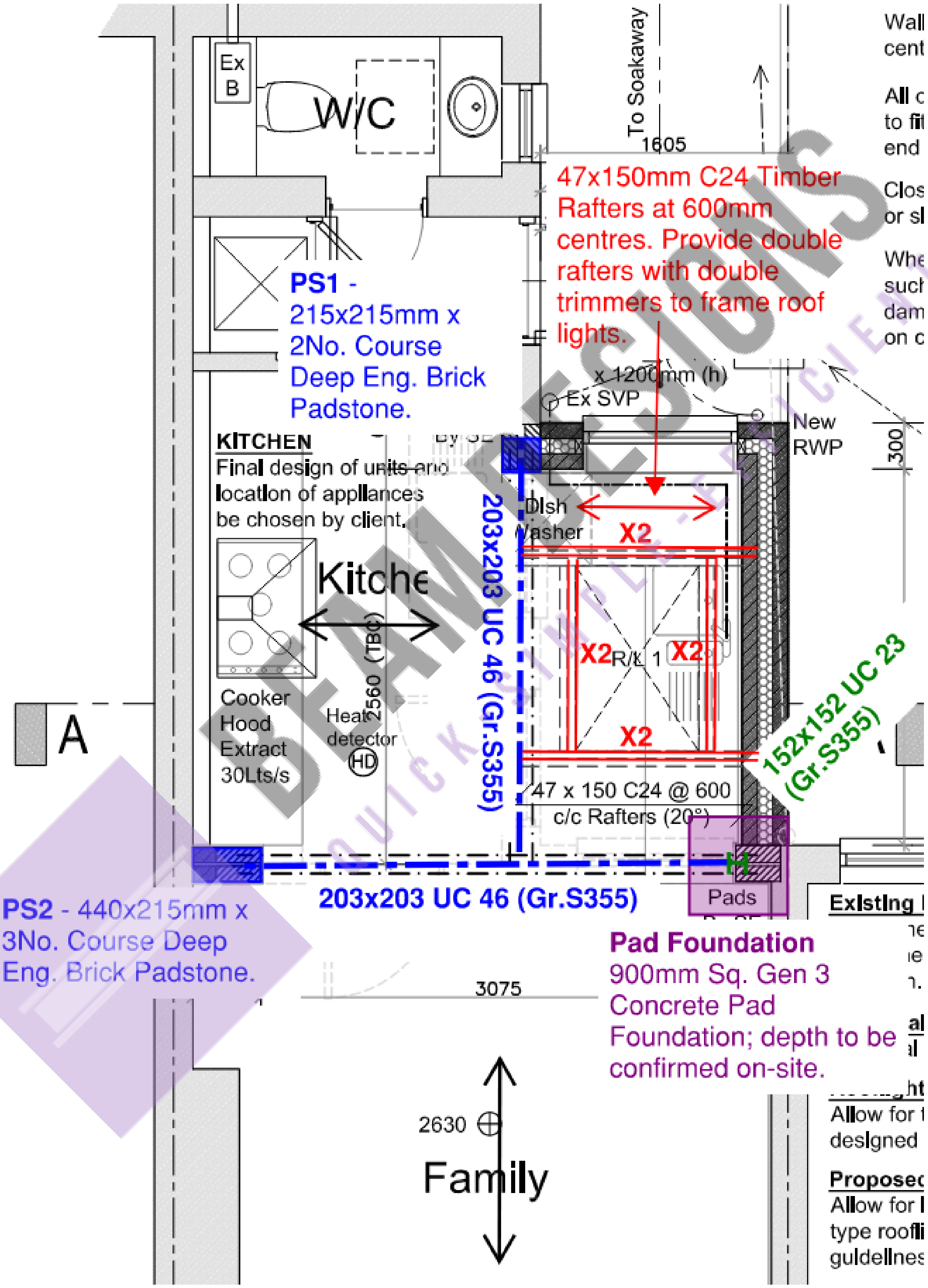
The principle use type for the building is
Residential

The structure has been determined to be a **Class 1 development.**

The disproporionate collapse building class is based on the following factors

Is the building less than or equal to 4 storeys?	**Yes**
Is the building multiple occupancy?	**No**

Part Ground Floor Plan (Structural Layout)



47x150mm C24 Timber Rafters at 600mm centres. Provide double rafters with double trimmers to frame roof lights.

PS1 - 215x215mm x 2No. Course Deep Eng. Brick Padstone.

203x203 UC 46 (Gr.S355)

152x152 UC 23 (Gr.S355)

203x203 UC 46 (Gr.S355)

PS2 - 440x215mm x 3No. Course Deep Eng. Brick Padstone.

Pad Foundation
900mm Sq. Gen 3 Concrete Pad Foundation; depth to be confirmed on-site.

Side Support Beam (Span = 2.60m)

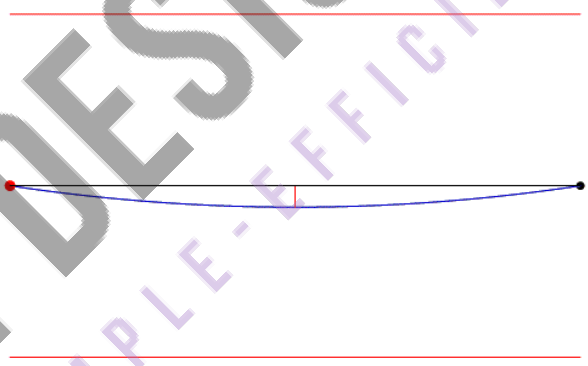
Loading:-			**Span =**	**m**
Dead Loads	**Load (kN/m²)**	**load-span / height (m)**	**Number**	**UDL (kN/m)**
Wall Load	4.75	2.5	1	11.88
Floor	0.6	2.1	1	0.63
Roof	1.41	3.6	1	2.54
Total Dead				**15 kN /m**
Live Loads	**Load**	**Span/height**	**Number**	**UDL**
Floor	2	2.1	1	2.10
Roof	0.75	3.6	1	1.35
Total Live				**3.5 kN /m**

AXIAL WITH MOMENTS (MEMBER)

Member SB L1 Id 2 @ Level 1 in Load Case 1

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

D1 D 077.010 (kN/m³)
D1 UDLY -015.000 (kN/m)
L1 UDLY -003.500 (kN/m)



Member Forces in Load Case 1 and Maximum Deflection from Load Case 2										
Member No.	**Node End 1 End 2**	**Axial Force (kN)**	**Torque Moment (kNm)**	**Shear Force (kN)**		**Bending Moment (kNm)**		**Maximum Moment (kNm @ m)**		**Max Def (mm @ m)**
				y-y	**z-z**	**y-y**	**z-z**	**y-y**	**z-z**	
3	3	0.00C	0.00	33.94	0.00	0.00	0.00	22.06	0.00	1.17
	5	0.00C	0.00	-33.94	0.00	0.00	0.00	@ 1.300	@ 0.000	@ 1.300

Classification and Effective Area (EN 1993: 2006)

Section (46.1 kg/m)	203x203 UC 46 [S 355]		
Class = Fn(b/T,d/t,f_y,N,M_y,M_z)	9.25, 22.33, 355, 0, 22.06, 0	(Axial: Non-Slender)	Class 2
Auto Design Load Cases	1		

Shear Capacity Check

$V_{y.Ed}/V_{pl.y.Rd}$	33.945 / 347.898 =	0.098	OK

Local Capacity Check

$V_{y.Ed}/V_{pl.y.Rd}$	0.003 / 347.898 =	0	Low Shear
$M_{c.y.Rd} = f_y.W_{pl.y}/\ \gamma_{M0}$	355 x 497.4/1	176.577 kN.m	
$N_{pl.Rd} = A_g.f_y/\ \gamma_{M0}$	58.73 x 355/1 =	2084.915 kN	
$n = N_{Ed}/N_{pl.Rd}$	0.002 / 2084.915 =	0.000	OK
$W_{pl.N.y} = Fn(W_{pl.y}, A_{vy},)$	497.4, 16.974, 0	497.4 cm³	
$M_{N.y.Rd} = W_{pl.N.y}.f_y/\ \gamma_{M0}$	497.4 x 355/1	176.577 kN.m	
$(M_{y.Ed}/M_{N.y.Rd+(M\_(z.Ed}/M_{N.z.Rd})$	$(22.061/176.577)^2+(0)^1=$	0.016	OK

Compression Resistance N.b.Rd

Ley = Ky.Ly	1x2.6 =	2.6	
λy = √A.fy/Ncr	√58.73x355/14013.77	0.386	
$N_{b.y.Rd} = Area.c.f_y/\ \gamma_{M1}$	58.73x0.932x355/10/1 =	1942.453 kN	Curve b
Lez = Kz.Lz	1x2.6 =	2.6	
λz = √A.fy/Ncrz	√58.73x355/4755.67	0.662	
$N_{b.z.Rd} = Area.c.f_y/\ \gamma_{M1}$	58.73x0.748x355/10/1 =	1559.628 kN	Curve c
Let = Kt.Lx	1x2.6 =	2.6	
λT = √A.fy/NcrT	√58.73x355/5921.16	0.593	

$N_{b.T.Rd}$= Area.c.fy/ γ_{M1}	58.73x0.789x355/10/1 =	1645.613 kN	Curve c

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

C_1= fn(M_1, M_2, M_o, ~y,~m)	0.0, 0.0, 22.0, 0.848, 300.000	1.127	Uniform
C_{mLT}=0.95+0.05α_h	M_h = 0.03, M_s = 22.06, ~y = 0.848, α_s = 0.001	0.95	Table B.3
C_{mz}=Max(0.6+0.4~y, 0.4)	M = 0, ~y = 1.000	1	Table B.3
C_{my}=0.95+0.05α_h	M_h = 0, M_s = 22.06, ~y = 0.000, α_s = 0.000	0.95	Table B.3

Lateral Buckling Check M.b.Rd

Le = 1.00 L	1 x 2.6 =	2.6 m	
M_{cr} = Fn(C_1,L_e,I_z,I_t,I_w,E)	1.127, 2.600, 1551, 22.15, 0.1429, 210000	610.509 kN.m	
λ_{LT} = √ W.f_y/M_{cr}	√ 497.4 x 355 / 610.509	0.538	
c_{LT} = Fn(λ_{LT}, Φ_{LT}, β, λ_{LT}0)	0.538, 0.632, 0.750, 0.400	0.944	Curve b
c_{LT}.mod = Fn(c_{LT},λ_{LT},k_c,f)	0.944, 0.538, 0.942, 0.975	0.969	6.3.2.3
$M_{b.Rd}$ = c $W_{pl.y}$.f_y ≤ $M_{c.y.Rd}$	0.969 x 497.4 x 355 ≤ 176.577 =	171.032 kN.m	

Buckling Resistance

$U_{N.y}$ = N_{Ed}/(c_y.N_{Rk}/γ_{M1})	0.002 / 1942.453	0.000	OK
$U_{N.z}$ = N_{Ed}/(c_z.N_{Rk}/γ_{M1})	0.002 / 1559.628	0.000	OK
$U_{M.y}$ = $M_{y.Ed}$/(c_{LT}.$M_{y.Rk}$/γ_{M1})	22.061 / 171.032	0.129	OK
$U_{M.z}$ = $M_{z.Ed}$/($M_{z.Rk}$/γ_{M1})	0 / 81.97	0.000	OK
$k_yy=C_{my}\{1+(\lambda_y-0.2)U_{N.y}\}$		0.950	
$k_zz=C_{mz}\{1+(2\lambda_z-0.6)U_{N.z}\}$		1.000	
$k_yz=0.6\ k_zz$		0.600	
$k_zy= 1- \{0.1/(C_{mLT}-0.25)\}U_{N.z}$		1.000	
$U_{Ny}+k_yy.U_{M.y}+k_yz.U_{M.z}$	0.000+0.950x0.129+0.600x0.000	0.123	OK
$U_{Nz}+k_zy.U_{M.y}+k_zz.U_{M.z}$	0.000+1.000x0.129+1.000x0.000	0.129	OK

Deflection Check - Load Case 2

In-span δ ≤ Span/360	1.17 ≤ 2600 / 360	1.17 mm	OK

Provide Section :- **203x203 UC 46 (Gr.S355)**
Provide :- **100mm bearing onto 215mm long x 215 wide 3 course Eng. Bwk Padstone**

Rear Knock Through Beam (Span = 3.1m)

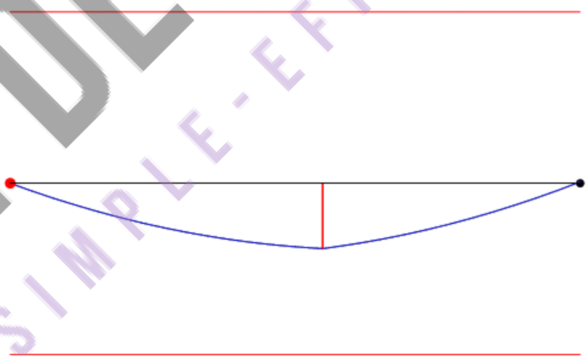
Loading:-			**Span =**	**m**
Dead Loads	**Load (kN/m²)**	**load-span / height (m)**	**Number**	**UDL (kN/m)**
Wall Load	4.75	2.5	1	11.88
Floor	0.6	3.6	1	1.08
Roof	1.41	7.8	1	5.50
Total Dead				**18.5 kN /m**
Live Loads	**Load**	**Span/height**	**Number**	**UDL**
Floor	2	3.6	1	3.60
Roof	0.75	7.8	1	2.93
Total Live				**6.5 kN /m**

AXIAL WITH MOMENTS (MEMBER)

Member SB L1 Id 1 @ Level 1 in Load Case 1

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

D1 D	077.010	(kN/m³)
D1 UDLY	-018.500	(kN/m)
L1 UDLY	-006.500	(kN/m)



Member Forces in Load Case 1 and Maximum Deflection from Load Case 2										
Member No.	**Node End 1 End 2**	**Axial Force (kN)**	**Torque Moment (kNm)**	**Shear Force (kN)**		**Bending Moment (kNm)**		**Maximum Moment (kNm @ m)**		**Max Def (mm @ m)**
				y-y	**z-z**	**y-y**	**z-z**	**y-y**	**z-z**	
-	2	0.07C	0.00	69.64	0.00	0.00	0.00	67.32	0.00	4.69
	4	0.50C	0.00	-73.85	0.00	-1.44	0.00	@ 1.700	@ 0.000	@ 1.564

Classification and Effective Area (EN 1993: 2006)

Section (46.1 kg/m)	203x203 UC 46 [S 355]		
Class = Fn(b/T,d/t,f_y,N,M_y,M_z)	9.25, 22.33, 355, 0.5, 67.32, 0	(Axial: Non-Slender)	Class 2
Auto Design Load Cases	1		

Shear Capacity Check

$V_{y.Ed}/V_{pl.y.Rd}$	73.848 / 347.898 =	0.212	OK

Local Capacity Check

$V_{y.Ed}/V_{pl.y.Rd}$	24.382 / 347.898 =	0.07	Low Shear
$M_{c.y.Rd} = f_y.W_{pl.y}/ \gamma_{M0}$	355 x 497.4/1	176.577 kN.m	
$N_{pl.Rd} = A_g.f_y/ \gamma_{M0}$	58.73 x 355/1 =	2084.915 kN	
$n = N_{Ed}/N_{pl.Rd}$	0.503 / 2084.915 =	0.000	OK
$W_{pl.N.y} = Fn(W_{pl.y}, A_{vy},)$	497.4, 16.974, 0	497.4 cm³	
$M_{N.y.Rd} = W_{pl.N.y}.f_y/ \gamma_{M0}$	497.4 x 355/1	176.577 kN.m	
$(M_{y.Ed}/M_{N.y.Rd+(M\_(z.Ed}/M_{N.z.Rd})$	$(67.323/176.577)^2+(0)^1=$	0.145	OK

Compression Resistance N.b.Rd

Ley = Ky.Ly	1x3.1 =	3.1	
λy = √A.fy/Ncr	√58.73x355/9857.76	0.46	
$N_{b.y.Rd} = Area.c.f_y/ \gamma_{M1}$	58.73x0.902x355/10/1 =	1879.601 kN	Curve b
Lez = Kz.Lz	1x3.1 =	3.1	
λz = √A.fy/Ncrz	√58.73x355/3345.3	0.789	

$N_{b.z.Rd}$ = Area.c.f_y/ γ_{M1}	58.73x0.669x355/10/1 =	1394.523 kN	Curve c
Let = Kt.Lx	1x3.1 =	3.1	
λT = √A.fy/NcrT	√58.73x355/4674.36	0.668	
$N_{b.T.Rd}$= Area.c.f_y/ γ_{M1}	58.73x0.744x355/10/1 =	1552.177 kN	Curve c

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

C_1= fn(M_1, M_2, M_o, ~y,~m)	0.1, -1.4, 66.1, -0.049, -48.560	1.158	Uniform
C_{mLT}=0.95+0.05α_h(1+2~y)	M_h = -1.36, M_s = 65.49, ~y = -0.049, α_s = -0.021	0.949	Table B.3
C_{mz}=Max(0.6+0.4~y, 0.4)	M = 0, ~y = 1.000	1	Table B.3
C_{my}=0.95+0.05α_h	M_h = -1.44, M_s = 65.49, ~y = 0.000, α_s = -0.022	0.949	Table B.3

Lateral Buckling Check M.b.Rd

Le = 1.00 L	1 x 3.1 =	3.1 m	
M_{cr} = Fn(C_1,L_e,I_z,I_t,I_w,E)	1.158, 3.100, 1551, 22.15, 0.1429, 210000	467.550 kN.m	
λ_{LT} = √ W.f_y/M_{cr}	√ 497.4 x 355 / 467.55	0.615	
c_{LT} = Fn(λ_{LT}, Φ_{LT}, β, λ_{LT}0)	0.615, 0.678, 0.750, 0.400	0.911	Curve b
c_{LT}.mod = Fn(c_{LT},λ_{LT},k_c,f)	0.911, 0.615, 0.929, 0.967	0.942	6.3.2.3
$M_{b.Rd}$ = c $W_{pl.y}$.f_y ≤ $M_{c.y.Rd}$	0.942 x 497.4 x 355 ≤ 176.577 =	166.255 kN.m	

Buckling Resistance

$U_{N.y}$ = N_{Ed}/(c_y.N_{Rk}/γ_{M1})	0.503 / 1879.601	0.000	OK
$U_{N.z}$ = N_{Ed}/(c_z.N_{Rk}/γ_{M1})	0.503 / 1394.523	0.000	OK
$U_{M.y}$ = $M_{y.Ed}$/(c_{LT}.$M_{y.Rk}$/γ_{M1})	67.323 / 166.255	0.405	OK
$U_{M.z}$ = $M_{z.Ed}$/($M_{z.Rk}$/γ_{M1})	0 / 81.97	0.000	OK
k_yy=C_{my}{1+(λ_y-0.2)$U_{N.y}$}		0.949	
k_zz=C_{mz}{1+(2λ_z-0.6)$U_{N.z}$}		1.000	
k_yz=0.6 k_zz		0.600	
k_zy= 1- {0.1/(C_{mLT}-0.25)}$U_{N.z}$		1.000	
U_{Ny}+k_yy.$U_{M.y}$+k_yz.$U_{M.z}$	0.000+0.949x0.405+0.600x0.000	0.385	OK
U_{Nz}+k_zy.$U_{M.y}$+k_zz.$U_{M.z}$	0.000+1.000x0.405+1.000x0.000	0.405	OK

Deflection Check - Load Case 2

In-span δ ≤ Span/360	4.69 ≤ 3100 / 360	4.69 mm	OK

Provide Section :- **203x203 UC 46 (Gr.S355)**
Provide :- **215 bearing onto 440mm long x 215 wide 3 course Eng. Bwk Padstone**

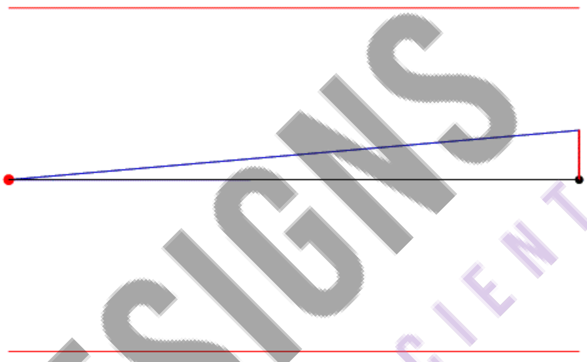
Support Post

AXIAL WITH MOMENTS (MEMBER)

Member SC L1 Id 3 @ Level 1 in Load Case 1

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

D1 D 077.010 (kN/m³)



Member Forces in Load Case 1 and Maximum Deflection from Load Case 2										
Member No.	Node End 1 End 2	Axial Force (kN)	Torque Moment (kNm)	Shear Force (kN)		Bending Moment (kNm)		Maximum Moment (kNm @ m)		Max Def (mm @ m)
				y-y	z-z	y-y	z-z	y-y	z-z	
4	1	74.58C	0.00	-0.60	0.00	0.00	0.00		0.00	0.15
	4	73.85C	0.00	-0.60	0.00	-1.44	0.00		@ 0.000	@ 1.392

Additional Nominal Moments

M_{yUp}	-11.576 kN.m

Classification and Effective Area (EN 1993: 2006)

Section (22.95 kg/m)	152x152 UC 23 [S 275]		
Class = Fn(b/T,d/t,f_y,N,M_y,M_z)	11.19, 21.31, 275, 74.58, 13.01, 0	(Axial: Non-Slender)	Class 3
Effective Properties	Area=29.24 cm², $W_{pl.y}$=179.39(182) cm³, $W_{pl.z}$=76.18(80.2) cm³		
Auto Design Load Cases	1		

Local Capacity Check

$V_{y.Ed}/V_{pl.y.Rd}$	0.598 / 158.276 =	0.004	Low Shear
$M_{c.y.Rd}$ = $f_y.W_{el.y}/\ \gamma_{M0}$	275 x 164.13/1 =	45.136 kN.m	
$V_{z.Ed}/V_{pl.z.Rd}$	0.001 / 328.644 =	0	Low Shear
$M_{c.z.Rd}$ = $f_y.W_{el.z}/\ \gamma_{M0}$	275 x 52.67/1 =	14.484 kN.m	
$N_{pl.Rd}$ = $A_g.f_y/\ \gamma_{M0}$	29.24 x 275/1 =	804.1 kN	
$N_{Ed}/N_{pl.Rd}+M_{y.Ed}/M_{c.y.Rd}+M_{z.Ed}/M_{c.z.Rd}$	74.578 / 804.1 + 13.012 / 45.136 + 0.003 / 14.484 =	0.381	OK

Compression Resistance N.b.Rd

Ley = Ky.Ly	1x2.4 =	2.4	
λy = √A.fy/Ncr	√29.24x275/4500.39	0.423	
$N_{b.y.Rd}$ = Area.c.f_y/ γ_{M1}	29.24x0.917x275/10/1 =	737.329 kN	Curve b
Lez = Kz.Lz	1x2.4 =	2.4	
λz = √A.fy/Ncrz	√29.24x275/1442.2	0.747	
$N_{b.z.Rd}$ = Area.c.f_y/ γ_{M1}	29.24x0.695x275/10/1 =	559.100 kN	Curve c
Let = Kt.Lx	1x2.4 =	2.4	
λT = √A.fy/NcrT	√29.24x275/2012.63	0.632	
$N_{b.T.Rd}$= Area.c.fy/ γ_{M1}	29.24x0.766x275/10/1 =	616.109 kN	Curve c

Equivalent Uniform Moment Factor C1

C_1= fn(M_1, M_2, ~y)	0.0, -13.0, 0.000	1.750	Not Loaded
C_{mLT}=Max(0.6+0.4~y, 0.4)	M = -13.01, ~y = 0.000	0.6	Table B.3
C_{mz}=Max(0.6+0.4~y, 0.4)	M = 0, ~y = 0.000	0.6	Table B.3
C_{my}=Max(0.6+0.4~y, 0.4)	M = -13.01, ~y = 0.000	0.6	Table B.3

Lateral Buckling Check M.b.Rd

Le = 1.00 L	1 x 2.4 =	2.4 m	
M_{cr} = Fn(C_1,L_e,I_z,I_t,I_w,E)	1.750, 2.400, 400.8, 4.635, 0.02118, 210000	223.979 kN.m	
λ_{LT} = √ $W_{el.y}/M_{cr}$	√ 164.1 x 275 / 223.979	0.449	
c_{LT} = Fn(λ_{LT}, Φ_{LT}, β, λ_{LT}0)	0.449, 0.584, 0.750, 0.400	0.981	Curve b
c_{LT}.mod = Fn(c_{LT},λ_{LT},k_c,f)	0.981, 0.449, 0.756, 0.908	1.000	6.3.2.3
$M_{b.Rd}$ = c $W_{el.y}.f_y$ ≤ $M_{c.y.Rd}$	1.000 x 164.1 x 275 ≤ 45.136 =	45.136 kN.m	

Buckling Resistance

$U_{N.y} = N_{Ed}/(c_y.N_{Rk}/\gamma_{M1})$	74.578 / 737.329	0.101	OK
$U_{N.z} = N_{Ed}/(c_z.N_{Rk}/\gamma_{M1})$	74.578 / 559.1	0.133	OK
$U_{M.y} = M_{y.Ed}/(c_{LT}.M_{y.Rk}/\gamma_{M1})$	13.012 / 45.136	0.288	OK
$U_{M.z} = M_{z.Ed}/(M_{z.Rk}/\gamma_{M1})$	0.003 / 14.484	0.000	OK
$k_yy=C_{my}\{1+0.6\lambda_yU_{N.y}\}$		0.615	
$k_zz=C_{mz}\{1+0.6\lambda_zU_{N.z}\}$		0.636	
$k_yz=k_zz$		0.636	
$k_zy=0.8\ k_yy$		0.492	
$U_{Ny}+k_yy.U_{M.y}+k_yz.U_{M.z}$	0.101+0.615x0.288+0.636x0.000	0.279	OK
$U_{Nz}+k_zy.U_{M.y}+k_zz.U_{M.z}$	0.133+0.492x0.288+0.636x0.000	0.275	OK

Deflection Check In-Span - Load Case 2

$\delta \leq$ Span/360	0.15 ≤ 2400 / 360	0.15 mm	OK

Deflection Check Lateral Sway - Load Case 2

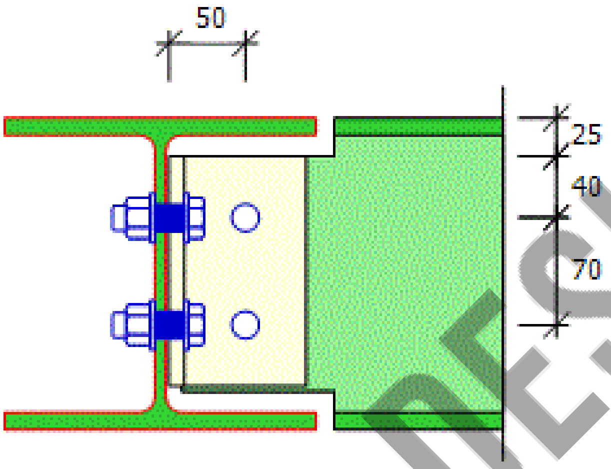
$\delta \leq$ Span/200	0.15 ≤ 2400 / 200	0.15 mm	OK

Provide Section :-   152x152 UC 23 (Gr.S355)

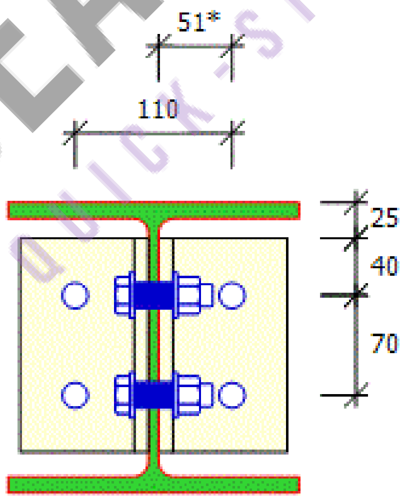
Beam to Beam Connection

BRIEF 2

BEAM TO BEAM: ANGLE CLEAT CONNECTION AT N.3 - MEMBER SB L1 ID 1 – LEVEL 1



Beam: 203x203 UC 46 [S355]
Beam to web gap 0 mm
Web to cleat gap 2 mm
Cleats grade: S355



Right: 203x203 UC 46 [S355]
Top Notch: 25 x 100 mm lg. net
Bot Notch: 25 x 100 mm lg. net
2 No. 90 x 90 x 10 L x 150 mm (2x2 kg)
With 2 No. 18 mm holes in webs
and 2 No. 18 mm holes in toes
For 6 No. 16 mm Ø Grade 8.8 Bolts.

Beam to Beam Angle Cleat Connection to EC 3 (UK NAD)

Basic Data

Integrated Applied Forces at Interface

Shear Forces	Right = 33.9 kN
Tie Force	75.0 kN
Design to	EC 3: Part 1-8: 2005 Design of Connections
SCI Green Book	P358: Joints in steel construction: Simple joints to Eurocode 3

Basic Dimensions

Supporting-203x203UC46 [S 355]	D=203.2, B=203.6, T=11.0, t=7.2, r=10.2, py=355
Right-203x203UC46 [S 355]	D=203.2, B=203.6, T=11.0, t=7.2, r=10.2, py=355
Bolts 16 mm Ø in 18 mm holes	Grade 8.8 Bolts
Plates S 355	All weld grades provided to suit minimum connected steel grade
Cleat Annotation	(Web) - Area of Cleats adjacent to supported beam web (like a fin-plate) (Toe) - Area of Cleats Perpendicular to supported beam web (like an end-plate)

Summary of Results (Unity Ratios)

Right Hand Beam

Checks 8 & 9 Bearing & Shear (Toes)	192.9, 192.9, 192.9, 562.4, 593.2 >= 33.9kN	0.18	OK
Check 10 Shear (Toes)	236.1, 220.2 >= 17.0kN	0.08	OK
Tie checks	0.46, 0.31, 0.71, 0.51	0.71	OK
Additional tie checks	0.75, 0.39	0.75	OK
Check 2: Bolt Shear & Bearing (web)	0.25, 0.13, 0.37	0.37	OK
Check 3: Sup.Beam Connect Elmts. (web)	0.07, 0.06, 0.07, 0.06	0.07	OK
Check 4: Supported Beam Resist.	0.17, 0.19, 0.20	0.20	OK
Check 5: Notch Resistance	0.37	0.37	OK
Check 6: Notch Stability	0.61	0.61	OK

Right Hand Beam

Check 8a: Bolt Shear (Toes)

$F_{v,Rd}=\alpha_v \bullet f_{ub} \bullet A / \gamma_{M2}$	0.6 • 800 • 157.0 / 1.25	60.3 kN	
$V_{Rd}1 = 0.8 \bullet n \bullet F_{v,Rd} \bullet cols$	0.8 • 2 • 60.3 • 2	192.9 kN	OK

Check 8b: End-Plate Bearing (Toes)

P_1 , e_1 , P_2 , e_2 , α_b , k_1	70.0, 40.0, 50.0, 38.6, 0.741, 2.189		
$F_{b,Rd}=k_1 \bullet \alpha_b \bullet f_{up} \bullet d \bullet t_p / \gamma_{M2}$	2.2 • 0.741 • 470 • 16 • 10.0 / 1.25	97.5 kN	
$0.80 \bullet F_{v,Rd} < F_{b,Rd}$			
$F_{Rd} = 0.80 \bullet n \bullet f_{v,Rd} \bullet rows$	0.80 • 2 • 60.3 • 2	192.9 kN	OK

Check 8c: Supporting Beam Web Bearing (Toes)

$\alpha_b=Min(p_1/d_0 /3-¼, f_{ub} / f_u , 1)$	Min(70/18/3-¼, 800/470, 1) » Min(1.046, 1.702, 1)	1.000	
$k_{1\ .inn}=min(2.5, 1.4 \bullet p_2 /d_0-1.7)$	min(2.5, 1.4 • 50/18 - 1.7 » Min(2.5, 2.2)	2.189	
$F_{b,Rd}=k_1 \bullet \alpha_b \bullet f_{u,2} \bullet d \bullet t_p / \gamma_{M2}$	2.2 • 1.000 • 470 • 16.0 • 7.2 / 1.25	94.8 kN	
$0.80 \bullet F_{v,Rd} < F_{b,Rd}$			
$F_{Rd} = 0.80 \bullet n \bullet f_{v,Rd} \bullet rows$	0.80 • 2 • 60.3 • 2	192.9 kN	OK

Check 9a: End-Plate Plane Shear (Toes)

$V_{Rd4} = 2 \bullet h_p \bullet t_p \bullet F_{yp} /(1.27 \bullet \sqrt{3} \bullet \gamma_{M0})$	2•150 • 10 • 355/(1.27 • √3 • 1.0)	484.2 kN	OK
$V_{Rd} = 2 \bullet t_p \bullet (h_p-n_1 \bullet d_0) \bullet F_{up} /(\sqrt{3} \bullet \gamma_{M2})$	2•10•(150 - 2 • 18) • 470/(√3 • 1.1)	562.4 kN	OK

Check 9b: End-Plate Block Shear (Toes)

A_{nv} , $A_{nt} = Fn(t_p , h_p , e_1 , e_2 , n_1 , d_0)$	(10, 150, 40, 39, 2, 18)	830, 296 mm²	
$V_{Rd}=2 \bullet (F_{up} \bullet A_{nt} /\gamma_{M2}+f_{yp} \bullet A_{nv} /(\sqrt{3} \bullet \gamma_{M0}))$	2•(470 • 296 / 1.1 + 355830 • /(√3 • 1.0)	593.2 kN	OK

Check 10: Supporting Member - Local Resistance (Toes)

A_v , $A_v net=Fn(e_t , e_b , p_1 , n, dia, T)$	fn(65, 25, 70, 2, 18.0, 7.2)	1152, 893 mm²	
$V_{Rd} = A_v \bullet F_{yp} /(\sqrt{3} \bullet \gamma_{M0})$	1152 • 355/(√3 • 1.0)	236.1 kN	OK
$V_{Rd}=A_v net \bullet F_{up} /(\sqrt{3} \bullet \gamma_{M2})$	893 • 470/(√3 • 1.1)	220.2 kN	OK

Tie force Checks

Basic Data	Tie force = 75.0 kN		
Tie Force	Tie forces are analysed independently of any vertical loads.		
	The tie force load should not be greater than the vertical shear load		Caution
EC3 Design	Using conservative SCI P212 method to BS 5950		
New Tie force	Magnified by 1.2 for lower EC3 load Factors	90.0 kN	
Cleat toe Le=e1+e2+(n-1)•pe-n•d'	39 + 39 + (2-1) x 70 - 2 x 18.0	111.2 mm	
Cleat Ten cap =0.50•Le•t•py	0.50 x 111.2 x 10 x 355	197.4 kN	OK

Beam Web Le=e1+e2+(n-1)•pe-n•d'	40 + 40 + (2-1) x 70 - 2 x 18.0	114.0 mm	
Web Ten cap=Le•t•py	114.0 x 7 x 355	291.4 kN	OK
Web Bear cap=Fn(t, Pbg, e, n, Ø, kbs)	7.2, 550, 40, 2, 16, 1.00	126.7 kN	OK
Cleat Bear cap=Fn(t, Pbg, e, n, Ø, kbs)	10.0, 550, 40, 2, 16, 1.00	176.0 kN	OK
Prying Ratio Pr=(l1+2•tk) / (2•tk)	(9 + 2 x 10) / (2 x 10)	1.47	
Bolt Cap=Us•Anet/(1.25•Pr)•bolts	785 x 157/(1.25 x 1.47) x 4	267.4 kN	OK
Additional checks for high tie force	Tie force is greater than the vertical shear load.		
Shear per Bolt Fv=Ft/n	90 / 2	45.0 kN	
Bolt Shear Ps=	16 mm Ø Grade 8.8 Bolts	60.3 kN > fvt	OK
Bolt Bearing Pbb=pb•tmin•Ø	1000 x 7.2 x 16	115.2 kN > Frt	OK

Check 2: Supported beam - Bolt Group (web)

i) Bolts in Shear

$\beta=(6\bullet z) / (n_1\bullet(n_1+1)\bullet p1)$	(6•50) / (2 • (2 + 1)• 70)	0.714	
$F_{v,Rd}=\alpha_v \bullet f_{ub} \bullet A / \gamma_{M2}$	0.6 • 800 • 157.0 / 1.25	60.3 kN	
$V_{Rd}=2\bullet n\bullet F_{v.Rd} / \sqrt{((1+a\bullet n)^2+(\beta\bullet n)^2)}$	$(2\bullet 2 \bullet 60.29) / \sqrt{((1+0.000\bullet 2)^2 + (0.714\bullet 2)^2)}$	138.3 kN	OK

ii) Fin plate bearing

Vert P_1 , e_1 , P_2 , e_2 , α_b , k_1	70.0, 40.0, 0.0, 40.0, 0.741, 2.500		
$F_{b,Rd}=k_1 \bullet \alpha_b \bullet f_{up} \bullet d\bullet t_p / \gamma_{M2}$	2.5 • 0.741 • 470 • 16 • 10.0 / 1.25	111.4 kN	
Horz P_1 , e_1 , P_2 , e_2 , α_b , k_1	0.0, 40.0, 70.0, 40.0, 0.741, 2.500		
$F_{b,Rd}=k_1 \bullet \alpha_b \bullet f_{up} \bullet d\bullet t_p / \gamma_{M2}$	2.5 • 0.741 • 470 • 16 • 10.0 / 1.25	111.4 kN	
$V_{.Rd}=2\bullet N / \sqrt{(}\ ((1+a\bullet N) / F_{b.vert.Rd})^2$	$2\bullet 2.00 / \sqrt{(((1 + 0.00 \bullet 2.00) / 111.41)^2}$		
$+(\beta\bullet N / F_{b.hor.Rd})^2)$	$+ (0.71 \bullet 2.00) / 111.41)^2)$	255.6 kN	OK

iii) Beam Web bearing

Vert P_1 , e_1 , P_2 , e_2 , α_b , k_1	70.0, 40.0, 0.0, 40.0, 0.741, 2.500		
$F_{b,Rd}=k_1 \bullet \alpha_b \bullet f_{ub} \bullet d\bullet t_p / \gamma_{M2}$	2.5 • 0.741 • 470 • 16 • 7.2 / 1.25	80.2 kN	
Horz P_1 , e_1 , P_2 , e_2 , α_b , k_1	0.0, 40.0, 70.0, 40.0, 0.741, 2.500		
$F_{b,Rd}=k_1 \bullet \alpha_b \bullet f_{ub} \bullet d\bullet t_p / \gamma_{M2}$	2.5 • 0.741 • 470 • 16 • 7.2 / 1.25	80.2 kN	
$V_{.Rd}=N / \sqrt{(((1+a\bullet N) / F_{b.vert.Rd})^2}$	$2.00 / \sqrt{(((1 + 0.00 \bullet 2.00) / 80.21)^2}$		
$+(\beta\bullet N / F_{b.hor.Rd})^2)$	$+ (0.71 \bullet 2.00) / 80.21)^2)$	92.0 kN	OK

Check 3:Supported Beam - Connecting Elements (web)

i) Shear

$V_{Rd.g}=2\bullet(h_p \bullet t_p / 1.27)\bullet(f_{yp} / \sqrt{3}/\gamma_{M0})$	$2\bullet(150\bullet 10/1.27) \bullet (355/\sqrt{3}/1.0)$	484.2 kN	OK
$V_{Rd.n}=2\bullet A_{v.Net} \bullet f_{up} /\sqrt{3}/\gamma_{M2net}$	$2\bullet 1140 \bullet 470/\sqrt{3}/1.1$	562.4 kN	OK
$V_{Rd.b}=2\bullet(0.5\bullet f_{up} \bullet A_{nt} /\gamma_{M2net})$	2•(0.5 • 470 • 310 / 1.1)		
$+2\bullet(F_{yp} \bullet A_{nv} /\sqrt{3}/\gamma_{M0})$	$+ 2\bullet(355 \bullet 830 / \sqrt{3}/1.0)$	472.7 kN	OK

ii) Bending

$V_{Rd}=8$	$H_p >= 2.73\bullet z$	n.a.	OK

iii) Lateral torsional buckling

$V_{Rd}=2\bullet(W_{el.P} / z)\bullet(F_{yP} / \gamma_{M0})$	2•(37500 / 50) • (355 / 1.0)	532.5 kN	OK

Check 4: Supported Beam - Resistance at the connection (web)

i) Shear

$V_{Rd.G}=A_{v.wb}\bullet f_{y.b1} / \sqrt{3}/ \gamma_{M0}$	$992.74 \bullet 355.00/\sqrt{3}/1.0$	203.5 kN	OK
$V_{Rd.N}=A_{v.wb.net}\bullet f_{u.b1} / \sqrt{3} / \gamma_{M2}$	$733.54 \bullet 470.00/ \sqrt{3}/1.1$	181.0 kN	OK
$V_{Rd.B}=(0.5\bullet f_{u.b1}\bullet A_Nt / \gamma_{M2})$	((0.5 • 470.00 • 223.20) / 1.1)		
$+ (f_{y.b1}\bullet A_NV / \sqrt{3} / \gamma_{M0})$	+ ((355.00 • 597.60 / (1.73 • 1.0))	170.2 kN	OK

Check 5: Supported Beam Notch Resistance

$V_{Rd}=0.9\bullet A_v.DN\bullet f_{ybw} /(\sqrt{3}\bullet\gamma_{M0})$	$0.9 \bullet 1103 \bullet 355 / (\sqrt{3} \bullet 1.0)$	203.5 kN	Low Shear
$M_{v.Rd}=f_{y.b1} \bullet W_{el.N.y} / \gamma_{M0}$	355 • 28/1.0	10.0 kN.m	
$V_{Rd}=Min(V_{Rd} , M_{v.Rd} / La)$	Min(203.47, 10.00 / 110)	90.9 kN	OK

Check 6: Supported Beam - Double Notched Web Stability (restrained)

$D_{nt} <= h_{b1} / 5$	25.0 <= 203.2/ 5 mm	25.0 <= 40.6	OK
$L_n <= h_{b1}$		100 <= 203	OK

Check 7: Unrestrained Supported Beam. Overall Stability of Notched Beam

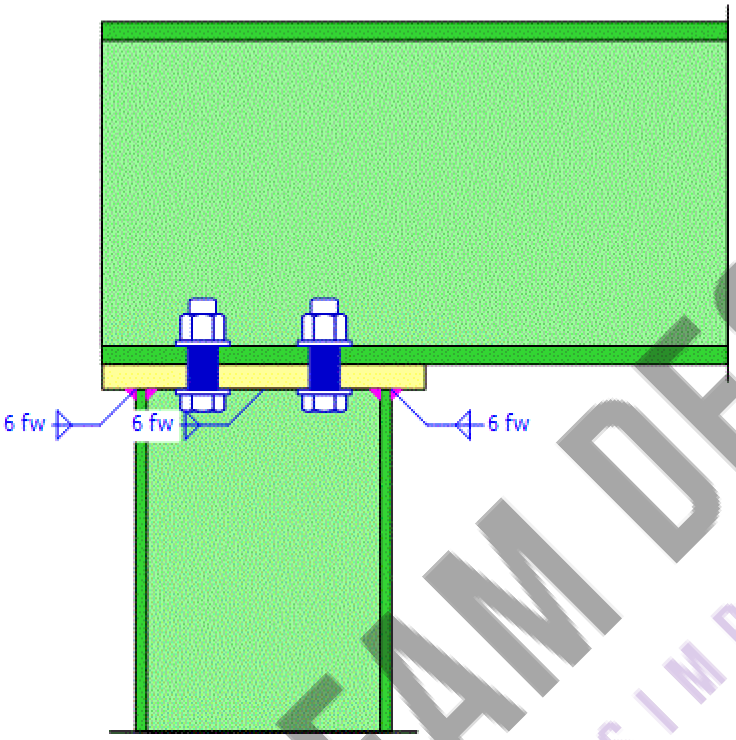
Beam assumed to be restrained — ignored

Provide Section :- 120x120x10mm Thick Steel Angle Cleat with M16 Gr.8.8 bolts as shown.

Beam To Column Cap Plate

BRIEF 3

COLUMN TO BEAM FLANGE CONNECTION AT : END 2 OF MEMBER SC L1 ID 3 - LEVEL 1



Plates S 355
Column 152x152 UC 23 [S275]
Beam 203x203 UC 46 [S355]
Left 20 aboveleft flange

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

LOADING CASE 001 : DEAD PLUS LIVE (ULTIMATE)

Basic Data

Integrated Applied Forces at Beam/Bottom Column Interface

Resultant Forces M, Fv, F	15.4 kNm, -6.7 kN, 78.6 kN	Load directionsLeft of Connection in Tension, Column moving Left and in Compression.
Design to	EC 3: Part 1-8: 2005 Design of Connections SCI Green Book P398: Joints in steel construction: Moment-Resisting Joints to Eurocode 3	
Weld Grades	All weld grades provided to suit minimum connected steel grade	

Basic Dimensions

Beam-203x203UC46 [S 355]	D=203.2, B=203.6, T=11.0, t=7.2, r=10.2, py=355		
Column-152x152UC23 [S 275]	D=152.4, B=152.2, T=6.8, t=5.8, r=7.6, py=275		
Bolts 16 mm Ø in 18 mm holes	Grade 8.8 Bolts		
Plates S 355	All weld grades provided to suit minimum connected steel grade		
Beam Forces M, Fv, F	1 x 16 = 16 kN.m, 79 kN, 7 kN		
Beam Capacities Mc, Fvc, Fc	176.6 kN.m, 269.5 kN, 2084.9 kN	Fvc = 269.5 kN	OK

Summary of Results (Unity Ratios)

Moment Capacity 16.0 kNm (for 1 rows of bolts) (Modified Applied Mom. M_{mod} =9.7 kNm)		0.61
Shear Capacity		0.04
Flange Welds	0.27	0.27
Web Welds	0.27, 0.18	0.27

Step 1: Tension Zone

Basics

Bm/Plt b_p , W, t_{wb} , S_w , m_p , e_p , n_p	200.0, 90, 5.8, 6, 37.3, 55.0, 46.6		
Beam B_c , t_wc , r_c , m_c , e_c , n_c	203.6, 7.2, 10.2, 33.2, 56.8, 41.6		
$F_{t,Rd}=k_2 \bullet f_{ub} \bullet A / \gamma_{m^2}$	0.90•800•157/1.25	90.4 kN	
$F_{T,3,Rd}=\Sigma F_{t,Rd}$	2•90.4	180.9 kN	

BOLT ROW 1

Beam Flange row 1 only

m, e, e_x	33.2, 56.8, 60.0		
$l_{eff,cp}$=min(Circle, Indiv End)	min(208.9, 224.4)	208.9 mm	T2.2 (b)
$l_{eff,ncp}$=min(Corner2 (b), Side (e))	min(162.0, 204.0)	162.0 mm	T2.2 (b)
Mode 1 $l_{eff,1}$=min($l_{eff,cp}$, $l_{eff,cnp}$)	min(208.9, 162.0)	162.0 mm	
Mode 2 $l_{eff,2}=l_{eff,ncp}$	162.0	162.0 mm	
$M_{pl,1} = l_{eff,1} \bullet t_f^2 \bullet p_y /4/\gamma_{M0}$	162.0•11.0•11.0•355.0/4/1.0	1739.5 kN.mm	
$M_{pl,2} = l_{eff,2} \bullet t_f^2 \bullet p_y /4/\gamma_{M0}$	162.0•11.0•11.0•355.0/4/1.0	1739.5 kN.mm	
$2 \bullet m \bullet n - e_w \bullet (m+n)$	2•33.24•41.55-7.50•(33.24+41.55)	2201.3	
$F_{T,1,Rd}=(8 \bullet n-2 \bullet e_w) \bullet M_{pl,1} /2201.3$	(8•41.55-2•7.50)•1739.5/2201.3	250.8 kN	
$F_{T,2,Rd}=(2 \bullet M_{pl,2}+n \bullet Nb \bullet F_{t,Rd})/(m+n)$	(2•1739.5 + 41.55•2•90.4)/(33.24 + 41.55)	147.0 kN	
$F_{t,Rd}$ =min($F_{t,Rd}$ mode1,2,3)	min(250.8, 147.0, 180.9)	147.0 kN	

Beam Web Tension row 1 only

ω=fn(b_{eff}, t_w, A_{vc}, β)	fn(162.0, 7.2, 1697, 1.00)	0.79	
$F_{t,wc,Rd} = \omega \bullet l_{eff,1} \bullet t_{wc} \bullet f_{y,wc}/\gamma_{M0}$	0.79•162.0•7.2•355/1.00	325.9 kN	

End Plate row 1 only

m, e, mx, α	37.3, 55.0, 28.4, 6.8		
l_{eff} modes	$l_{eff,cp}$ = 234.4, $l_{eff,ncp}$ = 254.0		T2.2 (c)
Mode 1 $l_{eff,1}$=min($l_{eff,cp}$, $l_{eff,cnp}$)	min(234.4, 254.0)	234.4 mm	
Mode 2 $l_{eff,2}=l_{eff,ncp}$	254.0	254.0 mm	
$M_{pl,1} = l_{eff,1} \bullet t_f^2 \bullet p_y /4/\gamma_{M0}$	234.4•15.0•15.0•355.0/4/1.0	4679.9 kN.mm	
$M_{pl,2} = l_{eff,2} \bullet t_f^2 \bullet p_y /4/\gamma_{M0}$	254.0•15.0•15.0•355.0/4/1.0	5071.7 kN.mm	
$2 \bullet m \bullet n - e_w \bullet (m+n)$	2•37.30•46.63-6.50•(37.30+46.63)	2932.7	
$F_{T,1,Rd}=(8 \bullet n-2 \bullet e_w) \bullet M_{pl,1} /2932.7$	(8•46.63-2•6.50)•4679.9/2932.7	574.5 kN	
$F_{T,2,Rd}=(2 \bullet M_{pl,2}+n \bullet Nb \bullet F_{t,Rd})/(m+n)$	(2•5071.7 + 46.63•2•90.4)/(37.3 + 46.63)	221.3 kN	
$F_{t,Rd}$ =min($F_{t,Rd}$ mode1,2,3)	min(574.5, 221.3, 180.9)	180.9 kN	

Column Web Tension row 1 only

$F_{t,wb,Rd} = l_{eff,1} \bullet t_{wb} \bullet f_{y,wb}/\gamma_{M0}$	234.4•5.8•275/1.00	373.8 kN	
Potential resistance of Bolt Row 1	$F_{t1,Rd}$	147.0 kN	Mode 2

Step 1C Plastic distribution Limit

$T_p < d/1.9 \bullet \sqrt{(f_{ub} /f_{yp})}$	15.0 < 16/1.9•√(800.0/355)	15.0>12.6	Elastic
$T_{fc} < d/1.9 \bullet \sqrt{(f_{ub} /f_{y,fc})}$	11.0 < 16/1.9•√(800.0/355)	11.0 <= 12.6	Plastic
$F_{t\ 1,Rd} < 1.9\ F_{t,rd}$	147.0 < 1.9•90.4		Plastic
Classification	Plastic Deformation occurs.	Use Plastic distribution	

Potential Tension Capacity

Sigma $F_{ti,Rd}$	147.0 kN	147.0 kN

Step 2: Compression Zone

Web Bearing

n =min(5,2+0.6•B_e /(t_{fc}+s))	min(5, 2 + 0.6•152.4/21.2)	5.000
$b_{eff,c}=t_{fb}+2s_f+n(t_{fc}+s)+s_p)$	6.80+2•6.0+5.000(11.00+10.20)+29.00	153.8
ω=fn(b_eff, t_w, A_{vc}, β)	fn(153.8, 7.2, 1697, 1.00)	0.80
$\sigma_{com,Ed}=M_ed/W_el+N_ed/A < fy$	16.0•1000/449.9+6.6•10/58.7	36.7 N/mm²
k_wc	$\sigma_{com,Ed} < 0.7 \bullet f_y$	1.00
$F_{c,wc,Rd} = \omega \bullet k_{wc} \bullet b_{eff,c} \bullet t_{wc} \bullet f_y/\gamma_{M0}$	0.802•1.000•153.8•7.2•355/1.00	315.4 kN

Web Buckling

$\lambda_p=0.932\sqrt{}(b_{eff,c} \bullet d_{wc} \bullet f_y/(E \bullet t_{wc}^2)$	$0.932\sqrt{}153.8 \bullet 160.8 \bullet 355/(210000 \bullet 7.20^2)$	0.837
Buckling factor ~r	$\lambda_p > 0.72$, $\sim r=(\lambda_p-0.2)/\lambda_p^2$	0.91
$F_{c,wc,Rd} = \sim r \bullet \omega \bullet k_{wc} \bullet b_{eff,c} \bullet t_{wc} \bullet f_y/\gamma_{M1}$	0.91•0.80•1.00•153.8•7.2•355/1.00	286.8 kN

Column Compression

$F_{c,fb,Rd}= M_{c,Rd}/(h_b-t_{fb})$	45.1•1000/(145.60)	310.0 kN

Step 3: Beam Web Shear

$V_{wp,Rd}=0.9•f_{yc}•A_{vc}/(\gamma_{M0}•\sqrt{3})$	0.9•355•1697.4/(1.0•√3)	313.1 kN	

Potential Compression Capacity

$F_{c,Rd}$min	Min(315.4, 286.8, 310.0)	286.8 kN	OK

Step 4: Moment Capacity

$F_{c,Rd}=\mathrm{Min}(F_{c,wc,Rd}, F_{c,fc,Rd})$	min(286.8, 310.0)	286.8 kN	
Shear limit $F_{ri}=\mathrm{min}(F_{t,Rd}$ Total, $V_{wp,Rd})$	min(147.0, 313.1)	147.0 kN	
$F_{t,Rd}$ Total $<$ Min($F_{c,Rd}$ $-N_{Ed}$, $V_{wp,Rd}$)	No reduction in bolt forces required		

Final Bolt Forces and Moment Capacities

Bolt row 1 $M_{c,Rd}1=F_{t1,Rd}•h_1$	147.0•109.0	16.0 kN.m	
$M_{c,Rd}$		16.0 kN.m	
$M_{mod,Ed}=M-N_{Ed}•h_n$	15.4 - 78.6•72.8	9.7 kN.m	OK
$F_{ri,Ed}$ for 1 rows	147.0	147.0 kN	
$F_{ri,Ed}$ design$=F_{ri,Ed}•M_{Ed}/M_{c,Rd}'$	147.0•9.7/16.0	88.9 kN	

Step 5: Shear Bolts

$F_{v,Rd}=\alpha_v•f_{ub}•A/\gamma_{M2}$	0.6 • 800 • 157.0 / 1.25	60.3 kN	
Bearing $F_{b,Rd,End}$-End Plate, End	e1=60, e2=55, k_1=2.5, α_b=1.00, d=16, t=15, f_u=470	225.6 kN	
Bearing $F_{b,Rd}$-End Plate, Inner	p1=72, e2=55, k_1=2.5, α_b=1.00, d=16, t=15, f_u=470	225.6 kN	
Bearing $F_{b,Rd,End}$-Col Flange, End	e1=60, e2=57, k_1=2.5, α_b=1.00, d=16, t=11, f_u=470	165.4 kN	
Bearing $F_{b,Rd}$-Beam Flange, Inner	p1=72, e2=57, k_1=2.5, α_b=1.00, d=16, t=11, f_u=470	165.4 kN	
$F_{v,Rd,Sh}$=Min(bearing, shear)	Min(225.6, 165.4, 60.3)	60.3 kN	
$F_{v,Rd,T}=\mathrm{min}(F_{b,Rd},0.28•\mathrm{shr})$	Min(225.6, 165.4, 16.9)	16.9 kN	
$F_{v,Rd,TEnd}=\mathrm{min}(F_{b,Rd,End},0.28•\mathrm{shr})$	Min(225.6, 165.4, 16.9)	16.9 kN	
Shear=$Ns•F_{v,Rd,Sh}$	2•60.3	120.6 kN	
Tension= $(Nt-1)•F_{v,Rd,T}+F_{v,Rd,TEnd}$	0•16.9 + 2•16.9	33.8 kN	
$F_{v,Rd}$ Total=Shear+Tension	120.6 + 33.8	154 kN	OK

Step 7: Welds

Column $f_{vw,d}=f_u/(\sqrt{3}•\beta_w)/\gamma_{M2}$	410.0 / √3 / 0.85 / 1.25	222.8 N/mm²	

Flange Tension Weld

$Ft_{,,flng}=\mathrm{min}(B•T•P_y, F_{r1})$	Min(152.2•6.8•275, 88.9)	88.9 kN	
$F_{vw,Rd}=K•0.7•t_s•L•f_{vw,d}$	1.225•0.7•6•(2•(152.2-2•0.707•6))•223	332.8 kN	OK

Flange Compression Weld

Direct Bearing assumed. No check required

Web Welds in Tension Zone

Lwt=L-proj-T-root+1.73•g/2	60 - 20 - 6.8 - 7.6 + 1.73 90/2	103.5 mm	
Load per row			
$Row_1=K_1•F_{r1}$	(28/(37 + 28))•147	63.5 kN	
Total Load F_t	63.5	63.5 kN	
$F_{wCap}=2•K•0.7•ts•Lwt•f_{vw,d}$	2•1.225•0.7•6•103.5•223	239.5 kN	OK

Web Welds in Shear Zone

$Lws=D-(T_t+T_b)-r_t-r_b-Lwt$	152.4 - 13.6 - 7.6 - 7.6 - 103	20.1 mm	
$F_{wCap}=2•0.7•ts•Lws•f_{vw,d}$	2•0.7•6•20.1•223	38.1 kN	OK

Provide Section :- 15mm Thick Cap Plate and 4No. M16 Gr.8.8 Bolts.

Foundation Design/Checks

Axial Load = 75kN @ ULS = 54kN @ SLS

Pier Foundation Design Check

Presumming a 450mm deep x 450mm wide brick corbel (Worst Case)

Area at Founding = (0.45m + 2 x 0.45mm) x 0.45m = **$0.61m^2$**

Bearing Stress = 54kN / $0.61m^2$ = $88.5kN/m^2$ $<100kN/m^2$

Existing Foundation is Adequate

Column Foundation Design

Pressuming an allowable bearing capacity of $100kN/m^2$

Area Req. = 54kN / $100kN/m^2$ = $0.54kN/m^2$

Provide Section :- **900mm Sq. Gen 3 Concrete Pad Foundation (Min 450m thick)
Final depth to be confirmed onsite with LABC.**