STRUCTURAL CALCULATIONS

Using
MASTERSERIES POWERPAD

House Extension

Address 1
Address 2
Address 3

ARCHITECT

ARCHITECT NAME

Location
Location
Location

STRUCTURAL ENGINEERS

MASTERSERIES USER COMPANY

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Fax: (028) 9036 5102

PROJECT NO: P12345

December 2017

V.A.T. Reg. No. ___ ____ __

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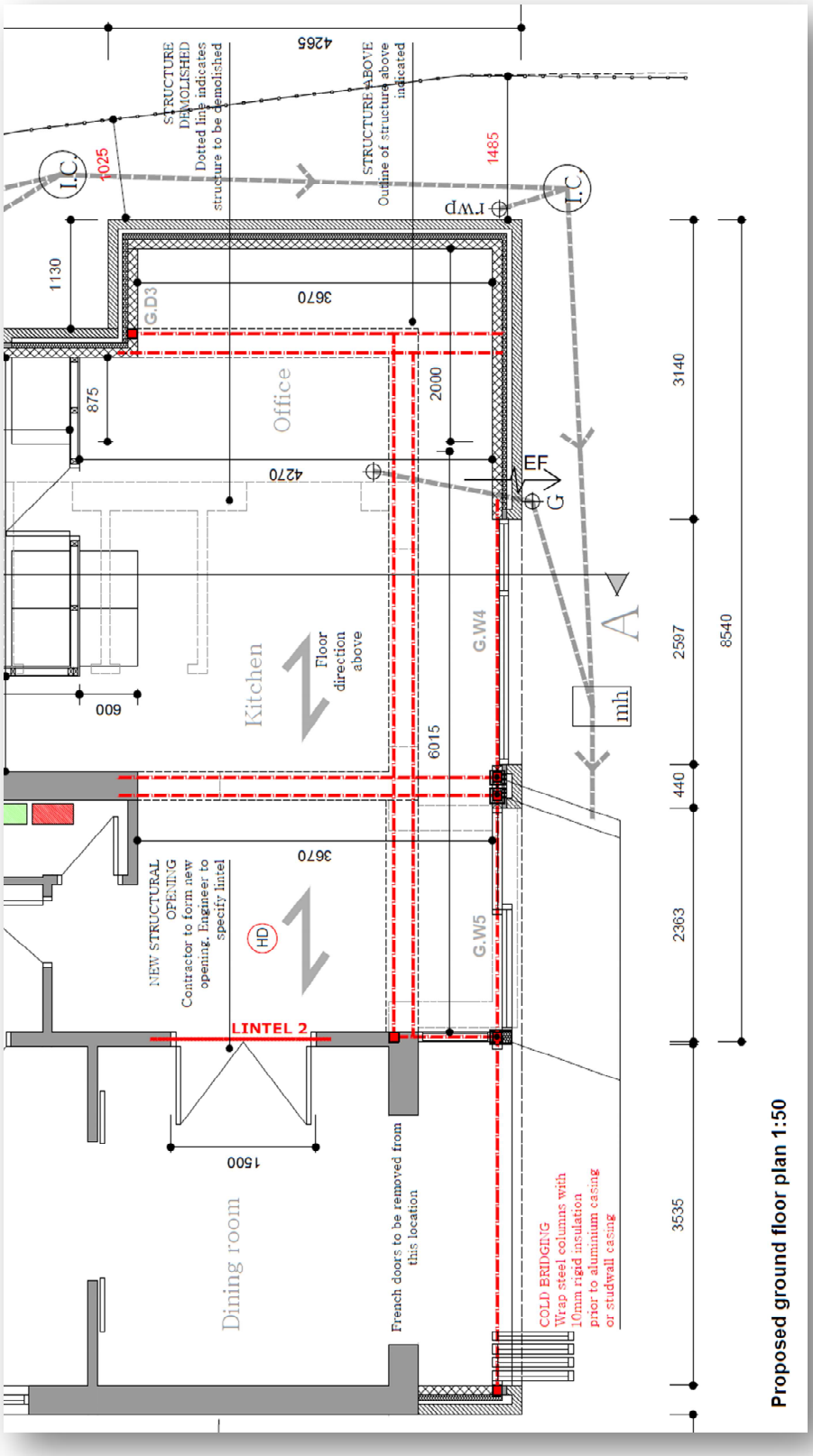
HOUSE EXTENSION FIRST FLOOR LAYOUT



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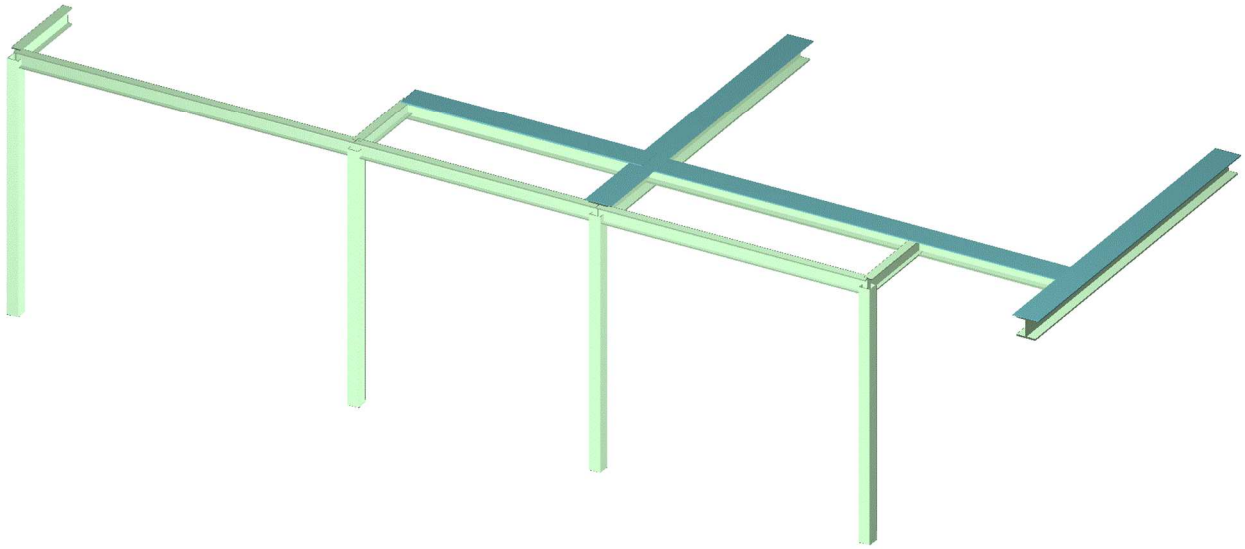

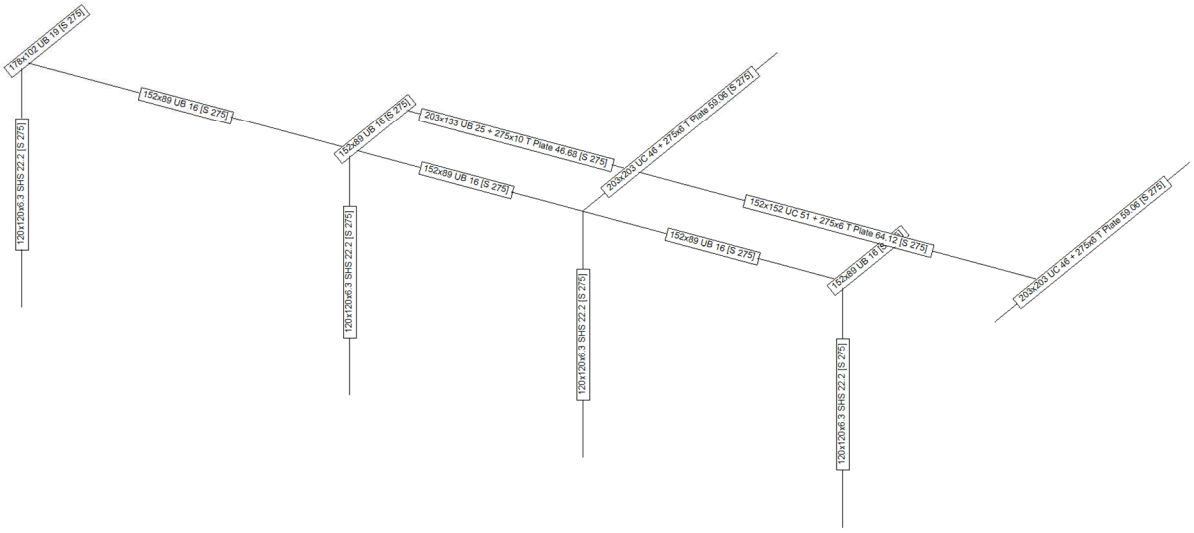

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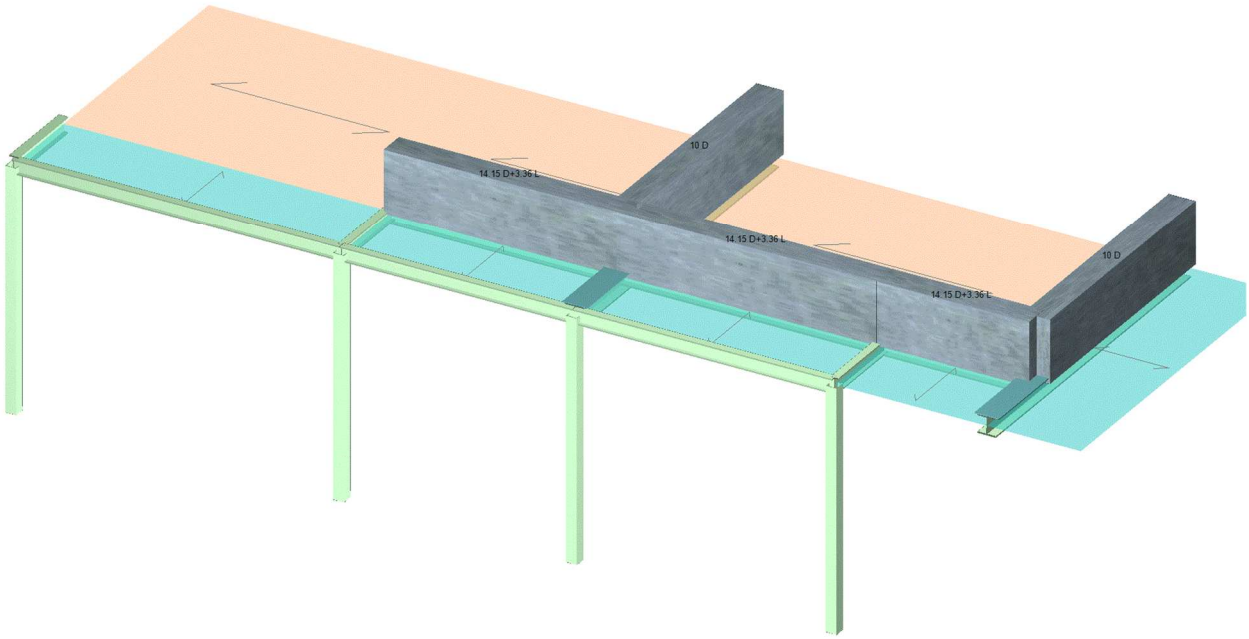
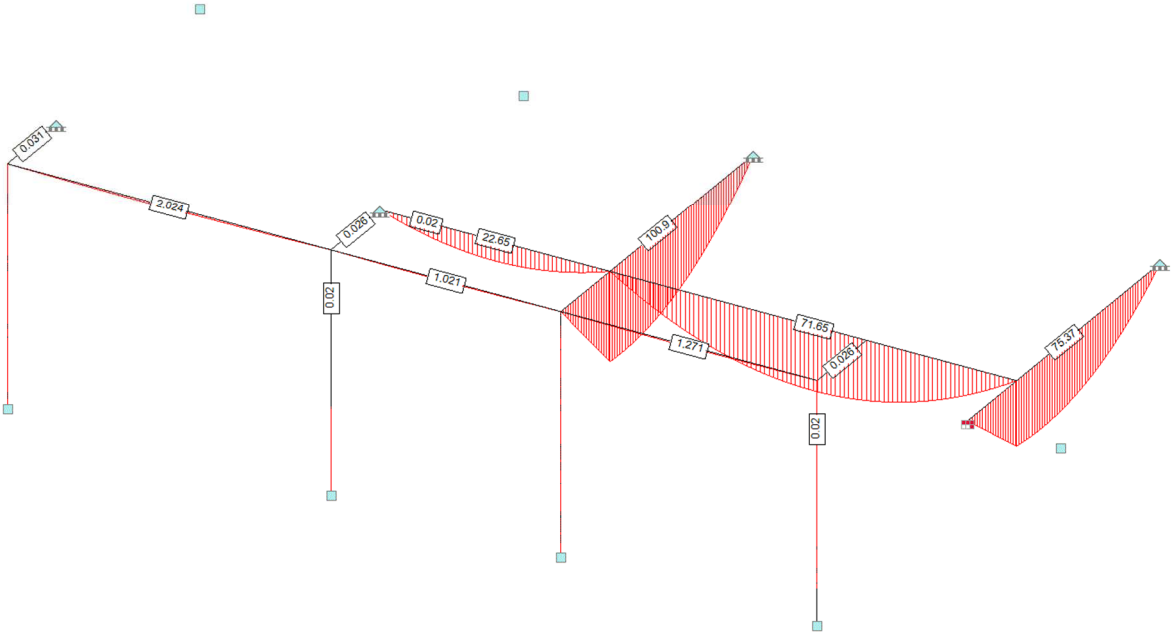


Loading



Load Case 001 : Dead plus Live (Ultimate)
Bending Moment Diagram (Major Axis)

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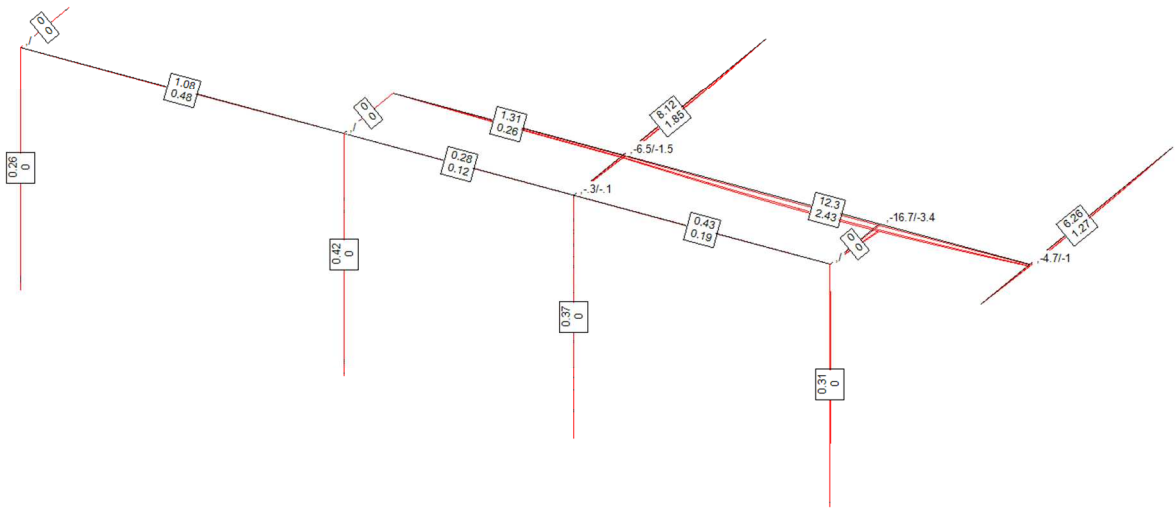

Envelope (Serviceability Cases)
Deflected Shape - (Full Frame)
Maximum and Minimum Nodal and Member Deflection (mm)
5 Magnification

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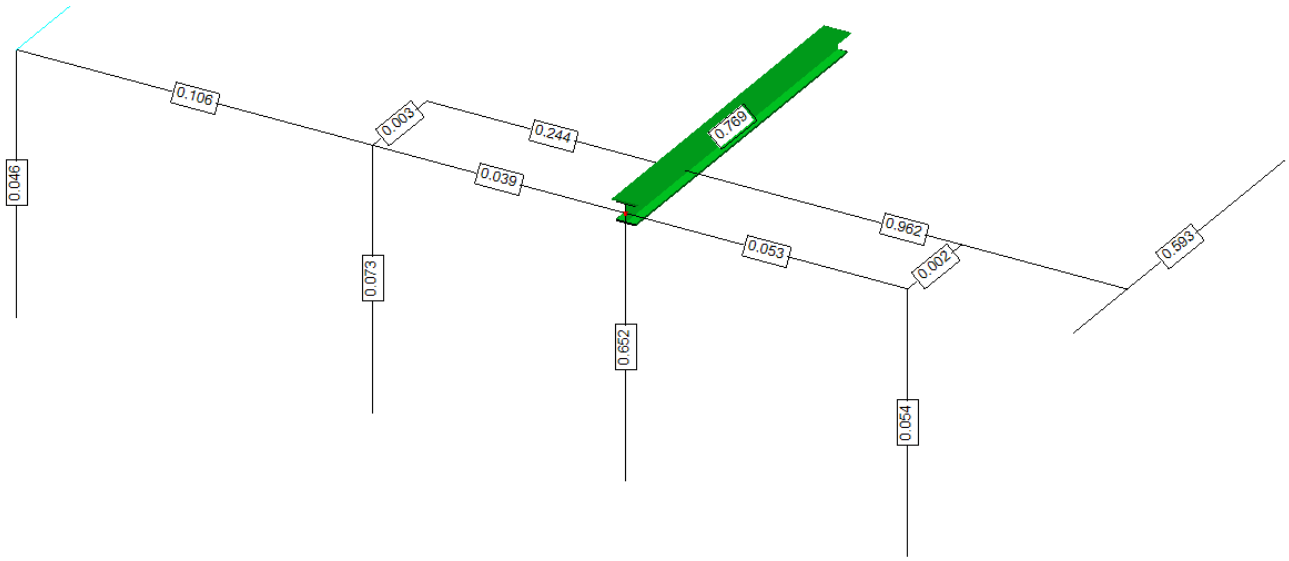
STEEL DESIGN

Unity Valuers



Unity Valies

List of Checks - Results Summary			
M SBL1Id 17	00.000 to 00.970 m, in Case 001	0.769	A+M Pass
	00.970 to 03.800 m, in Case 001	**0.769**	**A+M Pass**
M SBL1Id 18	**00.000 to 03.800 m, in Case 001**	**0.593**	**A+M Pass**
M SBL1Id 12	**00.000 to 04.600 m, in Case 001**	**0.962**	**A+M Pass**
M SBL1Id 11	00.000 to 02.600 m, in Case 001	0.244	A+M Pass
M SBL1Id 1	00.000 to 03.660 m, in Case 001	0.106	A+M Pass
M SBL1Id 20	00.000 to 00.970 m, in Case 005	0.002	S-T Pass
M SCL1Id 23	00.000 to 02.700 m, in Case 001	0.652	A+M Pass
M SBL1Id 2	00.000 to 02.600 m, in Case 001	0.039	A+M Pass
M SBL1Id 3	00.000 to 02.900 m, in Case 001	0.053	A+M Pass
M SBL1Id 15	00.000 to 00.970 m, in Case 005	0.003	S-T Pass
M SCL1Id 21	00.000 to 02.700 m, in Case 005	0.046	A+M Pass
M SCL1Id 22	00.000 to 02.700 m, in Case 005	0.073	A+M Pass
M SCL1Id 24	00.000 to 02.700 m, in Case 005	0.054	A+M Pass

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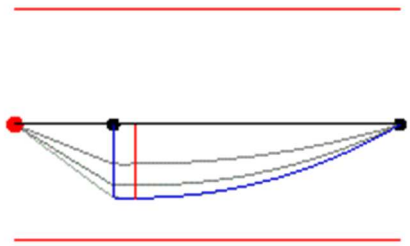
AXIAL WITH MOMENTS (MEMBER)
Member SBL1Id 17 @ Level 1 : Ex Gable Support Beam Between 0.970 and 3.800 m, in Load Case 1

Member Loading and Member Forces

```
Loading Combination : 1 UT + 1.4 D1 + 1.6 L1

D1 D     077.010              ( kN/m³)
D1 PTRY -011.620 0.970 3.799 -011.620
L1 PTRY -005.400 0.970 3.799 -005.400
D1 PTY2 -010.360 0.970 0.000 (kN,m,m)
```



Member Forces in Load Case 1 and Maximum Deflection from Load Case 7										
Member 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	
	6	0.00C	0.00	103.72	0.00	0.00	0.00	100.87		8.12
	18	0.00C	0.00	-81.47	0.00	0.00	0.00	@ 1.196		@ 1.819

Classification and Effective Area (EN 1993: 2006)

Section (59.06 kg/m)	203x203 UC 46 + 275x6 T Plate 59.06 [S 275]		
Class = Fn(b/T,d/t,f_y,N,M_y,M_z)	9.25, 22.33, 275, 0, 100.86, 0	(Axial: Non-Slender)	Class 1
Auto Design Load Cases	1 & 4-5		
Top plate in tension along member axis			

Local Capacity Check

$V_{y.Ed}/V_{pl.y.Rd}$	0.135 / 269.498 =	0.001	Low Shear
$M_{c.y.Rd} = f_y.W_{pl.y}/ \gamma_{M0}$	275 x 579.5/1	159.363 kN.m	
$N_{pl.Rd} = Ag.fy/\gamma_{M0}$	75.23x275/1 (No bearing / block tearing design)	2068.825 kN	
$n = N_{Ed}/N_{pl.Rd}$	0 / 2068.825 =	0.000	OK
$W_{pl.N.y}$ = Fn($W_{pl.y}$, A_{vy}, n)	579.5, 16.974, 0	579.5 cm³	
$M_{N.y.Rd} = W_{pl.N.y}.f_y/ \gamma_{M0}$	579.5 x 275/1	159.363 kN.m	
$(M_{y.Ed}/M_{N.y.Rd})+(M_{z.Ed}/M_{N.z.Rd})$	$(100.863/159.363)^2+(0)^1$=	0.401	OK

Compression Resistance N.b.Rd

$\lambda y = \sqrt{A.fy/Ncr}$	√75.23x275/8579.92	0.491	
$N_{b.y.Rd} = Area.\chi.f_y/ \gamma_{M1}$	75.23x0.888x275/10/1 =	1837.253 kN	Curve b
$\lambda z = \sqrt{A.fy/Ncrz}$	√75.23x275/6697.59	0.555	
$N_{b.z.Rd} = Area.\chi.f_y/ \gamma_{M1}$	75.23x0.812x275/10/1 =	1678.918 kN	Curve c
Let = Kt.Lx	1x3.8 =	3.8	
NcrT = min(NcrT,NcrtF)	min(4232.04, 4041.15)	4041.15	
$\lambda T = \sqrt{A.fy/NcrT}$	√75.23x275/4041.15	0.716	
$N_{b.T.Rd}= Area.\chi.fy/ \gamma_{M1}$	75.23x0.715x275/10/1 =	1479.358 kN	Curve c

Equivalent Uniform Moment Factors C1, C3, C.mLT, C.mz, and C.my

C_1= fn(M_1, M_2, M_o, ψ,μ)	100.2, 0.1, 30.8, 0.001, 0.308	1.251	Uniform
C_3, Z_j(bottom flange compression)	0.767, -39.60 mm		
C_{mLT}=Max(0.2+0.8α_s, 0.4)	M_h= 100.22, M_s= 80.98, ψ = 0.001, α_s= 0.808	0.846	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= 94.01, ψ = 1.000, α_s= 0.000	0.95	Table B.3

Lateral Buckling Check M.b.Rd

Le = (1.2 L+1.00 L)/2	(1.2 x 2.83 + 1.00 x 2.83)/2 =	3.113 m	
M_{cr}= Fn(C_1,L_e,I_z,I_t,I_w,E)	1.251, 3.113, 2588, 24.13, 0.2106, 210000	566.141 kN.m	
λ_{LT}= √ W.f_y/M_{cr}	√ 579.5 x 275 / 566.141	0.531	
χ_{LT}= Fn(λ_{LT}, λ_{LT5950})	0.531, 0.536	0.947	Curve b
χ_{LT}.mod = Fn(χ_{LT},λ_{LT},k_c,f)	0.947, 0.531, 0.894, 0.955	0.992	6.3.2.3
$M_{b.Rd} = \chi W_{pl.y}.f_y \leq M_{c.y.Rd}$	0.992 x 579.5 x 275 ≤ 159.363 =	158.136 kN.m	

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Buckling Resistance

$U_{N.y} = N_{Ed}/(\chi_y.N_{Rk}/\gamma_{M1})$	0 / 1837.253	0.000	OK
$U_{N.z} = N_{Ed}/(\chi_z.N_{Rk}/\gamma_{M1})$	0 / 1678.918	0.000	OK
$U_{M.y} = M_{y.Ed}/(\chi_{LT}.M_{y.Rk}/\gamma_{M1})$	100.863 / 158.136	0.638	OK
$U_{M.z} = M_{z.Ed}/(M_{z.Rk}/\gamma_{M1})$	0 / 94.683	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+0.950x0.638+0.600x0.000	0.606	OK
$U_{Nz}+k_{zy}.U_{M.y}+k_{zz}.U_{M.z}$	0.000+1.000x0.638+1.000x0.000	0.638	OK

Deflection Check - Load Case 7

In-span δ ≤ Span/360	8.12 ≤ 3800 / 360	8.12 mm	OK

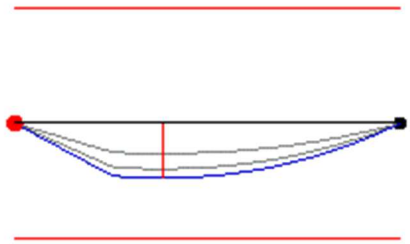
AXIAL WITH MOMENTS (MEMBER)

Member SBL1Id 18 @ Level 1 in Load Case 1 : New Gable Supprt Beam

Member Loading and Member Forces

```
Loading Combination : 1 UT + 1.4 D1 + 1.6 L1

D1 D     077.010              ( kN/m³)
D1 PTRY -000.315 0.000 0.970 -000.315
L1 PTRY -000.394 0.000 0.970 -000.394
D1 PTRY -011.350 0.970 3.799 -011.350
L1 PTRY -003.844 0.970 3.799 -003.844
D1 PTY2 -010.360 0.970 0.000 (kN,m,m)
```



Member Forces in Load Case 1 and Maximum Deflection from Load Case 7

Member 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	
	10	0.00C	0.00	75.67	0.00	0.00	0.00	75.37	0.00	6.26
	19	0.00C	0.00	-67.63	0.00	0.00	0.00	@ 1.451	@ 0.000	@ 1.847

Classification and Effective Area (EN 1993: 2006)

Section (59.06 kg/m)	203x203 UC 46 + 275x6 T Plate 59.06 [S 275]		
Class = Fn(b/T,d/t,f_y,N,M_y,M_z)	9.25, 22.33, 275, 0, 75.36, 0	(Axial: Non-Slender)	Class 1
Auto Design Load Cases	1 & 4-5		
Top plate in tension along member axis			

Local Capacity Check

$V_{y.Ed}/V_{pl.y.Rd}$	0.127 / 269.498 =	0	Low Shear
$M_{c.y.Rd} = f_y.W_{pl.y}/\gamma_{M0}$	275 x 579.5/1	159.363 kN.m	
$N_{pl.Rd} = Ag.fy/\gamma_{M0}$	75.23x275/1 (No bearing / block tearing design)	2068.825 kN	
$n = N_{Ed}/N_{pl.Rd}$	0 / 2068.825 =	0.000	OK
$W_{pl.N.y}$ = Fn($W_{pl.y}$, A_{vy}, n)	579.5, 16.974, 0	579.5 cm³	
$M_{N.y.Rd} = W_{pl.N.y}.f_y/\gamma_{M0}$	579.5 x 275/1	159.363 kN.m	
$(M_{y.Ed}/M_{N.y.Rd})+(M_{z.Ed}/M_{N.z.Rd})$	$(75.365/159.363)^2+(0)^1$=	0.224	OK

Equivalent Uniform Moment Factors C1, C3, C.mLT, C.mz, and C.my

C_1= fn(M_1, M_2, M_o, ψ,μ)	0.1, 0.1, 72.8, 0.932, 300.000	1.127	Uniform
C_3, Z_j(bottom flange compression)	0.525, -39.60 mm		
$C_{mLT}=0.95+0.05\alpha_h$	M_h= 0.07, M_s= 72.89, ψ = 0.932, 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= 72.89, ψ = 1.000, α_s= 0.000	0.95	Table B.3

Lateral Buckling Check M.b.Rd

Le = 1.00 L	1 x 3.8 =	3.8 m	

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M_{cr}= Fn(C_1,L_e,I_z,I_t,I_w,E)	1.127, 3.800, 2588, 24.13, 0.2106, 210000	405.070 kN.m	
λ_{LT}= √ W.f_y/M_{cr}	√ 579.5 x 275 / 405.07	0.627	
χ_{LT}= Fn(λ_{LT}, λ_{LT5950})	0.627, 0.625	0.905	Curve b
χ_{LT}.mod = Fn(χ_{LT},λ_{LT},k_c,f)	0.905, 0.627, 0.942, 0.973	0.930	6.3.2.3
$M_{b.Rd}$ = χ $W_{pl.y}$.f_y≤ $M_{c.y.Rd}$	0.930 x 579.5 x 275 ≤ 159.363 =	148.212 kN.m	

Buckling Resistance

$U_{N.y}$ = N_{Ed}/(χ_y.N_{Rk}/γ_{M1})	0 / 1837.253	0.000	OK
$U_{N.z}$ = N_{Ed}/(χ_z.N_{Rk}/γ_{M1})	0 / 1440.4	0.000	OK
$U_{M.y}$ = $M_{y.Ed}$/(χ_{LT}.$M_{y.Rk}$/γ_{M1})	75.365 / 148.212	0.508	OK
$U_{M.z}$ = $M_{z.Ed}$/($M_{z.Rk}$/γ_{M1})	0 / 94.683	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)$U_{N.z}$}		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+0.950x0.508+0.600x0.000	0.483	OK
U_{Nz}+k_{zy}.$U_{M.y}$+k_{zz}.$U_{M.z}$	0.000+1.000x0.508+1.000x0.000	0.508	OK

Deflection Check - Load Case 7

In-span δ ≤ Span/360	6.26 ≤ 3800 / 360	6.26 mm	OK

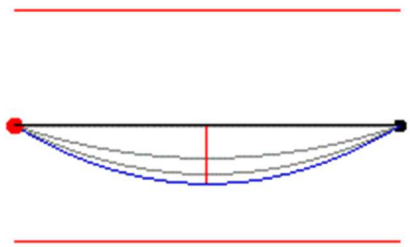
AXIAL WITH MOMENTS (MEMBER)

Member SBL1Id 12 @ Level 1 in Load Case 1 : New Rear Elevation support Beam

Member Loading and Member Forces

```
Loading Combination : 1 UT + 1.4 D1 + 1.6 L1

D1 UDLY -014.441            ( kN/m )
L1 UDLY -003.724            ( kN/m )
D1 D     077.010            ( kN/m³)
```



Member Forces in Load Case 1 and Maximum Deflection from Load Case 7

Member 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	
	13	0.00C	0.01	62.27	0.00	0.00	0.00	71.65	0.00	12.29
	15	0.00C	-0.01	-62.30	0.00	0.00	0.00	@ 2.291	@ 0.000	@ 2.291

Classification and Effective Area (EN 1993: 2006)

Section (64.12 kg/m)	152x152 UC 51 + 275x6 T Plate 64.12 [S 275]		
Class = Fn(b/T,d/t,f_y,N,M_y,M_z)	5.01, 11.24, 275, 0, 71.65, 0	(Axial: Non-Slender)	Class 1
Auto Design Load Cases	1 & 4-5		
Top plate in tension along member axis			

Local Capacity Check

$V_{y.Ed}$/$V_{pl.y.Rd}$	0.035 / 315.475 =	0	Low Shear
$M_{c.y.Rd}$ = f_y.$W_{pl.y}$/ γ_{M0}	275 x 521.9/1	143.523 kN.m	
$N_{pl.Rd}$ = A_g.f_y/ γ_{M0}	81.69 x 275/1 =	2246.475 kN	
n = N_{Ed}/$N_{pl.Rd}$	0.001 / 2246.475 =	0.000	OK
$W_{pl.N.y}$ = Fn($W_{pl.y}$, A_{vy}, n)	521.9, 19.87, 0	521.9 cm³	
$M_{N.y.Rd}$ = $W_{pl.N.y}$.f_y/ γ_{M0}	521.9 x 275/1	143.523 kN.m	
($M_{y.Ed}$/$M_{N.y.Rd}$)+($M_{z.Ed}$/$M_{N.z.Rd}$)	(71.645/143.523)²+(0)¹=	0.249	OK

Compression Resistance N.b.Rd

λy = √A.fy/Ncr	√81.69x275/4162.74	0.735	
$N_{b.y.Rd}$ = Area.χ.f_y/ γ_{M1}	81.69x0.764x275/10/1 =	1715.493 kN	Curve b
λz = √A.fy/Ncrz	√81.69x275/2019.76	1.056	

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$N_{b.z.Rd}$ = Area.χ.f_y/ γ_{M1}	81.69x0.508x275/10/1 =	1142.163 kN	Curve c
Let = Kt.Lx	1x4.6 =	4.6	
NcrT = min(NcrT,NcrtF)	min(6035.58, 1942.01)	1942.01	
λT = √A.fy/NcrT	√81.69x275/1942.01	1.076	
$N_{b.T.Rd}$= Area.χ.fy/ γ_{M1}	81.69x0.497x275/10/1 =	1117.432 kN	Curve c

Equivalent Uniform Moment Factors C1, C3, C.mLT, C.mz, and C.my

C_1= fn(M_1, M_2, M_o, ψ,μ)	0.1, 0.1, 71.6, 1.000, 300.000	1.127	Uniform
C_3, Z_j(bottom flange compression)	0.525, -40.83 mm		
C_{mLT}=0.95+0.05α_h	M_h= 0.06, M_s= 71.65, ψ = 1.000, 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= 71.65, ψ = 1.000, α_s= 0.000	0.95	Table B.3

Lateral Buckling Check M.b.Rd

Le = 1.00 L	1 x 4.6 =	4.6 m	
M_{cr}= Fn(C_1,L_e,I_z,I_t,I_w,E)	1.127, 4.600, 2062, 50.81, 0.1005, 210000	315.767 kN.m	
λ_{LT}= √ W.f_y/M_{cr}	√ 521.9 x 275 / 315.767	0.674	
χ_{LT}= Fn(λ_{LT}, λ_{LT5950})	0.674, 0.639	0.882	Curve b
χ_{LT}.mod = Fn(χ_{LT},λ_{LT},k_c,f)	0.882, 0.674, 0.942, 0.972	0.908	6.3.2.3
$M_{b.Rd}$ = χ $W_{pl.y}$.f_y≤ $M_{c.y.Rd}$	0.908 x 521.9 x 275 ≤ 143.523 =	130.299 kN.m	

Buckling Resistance

$U_{N.y}$ = N_{Ed}/(χ_y.N_{Rk}/γ_{M1})	0.003 / 1715.493	0.000	OK
$U_{N.z}$ = N_{Ed}/(χ_z.N_{Rk}/γ_{M1})	0.003 / 1142.163	0.000	OK
$U_{M.y}$ = $M_{y.Ed}$/(χ_{LT}.$M_{y.Rk}$/γ_{M1})	71.645 / 130.299	0.550	OK
$U_{M.z}$ = $M_{z.Ed}$/($M_{z.Rk}$/γ_{M1})	0 / 85.938	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+0.950x0.550+0.600x0.000	0.522	OK
U_{Nz}+k_{zy}.$U_{M.y}$+k_{zz}.$U_{M.z}$	0.000+1.000x0.550+1.000x0.000	0.550	OK

Deflection Check - Load Case 7

In-span δ ≤ Span/360	12.29 ≤ 4600 / 360	12.29 mm	OK

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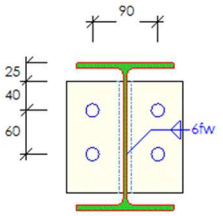
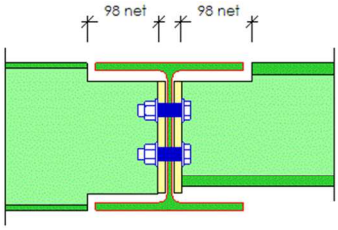
CONNECTIONS

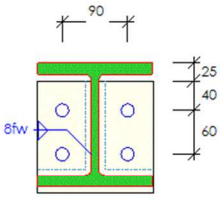
BEAM TO BEAM: FLEXIBLE END-PLATE CONNECTION AT END 1 OF MEMBER 12



Left Beam: 203x133 UB 25 [S275]
Top Notch: 25 x 98 mm lg. net
Bot Notch: 23 x 98 mm lg. net
Plate: 10 x 160 x 155 mm dp. (2 kg)
With 4 No. 18 mm holes
For 16 mm Ø Grade 8.8 Bolts.



Beam: 203x203 UC 46 [S275]
End-Plates S 275



Right Beam: 152x152 UC 51 [S275]
Top Notch: 25 x 98 mm lg. net
Plate: 10 x 160 x 155 mm dp. (2 kg)
With 4 No. 18 mm holes
For 16 mm Ø Grade 8.8 Bolts.

Beam to Beam Flexible End-plate Connection to EC 3 (UK NAD)

Basic Data

Integrated Applied Forces at Interface

Shear Forces	Left = 34.9 kN, Right = 62.3 kN
Tie Force	75.0 kN
Design to	EC 3: Part 1-8: 2005 Design of Joints
SCI Green Book	P358: Joints in steel construction: Simple joints to Eurocode 3

Basic Dimensions

Left-203x133UB25 [28]	D=203.2, B=133.2, T=7.8, t=5.7, r=7.6, py=275
Supporting-203x203UC46 [28]	D=203.2, B=203.6, T=11.0, t=7.2, r=10.2, py=275
Right-152x152UC51 [28]	D=170.2, B=157.4, T=15.7, t=11.0, r=7.6, py=275
Bolts 16 mm Ø in 18 mm holes	Grade 8.8 Bolts
Plates S 275	All weld grades provided to suit minimum connected steel grade

Summary of Results (Unity Ratios)

Left Hand Beam

Check 4 Supported Beam Web Shear	126.2 >= 34.9kN	0.28	OK
Checks 1 & 2 Detailing Practice	1.00, 0.83, 1.00, 0.64, 0.710.54	1.00	OK
Check 5: Notch Resistance	0.61	0.61	OK
Check 6: Notch Stability	0.61	0.61	OK
Checks 8 & 9 Bearing & Shear	192.9, 388.7, 325.4, 512.2, 473.3 >= 34.9kN	0.18	OK
Check 10 Shear	194.3, 207.6 >= 48.6kN	0.25	OK
Check 10 Combined Bearing	81.3 >= 24.3kN	0.30	OK
Check 11-14 Tie Forces	168.2, 236.1, 411.1, 329.3 >= 75.0kN	0.45	OK

Right Hand Beam

Check 4 Supported Beam Web Shear	227.9 >= 62.3kN	0.27	OK
Checks 1 & 2 Detailing Practice	1.00, 0.83, 1.00, 0.64, 0.540.79	1.00	OK
Check 5: Notch Resistance	0.41	0.41	OK
Check 6: Notch Stability	0.59	0.59	OK
Checks 8 & 9 Bearing & Shear	192.9, 388.7, 325.4, 512.2, 473.3 >= 62.3kN	0.32	OK
Check 11-14 Tie Forces	186.2, 249.5, 411.1, 569.1 >= 75.0kN	0.40	OK

Left Hand Beam

Check 1: Recommended Detailing Practice

Plate Depth >= 0.6•D	0.6 • 203.2 = 121.9 mm	155 mm	OK
10 >= t_p<= 12	10.0 mm		OK
90 <= p_3 <= 140	90 mm		OK

Check 4: Supported Beam Web Shear

$V_{pl\_Rd}=0.9•t_{bw}•h_{pw}•f_{ybw} / (\sqrt{3}•\gamma_{M0})$	0.9 • 5.7 • 155 • 275 / (√3 • 1.0)	126.2 kN	OK

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Check 2: Supported Beam Welds

$a=0.4\ t_{wb1}$	0.4 • 5.7	2.3 mm	
$a_1=a•V_{Ed}/\ V_{pl.Rd}•1.27$	2.3 • 34.9 / 126.2 • 1.27 (SCI AD370)	0.8 mm	
$S_{min}=max(3,\ min(\ a,\ a_1\ )\ )\ /\ 0.7$		4.3 mm	OK

Check 5: Supported Beam Notch Resistance

$V_{Rd}=0.9•A_v.DN•f_{ybw}\ /(\sqrt{3}•\gamma_{M0})$	0.9 • 885 • 275 / (√3 • 1.0)	126.4 kN	Low Shear
$M_{v.Rd}=f_{y.b1}•W_{el.N.y}/\ \gamma_{M0}$	275 • 23/1.0	6.3 kN.m	
$V_{Rd}=Min(V_{Rd}\ ,\ M_{v.rd}/\ La)$	Min(126.41, 6.29 / 110)	57.2 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

Check 8a: Bolt Shear

$F_{v,Rd}=a_v•f_{ub}•A\ /\ \gamma_{M2}$	0.6 • 800 • 157.0 / 1.25	60.3 kN	
$V_{Rd1}=0.8•n•F_{v,Rd}•cols$	0.8 • 2 • 60.3 • 2	192.9 kN	OK

Check 8b: End-Plate Bearing

$P_1,\ e_1,\ P_2,\ e_2,\ a_b,\ k_1$	60.0, 40.0, 90.0, 35.0, 0.741, 2.500		
$F_{b,Rd}=k_1•\ a_b•f_{up}•d•t_p/\ \gamma_{M2}$	2.5 • 0.741 • 410 • 16 • 10.0 / 1.25	97.2 kN	
$F_{b,Rd}<=\ 0.80•F_{v,Rd}$			
$F_{Rd}=n•f_{b,Rd•rows}$	2 • 97.2 • 2	388.7 kN	OK

Check 8c: Supporting Beam Web Bearing

$a_b=Min(\ p_1/d_0/3\text{-}¼,\ f_{ub}/\ f_u,\ 1)$	Min(60/18/3-¼, 800/410, 1) » Min(0.861, 1.951, 1)	0.861	
$k_{1.inn}=min(2.5,\ 1.4•p_2/d_0\text{-}1.7)$	min(2.5, 1.4 • 90/18 - 1.7 » Min(2.5, 5.3)	2.500	
$F_{b,Rd}=k_1•a_b•f_{u,2}\ •d•t_p/\ \gamma_{M2}$	2.5 • 0.861 • 410 • 16.0 • 7.2 / 1.25	81.3 kN	
$F_{b,Rd}<=\ 0.80•F_{v,Rd}$			
$F_{Rd}=n•f_{b,Rd•rows}$	2 • 81.3 • 2	325.4 kN	OK

Check 9a: End-Plate Plane Shear

$V_{Rd4}=2•\ h_p•t_p•F_{yp}/(1.27•\sqrt{3}•\gamma_{M0})$	2•155 • 10 • 275/(1.27 • √3 • 1.0)	387.6 kN	OK
$V_{Rd}=2•\ t_p.(h_p\text{-}n_{1•d\_0})•F_{up}/(\ \sqrt{3}•\gamma_{M2})$	2•10•(155 - 2 • 18) • 410/(√3 • 1.1)	512.2 kN	OK

Check 9b: End-Plate Block Shear

$A_{nv},\ A_{nt}=Fn(t_p,\ h_p,\ e_1,\ e_2,\ n_1,\ d_0)$	(10, 155, 40, 35, 2, 18)	880, 260 mm²	
$V_{Rd}=2•(F_{up}•A_{nt}/\gamma_{M2}+f_{yp}•A_{nv}/(\sqrt{3}•\gamma_{M0}))$	2•(410 • 260 / 1.1 + 275880 • /(√3 • 1.0)	473.3 kN	OK

Check 10: Supporting Member - Local Resistance

$V_{ed}=(\ V_{ed1}+V_{ed2})/2$	(35 + 62)/2 (per shear plane)	48.6 kN	
$A_v,\ A_{Vnet}=Fn(e_t,\ e_b,\ p_1,\ n,\ dia,\ T)$	fn(65, 45, 60, 2, 18.0, 7.2)	1224, 965 mm²	
$V_{Rd}=A_v•F_{yp}/(\ \sqrt{3}•\gamma_{M0})$	1224 • 275/(√3 • 1.0)	194.3 kN	OK
$V_{Rd}=A_{vnet•F\_up}/(\ \sqrt{3}•\gamma_{M2})$	965 • 410/(√3 • 1.1)	207.6 kN	OK

Check 10: Combined Load Bearing on supporting web

$F_{ed}=V_{ed1}/n_1+V_{ed2}/n_2\ <=\ F_{b,Rd}$	34.9/4 + 62.3/4	(per bolt) 24.3 kN	OK

Tie Forces

Applied Tie Force	75.0 kN		

Check 11: End-Plate in Bending

$L_{eff}=e_{1A}+(n\text{-}1)•p_{1A}+e_{1A}$	40.0 + (2-1) • 60 + 45.2	145.2 mm	
$M_{pl1}=0.25•L_{eff}•t^2•f_{u.p}/\ \gamma_{Mu}$	0.25 • 145.2 • 10² • 410 / 1.1	1.35 kN.m	
$F_{Rd.u.1}=(8•n\text{-}2•e_w)•\ m_{pl1}$ / $(2•m\ •n\text{-}e_w•(m+n))$	(8 • 35.0 - 2 • 6.5) • 1 / (2 • 37.4 • 35.0 - 6.5 • (37.4 + 35.0))	168.2 kN	
$F_{Rd.u.2}=$ $(2•M_{pl2}+n•S(\ k_2•f_{ub}•A\ /\ \gamma_{mu}\ ))/(m+n)$	(2•1.35 + 35.00•4•0.90•800•157/1.1)/ (37.4 + 35.0)	236.1 kN	
$F_{rd.u.3}=S•k_2•F_{ub}•A_s/\ \gamma_{Mu}$	4 • 0.90•800 • 157/1.1	411.1 kN	
$F_{Rd}=min(F_{rd.u.1},\ F_{rd.u.2},\ F_{rd.u.3})$	Min (168.25, 236.10, 411.05)	168.2 kN	OK

Check 12: Beam Web Tension

$F_{Rd}=t_{bw}•h_{peff}•F_{ubw}\ /\ \gamma_{Mu}$	5.7 • 155 • 410 / 1.1	329.3 kN	OK

Check 13: Welds

n/a	Covered by Check 2		OK

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Right Hand Beam

Check 1: Recommended Detailing Practice

Plate Depth >= 0.6•D	0.6 • 170.2 = 102.1 mm	155 mm	OK
10 >= t_p<= 12	10.0 mm		OK
90 <= p_3 <= 140	90 mm		OK

Check 4: Supported Beam Web Shear

$V_{pl\_Rd}=0.9•t_{bw•h\_pw•f\_ybw} / (\sqrt{3}•\gamma_{M0})$	0.9 • 11.0 • 145 • 275 / (√3 • 1.0)	227.9 kN	OK

Check 2: Supported Beam Welds

$a=0.4\ t_{wb1}$	0.4 • 11.0	4.4 mm	
$a_1=a•V_{Ed}/ V_{pl.Rd}•1.27$	4.4 • 62.3 / 227.9 • 1.27 (SCI AD370)	1.5 mm	
$S_{min}=max(3, min(a, a_1)) / 0.7$		4.3 mm	OK

Check 5: Supported Beam Notch Resistance

$V_{Rd}=A_v.N•f_{ybw} /(\sqrt{3}•\gamma_{M0})$	1643 • 275 / (√3 • 1.0)	260.8 kN	Low Shear
$M_{v.Rd}=f_{y.b1}•W_{el.N.y}/ \gamma_{M0}$	275 • 61/1.0	16.9 kN.m	
$V_{Rd}=Min(V_{Rd} , M_{v.rd}/ La)$	Min(260.79, 16.89 / 110)	153.5 kN	OK

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

$D_{nt}<= h_{\_b1}/ 2$	25.0 <= 170.2/ 2 mm	25.0 <= 85.1	OK
$L_n <= h_{b1}$		100 <= 170	OK

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

	Beam assumed to be restrained		ignored

Check 8a: Bolt Shear

$F_{v,Rd}=\alpha_v•f_{ub}•A / \gamma_{M2}$	0.6 • 800 • 157.0 / 1.25	60.3 kN	
$V_{Rd1}=0.8•n•F_{v,Rd}•cols$	0.8 • 2 • 60.3 • 2	192.9 kN	OK

Check 8b: End-Plate Bearing

P_1, e_1, P_2, e_2, α_b, k_1	60.0, 40.0, 90.0, 35.0, 0.741, 2.500		
$F_{b,Rd}=k_1• \alpha_b•f_{up}•d•t_p/ \gamma_{M2}$	2.5 • 0.741 • 410 • 16 • 10.0 / 1.25	97.2 kN	
$F_{b,Rd}<= 0.80•F_{v,Rd}$			
$F_{Rd}=n•f_{b,Rd•rows}$	2 • 97.2 • 2	388.7 kN	OK

Check 8c: Supporting Beam Web Bearing

$\alpha_b=Min(p_1/d_0/3-¼, f_{ub}/ f_u, 1)$	Min(60/18/3-¼, 800/410, 1) » Min(0.861, 1.951, 1)	0.861	
$k_{1.inn}=min(2.5, 1.4•p_2/d_0-1.7)$	min(2.5, 1.4 • 90/18 - 1.7 » Min(2.5, 5.3)	2.500	
$F_{b,Rd}=k_1•\alpha_b•f_{u,2} •d•t_p/ \gamma_{M2}$	2.5 • 0.861 • 410 • 16.0 • 7.2 / 1.25	81.3 kN	
$F_{b,Rd}<= 0.80•F_{v,Rd}$			
$F_{Rd}=n•f_{b,Rd•rows}$	2 • 81.3 • 2	325.4 kN	OK

Check 9a: End-Plate Plane Shear

$V_{Rd4}=2• h_p•t_p•F_{yp}/(1.27•\sqrt{3}•\gamma_{M0})$	2•155 • 10 • 275/(1.27 • √3 • 1.0)	387.6 kN	OK
$V_{Rd}=2• t_p•(h_p-n_{1•d\_0})•F_{up}/(\sqrt{3}•\gamma_{M2})$	2•10•(155 - 2 • 18) • 410/(√3 • 1.1)	512.2 kN	OK

Check 9b: End-Plate Block Shear

A_{nv}, $A_{nt}=Fn(t_p, h_p, e_1, e_2, n_1, d_0)$	(10, 155, 40, 35, 2, 18)	880, 260 mm²	
$V_{Rd}=2•(F_{up}•A_{nt}/\gamma_{M2}+f_{yp}•A_{nv}/(\sqrt{3}•\gamma_{M0}))$	2•(410 • 260 / 1.1 + 275880 • /(√3 • 1.0)	473.3 kN	OK

Check 10: Supporting Member - Local Resistance

	See other side		

Tie Forces

Applied Tie Force	75.0 kN		

Check 11: End-Plate in Bending

$L_{eff}=e_{1A}+(n-1)•p_{1A}+e_{1A}$	40.0 + (2-1) • 60 + 40.6	140.6 mm	
$M_{pl1}=0.25•L_{eff}•t^2•f_{u.p}/ \gamma_{Mu}$	0.25 • 140.6 • 10² • 410 / 1.1	1.31 kN.m	
$F_{Rd.u.1}=(8•n-2•e_w)• m_{pl1}$ / $(2•m •n-e_w•(m+n))$	(8 • 35.0 - 2 • 6.5) • 1 / (2 • 33.2 • 35.0 - 6.5 • (33.2 + 35.0))	186.2 kN	
$F_{Rd.u.2}=$ $(2•M_{pl2}+n•S(k_2•f_{ub}•A / \gamma_{mu}))/(m+n)$	(2•1.31 + 35.00•4•0.90•800•157/1.1)/ (33.2 + 35.0)	249.5 kN	
$F_{rd.u.3}=S•k_2•F_{ub}•A_s/ \gamma_{Mu}$	4 • 0.90•800 • 157/1.1	411.1 kN	
$F_{Rd}=min(F_{rd.u.1}, F_{rd.u.2}, F_{rd.u.3})$	Min (186.20, 249.50, 411.05)	186.2 kN	OK

Check 12: Beam Web Tension

$F_{Rd}=t_{bw}•h_{peff}•F_{ubw} / \gamma_{Mu}$	11.0 • 139 • 410 / 1.1	569.1 kN	OK

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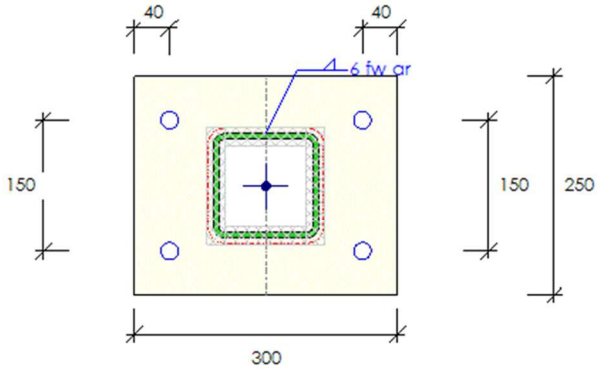
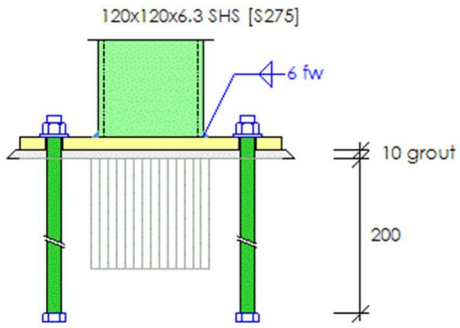
Check 13: Welds

n/a	Covered by Check 2	OK

BASE PLATE AT : END 1 OF MEMBER 23



Base-Plate 300 x 250 x 15 mm (9 kg)
With 4 No. 20 mm holes
For 16 mm Grade 8.8 Bolts.



No Void: Bolts Fully Embedded in Base
All Plates S 275

Base-Plate Connection to EC 3 (UK NAD)

LOADING CASE 001 : DEAD PLUS LIVE (ULTIMATE)

Basic Data

Integrated Applied Forces at Interface

Resultant Forces M, Fv, F	Moment +0.0 kNm, Shear +0.0 kN, Axial +107.9 kN Forces taken from Support Reaction (Axial Compression)		
Design to	EC 3: Part 1-8: 2005 Design of Joints		
SCI Green Book	P358: Joints in steel construction: Simple joints to Eurocode 3		
Column Capacities Mc, Fvc, Fc	33.6 kN.m, 227.5 kN, 787.9 kN	Fc = 787.9 kN	OK

Summary of Results (Unity Ratios)

Concrete Pressure	1.00	OK
Base-Plate thickness in Compression	0.17	OK

STEP 2 - COMPRESSION T-STUB

Pad Diffusion Ratio α	Using SCI default Value	1.50	
Grout Ratio β_J		0.67	
$F_{cd}=\alpha_{cc} \bullet F_{ck} / \gamma_c$	0.85 • 20.00 / 1.50	11.33 N/mm²	
Bearing Stress $F_{jd}= \beta_J \bullet \alpha \bullet F_{cd}$	0.67 • 1.50 • 11.33	11.33 N/mm²	
Max $c = T_{bp} \bullet \sqrt{(f_{y.bp} / (3 \bullet f_{jd} \bullet \gamma_{M0}))}$	15•√(275 / 3•(11.33•1.0))	42.7 mm	

Step 1-2: Base-Plate Pressure

Base ecc=M/F	0.0/107.9	0.0 mm	
Pressure Configuration	Compression Only with Optimised Compression Area		
Proj C, Xcc	7.3, 150.0		
L Zones X1, X2, X3, X4, X5	82.7, 20.9, 92.8, 20.9, 0.0		
W Zones Wstiff, Wflange, Wweb	0.0, 134.6, 41.9		
$Ac=x_f \bullet wf_c+x_w \bullet ww_c+x_f \bullet wf_t$	20.94•134.64 + 92.76•41.89 + 20.94•134.64	95.3 cm²	
Conc Cap Fc=•F_{jd}•Ac	11.3•9525.1	108.0 kN	OK
Pressure=P•1000/Ac	107.9•1000/9525	11.32 N/mm²	OK
$h_f=h_c-t_{fc}$	120.0 - 6.3	113.7	
$N_{R,T}= M_{Ed}/h_f+F/2$	0.0/113.7 + 107.9/2	53.9 kN	
$F_{c,fc,Rd}= M_{c,Rd}/h_f$	32.9•1000/(113.70)	289.4 kN	OK

Step 2a: Plate Compression Bending

e=C	7.3	7.3 mm	
$M_{app}=p \bullet e^2/2$	11.3•7.3²/2	304 Nmm/mm	

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$tp=\sqrt{(6 \bullet M_{app} \bullet \gamma_{M0}/p_y)}$	$\sqrt{(6 \bullet 304 \bullet 1.00/275)}$	2.6 mm	OK

Step 5: Flange & Web Welds

Load dispersal	Flanges resist Moment and Axial, Web resists Axial and Shear. Direct Bearing therefore design for tensile forces only.	
Areas A, Af, Aw	28.7, 2 x 7.2, 14.3 cm²	

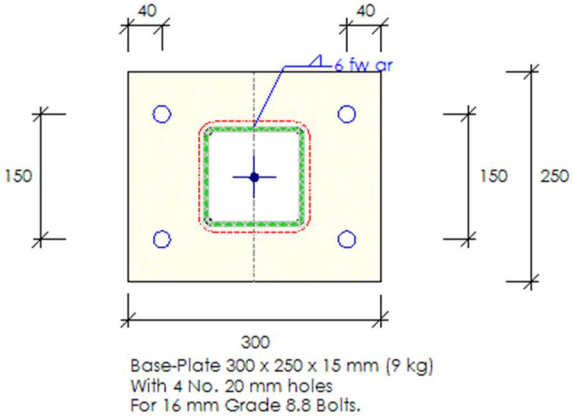
Flange Welds

$Fapp=F \bullet Af/A$	$107.9 \bullet 7.2/28.7$ No Resultant Tensile Force	-27.0 kN

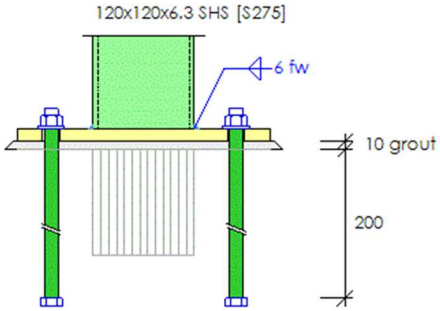
Web Welds

No Shear or Resultant Axial Force in Web.

Base Plate at : End 1 of Member 22



Base-Plate 300 x 250 x 15 mm (9 kg)
With 4 No. 20 mm holes
For 16 mm Grade 8.8 Bolts.



No Void: Bolts Fully Embedded in Base
All Plates S 275

Base-Plate Connection to EC 3 (UK NAD)

Loading Case 001 : Dead plus Live (Ultimate)

Basic Data

Integrated Applied Forces at Interface

Resultant Forces M, Fv, F	Moment +0.0 kNm, Shear +0.0 kN, Axial +4.7 kN Forces taken from Support Reaction (Axial Compression)		
Design to	EC 3: Part 1-8: 2005 Design of Joints		
SCI Green Book	P358: Joints in steel construction: Simple joints to Eurocode 3		
Column Capacities Mc, Fvc, Fc	33.6 kN.m, 227.5 kN, 787.9 kN	Fc = 787.9 kN	OK

Summary of Results (Unity Ratios)

Concrete Pressure	0.14	OK
Base-Plate thickness in Compression	0.01	OK

Step 2 - Compression T-Stub

Pad Diffusion Ratio α	Using SCI default Value	1.50
Grout Ratio β_J		0.67
$F_{cd}=\alpha_{cc} \bullet F_{ck} / \gamma_c$	$0.85 \bullet 20.00 / 1.50$	11.33 N/mm²
Bearing Stress $F_{jd}= \beta_J \bullet \alpha \bullet F_{cd}$	$0.67 \bullet 1.50 \bullet 11.33$	11.33 N/mm²
Max $c =T_{bp} \bullet \sqrt{(f_{y.bp} / (3 \bullet f_{jd} \bullet \gamma_{M0}))}$	$15 \bullet \sqrt{(275 / 3 \bullet (11.33 \bullet 1.0))}$	42.7 mm

Step 1-2: Base-Plate Pressure

Base ecc=M/F	0.0/4.7	0.0 mm
Pressure Configuration	Compression Only with Optimised Compression Area	
Proj C, Xcc	0.0, 150.0	
L Zones X1, X2, X3, X4, X5	90.0, 6.3, 107.4, 6.3, 0.0	

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W Zones Wstiff, Wflange, Wweb	0.0, 120.0, 12.6		
$Ac=x_f•wf_c+x_w•ww_c+x_f•wf_t$	6.32•120.02 + 107.38•12.64 + 6.32•120.02	28.7 cm²	
Conc Cap $Fc=•F_{jd}•Ac$	11.3•2874.7	32.6 kN	OK
Pressure=P•1000/Ac	4.7•1000/2875	1.64 N/mm²	OK
$h_f=h_c-t_{fc}$	120.0 - 6.3	113.7	
$N_{R,T}= M_{Ed}/h_f+F/2$	0.0/113.7 + 4.7/2	2.4 kN	
$F_{c,fc,Rd}= M_{c,Rd}/h_f$	32.9•1000/(113.70)	289.4 kN	OK

Step 2a: Plate Compression Bending

e=C	0.0	0.0 mm	
$M_{app}=p•e^2/2$	1.6•0.0²/2	0 Nmm/mm	
$tp=\sqrt{(6•M_{app}•\gamma_{M0}/p_y)}$	√(6•1•1.00/275)	0.2 mm	OK

Step 5: Flange & Web Welds

Load dispersal	Flanges resist Moment and Axial, Web resists Axial and Shear. Direct Bearing therefore design for tensile forces only.
Areas A, Af, Aw	28.7, 2 x 7.2, 14.3 cm²

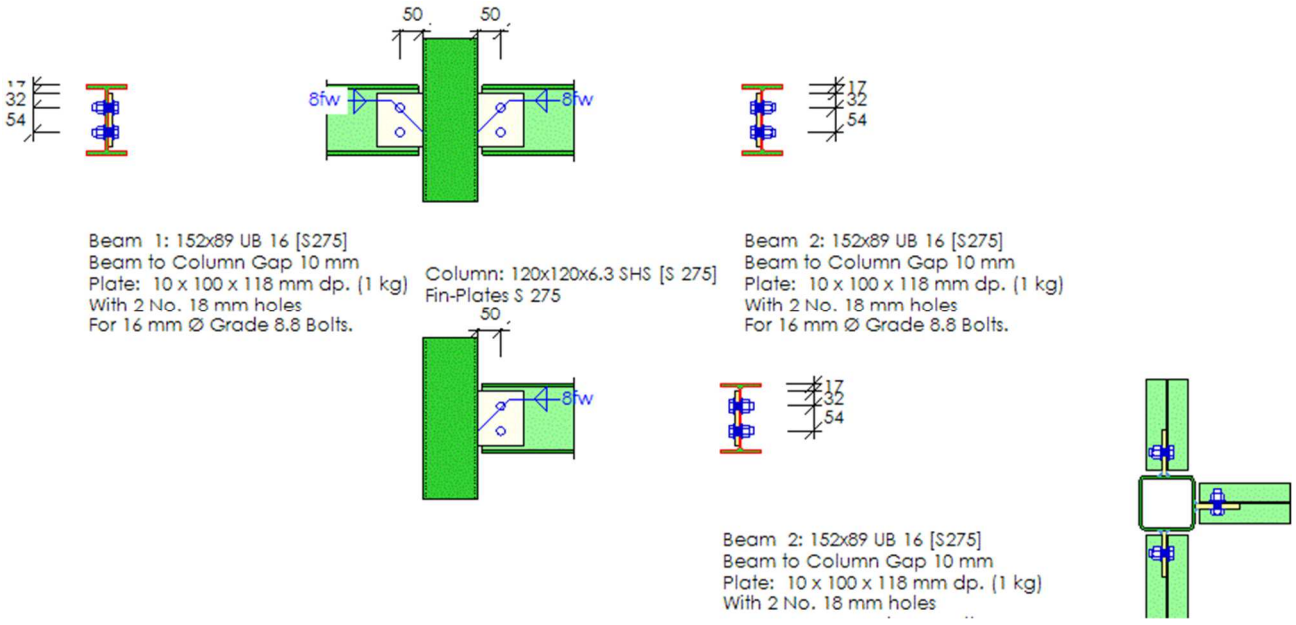
Flange Welds

Fapp=F•Af/A	4.7•7.2/28.7	-1.2 kN
	No Resultant Tensile Force	

Web Welds

No Shear or Resultant Axial Force in Web.

BEAM TO COLUMN: FIN-PLATE CONNECTION AT END 1 OF MEMBER 2



Beam to Column Fin-plate Connection to EC 3 (UK NAD)

Basic Data

Integrated Applied Forces at Interface

Shear Forces	Flange 1 = 1.6 kN, Web 1 = 0.0 kN
	Flange 2 = 2.2 kN, Web 2 = 0.1 kN
Tie Forces	Flanges = 75.0 kN, Webs = 75.0 kN
Design to	EC 3: Part 1-8: 2005 Design of Joints
SCI Green Book	P358: Joints in steel construction: Simple joints to Eurocode 3

Basic Dimensions

Flange 1-152x89UB16 [28]	D=152.4, B=88.7, T=7.7, t=4.5, r=7.6, py=275

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Flange 2-152x89UB16 [28]	D=152.4, B=88.7, T=7.7, t=4.5, r=7.6, py=275
Web 2-152x89UB16 [28]	D=152.4, B=88.7, T=7.7, t=4.5, r=7.6, py=275
Column -120x120x6.3SHS [28]	D=120.0, T=6.3, py=275
Bolts 16 mm Ø in 18 mm holes	Grade 8.8 Bolts
Plates S 275	All weld grades provided to suit minimum connected steel grade

Summary of Results (Unity Ratios)

Checks for Flange Beams 1 & 2

Check 1 Detailing Practice	0.77, 0.56	0.77	OK
Check 2: Bolt Shear & Bearing	0.04, 0.02, 0.05	0.05	OK
Check 3: Sup.Beam Connect Elmts.	0.01, 0.01, 0.01, 0.02, 0.02	0.02	OK
Check 4: Shear & Bending	0.02, 0.02, 0.05	0.05	OK
Check 8: Welds	0.890.89	0.89	OK
Check 10: Supporting Member	0.01, 0.02	0.02	OK
Check 11: Structural integrity-Plate	0.26, 0.28, 0.27, 0.5, 0.3	0.50	OK
Check 12: Structural integrity-Beam	0.76, 0.72, 0.43, 0.8	0.80	OK
Check 15: Structural integrity-Wall	0.8	0.80	OK

Checks for Web Beam on Side 2

Check 1 Detailing Practice	0.77, 0.56	0.77	OK
Check 2: Bolt Shear & Bearing	0.00, 0.00, 0.00	0.00	OK
Check 3: Sup.Beam Connect Elmts.	0.00, 0.00, 0.00, 0.00, 0.00	0.00	OK
Check 4: Shear & Bending	0.00, 0.00, 0.00	0.00	OK
Check 8: Welds	0.890.89	0.89	OK
Check 10: Supporting Member	0.00, 0.00	0.00	OK
Check 11: Structural integrity-Plate	0.26, 0.28, 0.27, 0.5, 0.3	0.50	OK
Check 12: Structural integrity-Beam	0.76, 0.72, 0.43, 0.8	0.80	OK
Check 15: Structural integrity-Wall	0.8	0.80	OK

Checks for Flange Beams 1 & 2

Check 1: Recommended Detailing Practice

Plate Depth >= 0.6•D	0.6 • 152.4 = 91.4 mm	118 mm	OK
Plate thickness check	min(tfin 10.0/0.5, tweb 4.5/0.5) / F	0.56 < 1	OK

Check 2: Supported beam - Bolt Group

i) Bolts in Shear

$ß=(6•z) / (n_1•(n_1+1)•p1)$	(6•50) / (2 • (2 + 1)• 54)	0.926	
$F_{v,Rd}=a_v•f_{ub}•A / \gamma_{M2}$	0.6 • 800 • 157.0 / 1.25	60.3 kN	
$V_{Rd}=n•F_{v.Rd}/ \sqrt{((1+a•n)^2+(ß•n)^2)}$	(2 • 60.29) / √((1+0.000•2)² + (0.926•2)²)	57.3 kN	OK

ii) Fin plate bearing

Vert P_1, e_1, P_2, e_2, a_b, k_1	54.0, 32.0, 0.0, 50.0, 0.593, 2.500		
$F_{b,Rd}=k_1• a_b•f_{up}•d•t_p/ \gamma_{M2}$	2.5 • 0.593 • 410 • 16 • 10.0 / 1.25	77.7 kN	
Horz P_1, e_1, P_2, e_2, a_b, k_1	0.0, 50.0, 54.0, 32.0, 0.926, 2.500		
$F_{b,Rd}=k_1• a_b•f_{up}•d•t_p/ \gamma_{M2}$	2.5 • 0.926 • 410 • 16 • 10.0 / 1.25	121.5 kN	
$V_{.Rd}=N / \sqrt{(((1+a•N) / F_{b.vert.Rd})^2}$	2.00 / √(((1 + 0.00 • 2.00) / 77.75)²		
$+(ß•N / F_{b.hor.Rd})^2)$	+ (0.93 • 2.00) / 121.48)²)	100.3 kN	OK

iii) Beam Web bearing

Vert P_1, e_1, P_2, e_2, a_b, k_1	54.0, 49.0, 0.0, 40.0, 0.750, 2.500		
$F_{b,Rd}=k_1• a_b•f_{ub}•d•t_p/ \gamma_{M2}$	2.5 • 0.750 • 410 • 16 • 4.5 / 1.25	44.3 kN	
Horz P_1, e_1, P_2, e_2, a_b, k_1	0.0, 40.0, 54.0, 49.0, 0.741, 2.500		
$F_{b,Rd}=k_1• a_b•f_{ub}•d•t_p/ \gamma_{M2}$	2.5 • 0.741 • 410 • 16 • 4.5 / 1.25	43.7 kN	
$V_{.Rd}=N / \sqrt{(((1+a•N) / F_{b.vert.Rd})^2}$	2.00 / √(((1 + 0.00 • 2.00) / 44.28)²		
$+(ß•N / F_{b.hor.Rd})^2)$	+ (0.93 • 2.00) / 43.73)²)	41.7 kN	OK

Check 3:Supported Beam - Connecting Elements

i) Shear

$V_{Rd.g}=(h_p•t_p/ 1.27)•(f_{yp}/ \sqrt{3}/\gamma_{M0})$	(118•10/1.27) • (275/√3/1.0)	147.5 kN	OK
$V_{Rd.n}=A_{v.Net} •f_{up}/\sqrt{3}/\gamma_{M2net}$	820 • 410/√3/1.1	176.5 kN	OK
$V_{Rd.b}=(0.5•f_{up}•A_{nt} /\gamma_{M2net})$	(0.5 • 410 • 410 / 1.1)		
$+(F_{yp}•A_{nv} /\sqrt{3}/\gamma_{M0})$	+ (275 • 590 / √3/1.0)	170.1 kN	OK

ii) Bending

$V_{Rd}= (W_{el.P} / z)•(F_{yp}/ \gamma_{M0})$	(23207 / 50) • (275 / 1.0)	127.6 kN	OK

iii) Lateral torsional buckling

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$V_{Rd}=(W_{el.P} / z)•(F_{yP}/ \gamma_{M0})$	(23207 / 50) • (275 / 1.0)	127.6 kN	OK

Check 4: Supported Beam - Resistance at the connection

i) Shear

$V_{Rd.G}=A_{v.wb}•f_{y.b1} / \sqrt{3}/ \gamma_{M0}$	817.71 • 275.00/√3/1.0	129.8 kN	OK
$V_{Rd.N}=A_{v.wb.net}•f_{u.b1} / \sqrt{3} / \gamma_{M2}$	655.71 • 410.00/ √3/1.1	141.1 kN	OK

iii) Shear & Bending Interaction of the Beam Web

data $V_{AB.Rd}$, $V_{BC.Rd}$, $V_{BC.Ed}$	29, 38.58, 0.78		
$M_{c.BC.Rd}=f_y•t_w/6/\gamma_{M0}•((n_1-1)•p_1)^2$	(275.00 • 4.50 / 6 / 1.0) •(2-1) • 54)²	0.6 kN.m	
$M_{c.Ed}=V_{ed}•Z_p$	2.21 • 50.00	0.11 kN.m	
$M_{c.Rd}=M_{c.BC.Rd}+V_{AB.Rd}•(n_1-1)•p_1$	0.60 + 28.58 •(2-1)•54	2.14 kN.m	OK

Check 8: Supporting Member Welds

$a=0.7s >= 0.5\ t_p$	5.6 = 0.7 • 8 >= 0.5 • 10.0		OK

Check 10: Supporting Member Local Resistance

i) Local Shear

$F_{Rd}=2•h_p•t_p•F_{yp}/(\sqrt{3}•\gamma_{M0})$	2 • 118 • 6.3 • 275/(√3 • 1.0)	236.1 kN	OK

ii) Punching Shear

$t_{2•f\_}(u.2)•h_p^2 /(V_{Ed}•6•z_p•\gamma_{M2}) > 1$	6.3 • 410 • 118² / (2.2 • 6 • 50 • 1.25)	43.398>1	OK

Tie Forces

Applied Tie Force	75.0 kN		

Check 11: Structural integrity - Fin Plate

i) Tension

Part Block Tension	Only 1 Shear Plane		
$F_{Rd.b}=f_{u.p}•A_{nt}/ \gamma_{Mu}$	410 • 590 / 1.1		
$+ f_{y.p}•A_{nv} / \sqrt{3} / \gamma_{M0}$	+ 275 • 410 / 1.73 • 1.0	285.0 kN	OK
Full Block Tension	Only 2 Shear Planes		
$F_{Rd.b}=f_{u.p}•A_{nt}/ \gamma_{Mu}$	410 • 360 / 1.1		
$+ f_{y.p}•A_{nv} / \sqrt{3} / \gamma_{M0}$	+ 275 • 820 / 1.73 • 1.0	264.4 kN	OK
Full Depth Tension	No Shear Planes		
$F_{Rd.n}=0.9•F_{u.p}•A_{net.p} / \gamma_{Mu}$	0.9 • 410 • 820 / 1.1	275.1 kN	OK

ii) Shear

$F_{v,Rd}=a_v•f_{ub}•A / \gamma_{Mu}$	0.6 • 800 • 157.0 / 1.1	68.5 kN	
$F_{Rd}=n•F_{v,Rd}$	2 • 68.5	137.0 kN	OK

iii) Bearing

P_1, e_1, P_2, e_2, a_b, k_1	0.0, 50.0, 54.0, 32.0, 0.926, 2.500		
$F_{b,Rd}=k_1•\ a_b•f_{up}•d•t_p/ \gamma_{Mu}$	2.5 • 0.926 • 410 • 16 • 10.0 / 1.1	138.0 kN	
$F_{Rd}=n•F_{b,Rd}$	2 • 138.0	276.1 kN	OK

Check 12: Structural integrity - Supported Beam

i) Tension

Part Block Tension	Only 1 Shear Plane		
$F_{Rd.b}=F_{u.}•A_{nt}/ \gamma_{Mu}$	410 • 206 / 1.1		
$+ F_{y.B1}•A_{nv} / \sqrt{3} / \gamma_{M0}$	+ 275 • 140 / 1.73 • 1.0	99.0 kN	OK
Full Block Tension	Only 2 Shear Planes		
$F_{Rd.b}=F_{u.}•A_{nt}/ \gamma_{Mu}$	410 • 162 / 1.1		
$+ F_{y.B1}•A_{nv} / \sqrt{3} / \gamma_{M0}$	+ 275 • 279 / 1.73 • 1.0	104.7 kN	OK
Full Depth Tension	No Shear Planes		
$F_{Rd.n}=0.9•F_{u.p}•A_{net.p} / \gamma_{Mu}$	0.9 • 410 • 524 / 1.1	175.7 kN	OK

ii) Bearing

P_1, e_1, P_2, e_2, a_b, k_1	0.0, 40.0, 54.0, 32.0, 0.741, 2.500		
$F_{b,Rd}=k_1•\ a_b•f_{ub}•d•t_p/ \gamma_{Mu}$	2.5 • 0.741 • 410 • 16 • 4.5 / 1.1	49.7 kN	
$F_{Rd}=n•F_{b,Rd}$	2 • 49.7	99.4 kN	OK

Check 15: Structural integrity - Supporting Column Wall - RHS

Web Tie Force Applied	75.0 kN		
$M_{pl.Rd.u}=1/4•f_{u.c}•t_{w.c}^2$	1/4 • 410 • 6.3²	4.1 kN	
$F_{Rd}=8•M_{pl.Rd.u} / \gamma_{Mu} / (1-ß)$	8 • 4.1 /1.1 / (1 - 0.26)		
•(?+1.5•v(1-ß))	• 1.2 + 1.5 v(1 -0.26)	98.0 kN	OK

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Checks for Web Beam on Side 2

Check 1: Recommended Detailing Practice

Plate Depth >= 0.6•D	0.6 • 152.4 = 91.4 mm	118 mm	OK
Plate thickness check	min(tfin 10.0/0.5, tweb 4.5/0.5) / F	0.56 < 1	OK

Check 2: Supported beam - Bolt Group

i) Bolts in Shear

$\beta=(6•z) / (n_1•(n_1+1)•p1)$	(6•50) / (2 • (2 + 1)• 54)	0.926	
$F_{v,Rd}=\alpha_v•f_{ub}•A / \gamma_{M2}$	0.6 • 800 • 157.0 / 1.25	60.3 kN	
$V_{Rd}=n•F_{v.Rd}/ \sqrt{((1+a•n)^2+(\beta•n)^2)}$	(2 • 60.29) / √((1+0.000•2)² + (0.926•2)²)	57.3 kN	OK

ii) Fin plate bearing

Vert P_1, e_1, P_2, e_2, α_b, k_1	54.0, 32.0, 0.0, 50.0, 0.593, 2.500		
$F_{b,Rd}=k_1• \alpha_b•f_{up}•d•t_p/ \gamma_{M2}$	2.5 • 0.593 • 410 • 16 • 10.0 / 1.25	77.7 kN	
Horz P_1, e_1, P_2, e_2, α_b, k_1	0.0, 50.0, 54.0, 32.0, 0.926, 2.500		
$F_{b,Rd}=k_1• \alpha_b•f_{up}•d•t_p/ \gamma_{M2}$	2.5 • 0.926 • 410 • 16 • 10.0 / 1.25	121.5 kN	
$V_{.Rd}=N / \sqrt{(((1+a•N) / F_{b.vert.Rd})^2}$	2.00 / √(((1 + 0.00 • 2.00) / 77.75)²		
$+(\beta•N / F_{b.hor.Rd})^2)$	+ (0.93 • 2.00) / 121.48)²)	100.3 kN	OK

iii) Beam Web bearing

Vert P_1, e_1, P_2, e_2, α_b, k_1	54.0, 49.0, 0.0, 40.0, 0.750, 2.500		
$F_{b,Rd}=k_1• \alpha_b•f_{ub}•d•t_p/ \gamma_{M2}$	2.5 • 0.750 • 410 • 16 • 4.5 / 1.25	44.3 kN	
Horz P_1, e_1, P_2, e_2, α_b, k_1	0.0, 40.0, 54.0, 49.0, 0.741, 2.500		
$F_{b,Rd}=k_1• \alpha_b•f_{ub}•d•t_p/ \gamma_{M2}$	2.5 • 0.741 • 410 • 16 • 4.5 / 1.25	43.7 kN	
$V_{.Rd}=N / \sqrt{(((1+a•N) / F_{b.vert.Rd})^2}$	2.00 / √(((1 + 0.00 • 2.00) / 44.28)²		
$+(\beta•N / F_{b.hor.Rd})^2)$	+ (0.93 • 2.00) / 43.73)²)	41.7 kN	OK

Check 3:Supported Beam - Connecting Elements

i) Shear

$V_{Rd.g}=(h_p•t_p/ 1.27)•(f_{yp}/ \sqrt{3}/\gamma_{M0})$	(118•10/1.27) • (275/√3/1.0)	147.5 kN	OK
$V_{Rd.n}=A_{v.Net} •f_{up}/\sqrt{3}/\gamma_{M2net}$	820 • 410/√3/1.1	176.5 kN	OK
$V_{Rd.b}=(0.5•f_{up}•A_{nt} /\gamma_{M2net})$	(0.5 • 410 • 410 / 1.1)		
$+(F_{yp}•A_{nv} /\sqrt{3}/\gamma_{M0})$	+ (275 • 590 / √3/1.0)	170.1 kN	OK

ii) Bending

$V_{Rd}= (W_{el.P} / z)•(F_{yp}/ \gamma_{M0})$	(23207 / 50) • (275 / 1.0)	127.6 kN	OK

iii) Lateral torsional buckling

$V_{Rd}=(W_{el.P} / z)•(F_{yP}/ \gamma_{M0})$	(23207 / 50) • (275 / 1.0)	127.6 kN	OK

Check 4: Supported Beam - Resistance at the connection

i) Shear

$V_{Rd.G}=A_{v.wb}•f_{y.b1} / \sqrt{3}/ \gamma_{M0}$	817.71 • 275.00/√3/1.0	129.8 kN	OK
$V_{Rd.N}=A_{v.wb.net}•f_{u.b1} / \sqrt{3} / \gamma_{M2}$	655.71 • 410.00/ √3/1.1	141.1 kN	OK

iii) Shear & Bending Interaction of the Beam Web

data $V_{AB.Rd}$, $V_{BC.Rd}$, $V_{BC.Ed}$	29, 38.58, 0.04		
$M_{c.BC.Rd}=f_y•t_w/6/\gamma_{M0} •((n_1-1)•p_1)^2$	(275.00 • 4.50 / 6 / 1.0) •(2-1) • 54)²	0.6 kN.m	
$M_{c.Ed}=V_{ed}•Z_p$	0.11 • 50.00	0.01 kN.m	
$M_{c.Rd}=M_{c.BC.Rd}+V_{AB.Rd}•(n_1-1)•p_1$	0.60 + 28.58 •(2-1)•54	2.14 kN.m	OK

Check 8: Supporting Member Welds

$a=0.7s >= 0.5\ t_p$	5.6 = 0.7 • 8 >= 0.5 • 10.0		OK

Check 10: Supporting Member Local Resistance

i) Local Shear

$F_{Rd}=2•h_p•t_p•F_{yp}/(\sqrt{3}•\gamma_{M0})$	2 • 118 • 6.3 • 275/(√3 • 1.0)	236.1 kN	OK

ii) Punching Shear

$t_{2•f}(u.2)•h_p^2 /(V_{Ed}•6•z_p•\gamma_{M2}) > 1$	6.3 • 410 • 118² / (0.1 • 6 • 50 • 1.25)	902.644>1	OK

Tie Forces

Applied Tie Force	75.0 kN		

Check 11: Structural integrity - Fin Plate

i) Tension

Part Block Tension	Only 1 Shear Plane		
$F_{Rd.b}=f_{u.p}•A_{nt}/ \gamma_{Mu}$	410 • 590 / 1.1		
$+ f_{y.p}•A_{nv} / \sqrt{3} / \gamma_{M0}$	+ 275 • 410 / 1.73 • 1.0	285.0 kN	OK
Full Block Tension	Only 2 Shear Planes		
$F_{Rd.b}=f_{u.p}•A_{nt}/ \gamma_{Mu}$	410 • 360 / 1.1		

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+ $f_{y.p} \bullet A_{nv} / \sqrt{3} / \gamma_{M0}$	+ 275 • 820 / 1.73 • 1.0	264.4 kN	OK
Full Depth Tension	No Shear Planes		
$F_{Rd.n}=0.9 \bullet F_{u.p} \bullet A_{net.p} / \gamma_{Mu}$	0.9 • 410 • 820 / 1.1	275.1 kN	OK

ii) Shear

$F_{v,Rd}=a_v \bullet f_{ub} \bullet A / \gamma_{Mu}$	0.6 • 800 • 157.0 / 1.1	68.5 kN	
$F_{Rd}=n \bullet F_{v,Rd}$	2 • 68.5	137.0 kN	OK

iii) Bearing

P_1, e_1, P_2, e_2, a_b, k_1	0.0, 50.0, 54.0, 32.0, 0.926, 2.500		
$F_{b,Rd}=k_1 \bullet a_b \bullet f_{up} \bullet d \bullet t_p / \gamma_{Mu}$	2.5 • 0.926 • 410 • 16 • 10.0 / 1.1	138.0 kN	
$F_{Rd}=n \bullet F_{b,Rd}$	2 • 138.0	276.1 kN	OK

Check 12: Structural integrity - Supported Beam

i) Tension

Part Block Tension	Only 1 Shear Plane		
$F_{Rd.b}=F_{u.} \bullet A_{nt} / \gamma_{Mu}$	410 • 206 / 1.1		
+ $F_{y.B1} \bullet A_{nv} / \sqrt{3} / \gamma_{M0}$	+ 275 • 140 / 1.73 • 1.0	99.0 kN	OK
Full Block Tension	Only 2 Shear Planes		
$F_{Rd.b}=F_{u.} \bullet A_{nt} / \gamma_{Mu}$	410 • 162 / 1.1		
+ $F_{y.B1} \bullet A_{nv} / \sqrt{3} / \gamma_{M0}$	+ 275 • 279 / 1.73 • 1.0	104.7 kN	OK
Full Depth Tension	No Shear Planes		
$F_{Rd.n}=0.9 \bullet F_{u.p} \bullet A_{net.p} / \gamma_{Mu}$	0.9 • 410 • 524 / 1.1	175.7 kN	OK

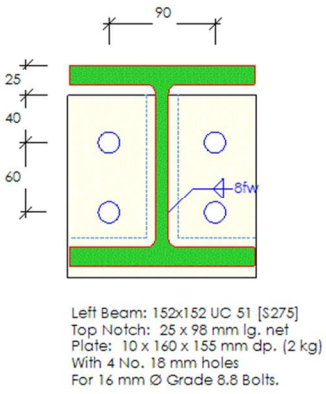
ii) Bearing

P_1, e_1, P_2, e_2, a_b, k_1	0.0, 40.0, 54.0, 32.0, 0.741, 2.500		
$F_{b,Rd}=k_1 \bullet a_b \bullet f_{ub} \bullet d \bullet t_p / \gamma_{Mu}$	2.5 • 0.741 • 410 • 16 • 4.5 / 1.1	49.7 kN	
$F_{Rd}=n \bullet F_{b,Rd}$	2 • 49.7	99.4 kN	OK

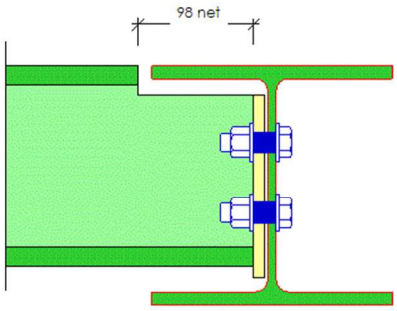
Check 15: Structural integrity - Supporting Column Wall - RHS

Web Tie Force Applied	75.0 kN		
$M_{pl.Rd.u}=1/4 \bullet f_{u.c} \bullet t_{w.c}^2$	1/4 • 410 • 6.3^2	4.1 kN	
$F_{Rd}=8 \bullet M_{pl.Rd.u} / \gamma_{Mu} / (1-ß)$	8 • 4.1 /1.1 / (1 - 0.26)		
•(?+1.5•v(1-ß))	• 1.2 + 1.5 v(1 -0.26)	98.0 kN	OK

BEAM TO BEAM: FLEXIBLE END-PLATE CONNECTION AT END 2 OF MEMBER 12



Left Beam: 152x152 UC 51 [S275]
Top Notch: 25 x 98 mm lg. net
Plate: 10 x 160 x 155 mm dp. (2 kg)
With 4 No. 18 mm holes
For 16 mm Ø Grade 8.8 Bolts.



Beam: 203x203 UC 46 [S275]
End-Plates S 275

Beam to Beam Flexible End-plate Connection to EC 3 (UK NAD)
Basic Data

Integrated Applied Forces at Interface

Shear Forces	Left = 62.3 kN, Right = 0.0 kN
Tie Force	75.0 kN
Design to	EC 3: Part 1-8: 2005 Design of Joints
SCI Green Book	P358: Joints in steel construction: Simple joints to Eurocode 3

Basic Dimensions

Left-152x152UC51 [28]	D=170.2, B=157.4, T=15.7, t=11.0, r=7.6, py=275
Supporting-203x203UC46 [28]	D=203.2, B=203.6, T=11.0, t=7.2, r=10.2, py=275
Bolts 16 mm Ø in 18 mm holes	Grade 8.8 Bolts

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Plates S 275 All weld grades provided to suit minimum connected steel grade

Summary of Results (Unity Ratios)

Left Hand Beam

Check 4 Supported Beam Web Shear	227.9 >= 62.3kN	0.27	OK
Checks 1 & 2 Detailing Practice	1.00, 0.83, 1.00, 0.64, 0.540.79	1.00	OK
Check 5: Notch Resistance	0.41	0.41	OK
Check 6: Notch Stability	0.59	0.59	OK
Checks 8 & 9 Bearing & Shear	192.9, 388.7, 325.4, 512.2, 473.3 >= 62.3kN	0.32	OK
Check 10 Shear	194.3, 207.6 >= 31.2kN	0.16	OK
Check 11-14 Tie Forces	186.2, 249.5, 411.1, 569.1 >= 75.0kN	0.40	OK

Left Hand Beam

Check 1: Recommended Detailing Practice

Plate Depth >= 0.6•D	0.6 • 170.2 = 102.1 mm	155 mm	OK
10 >= t_p<= 12	10.0 mm		OK
90 <= p_3 <= 140	90 mm		OK

Check 4: Supported Beam Web Shear

$V_{pl\_Rd}=0.9•t_{bw}•h_{pw}•f_{ybw}$ / (√3•γ_{M0})	0.9 • 11.0 • 145 • 275 / (√3 • 1.0)	227.9 kN	OK

Check 2: Supported Beam Welds

a=0.4 t_{wb1}	0.4 • 11.0	4.4 mm	
a_1=a•V_{Ed}/ $V_{pl.Rd}$•1.27	4.4 • 62.3 / 227.9 • 1.27 (SCI AD370)	1.5 mm	
S_{min}=max(3, min(a, a_1)) / 0.7		4.3 mm	OK

Check 5: Supported Beam Notch Resistance

$V_{Rd}=A_v.N•f_{ybw}$ /(√3•γ_{M0})	1643 • 275 / (√3 • 1.0)	260.8 kN	Low Shear
$M_{v.Rd}=f_{y.b1}•W_{el.N.y}$/ γ_{M0}	275 • 61/1.0	16.9 kN.m	
V_{Rd}=Min(V_{Rd} , $M_{v.rd}$/ La)	Min(260.79, 16.89 / 110)	153.5 kN	OK

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

D_{nt}<= $h_{\_b1}$/ 2	25.0 <= 170.2/ 2 mm	25.0 <= 85.1	OK
L_n <= h_{b1}		100 <= 170	OK

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

	Beam assumed to be restrained		ignored

Check 8a: Bolt Shear

$F_{v,Rd}=a_v•f_{ub}•A$ / γ_{M2}	0.6 • 800 • 157.0 / 1.25	60.3 kN	
V_{Rd1}=0.8•n•$F_{v,Rd}$•cols	0.8 • 2 • 60.3 • 2	192.9 kN	OK

Check 8b: End-Plate Bearing

P_1, e_1, P_2, e_2, a_b, k_1	60.0, 40.0, 90.0, 35.0, 0.741, 2.500		
$F_{b,Rd}=k_1•a_b•f_{up}•d•t_p$/ γ_{M2}	2.5 • 0.741 • 410 • 16 • 10.0 / 1.25	97.2 kN	
$F_{b,Rd}$<= 0.80•$F_{v,Rd}$			
$F_{Rd}=n•f_{b,Rd•rows}$	2 • 97.2 • 2	388.7 kN	OK

Check 8c: Supporting Beam Web Bearing

a_b=Min(p_1/d_0/3-¼, f_{ub}/ f_u, 1)	Min(60/18/3-¼, 800/410, 1) » Min(0.861, 1.951, 1)	0.861	
$k_{1.inn}$=min(2.5, 1.4•p_2/d_0-1.7)	min(2.5, 1.4 • 90/18 - 1.7 » Min(2.5, 5.3)	2.500	
$F_{b,Rd}=k_1•a_b•f_{u,2}$ •d•t_p/ γ_{M2}	2.5 • 0.861 • 410 • 16.0 • 7.2 / 1.25	81.3 kN	
$F_{b,Rd}$<= 0.80•$F_{v,Rd}$			
$F_{Rd}=n•f_{b,Rd•rows}$	2 • 81.3 • 2	325.4 kN	OK

Check 9a: End-Plate Plane Shear

V_{Rd4}=2• $h_p•t_p•F_{yp}$/(1.27•√3•γ_{M0})	2•155 • 10 • 275/(1.27 • √3 • 1.0)	387.6 kN	OK
V_{Rd}=2• t_p•(h_p-$n_{1•d\_0}$)•F_{up}/(√3•γ_{M2})	2•10•(155 - 2 • 18) • 410/(√3 • 1.1)	512.2 kN	OK

Check 9b: End-Plate Block Shear

A_{nv}, A_{nt}=Fn(t_p, h_p, e_1, e_2, n_1, d_0)	(10, 155, 40, 35, 2, 18)	880, 260 mm²	
V_{Rd}=2•($F_{up}•A_{nt}/\gamma_{M2}$+$f_{yp}•A_{nv}$/(√3•$\gamma_{M0}$))	2•(410 • 260 / 1.1 + 275880 • /(√3 • 1.0)	473.3 kN	OK

Check 10: Supporting Member - Local Resistance

A_v, A_{Vnet}=Fn(e_t, e_b, p_1, n, dia, T)	fn(65, 45, 60, 2, 18.0, 7.2)	1224, 965 mm²	
$V_{Rd}=A_v•F_{yp}$/(√3•γ_{M0})	1224 • 275/(√3 • 1.0)	194.3 kN	OK
$V_{Rd}=A_{vnet•F\_up}$/(√3•γ_{M2})	965 • 410/(√3 • 1.1)	207.6 kN	OK

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Tie Forces

Applied Tie Force	75.0 kN		

Check 11: End-Plate in Bending

$L_{eff}=e_{1A}+(n-1)\bullet p_{1A}+e_{1A}$	40.0 + (2-1) • 60 + 40.6	140.6 mm	
$M_{pl1}=0.25\bullet L_{eff}\bullet t^2\bullet f_{u.p}/ \gamma_{Mu}$	0.25 • 140.6 • 10² • 410 / 1.1	1.31 kN.m	
$F_{Rd.u.1}=(8\bullet n-2\bullet e_w)\bullet m_{pl1}$ / $(2\bullet m \bullet n-e_w\bullet(m+n))$	(8 • 35.0 - 2 • 6.5) • 1 / (2 • 33.2 • 35.0 - 6.5 • (33.2 + 35.0))	186.2 kN	
$F_{Rd.u.2}=(2\bullet M_{pl2}+n\bullet S(k_2\bullet f_{ub}\bullet A / \gamma_{mu}))/(m+n)$	(2•1.31 + 35.00•4•0.90•800•157/1.1)/ (33.2 + 35.0)	249.5 kN	
$F_{rd.u.3}=S\bullet k_2\bullet F_{ub}\bullet A_s/ \gamma_{Mu}$	4 • 0.90•800 • 157/1.1	411.1 kN	
$F_{Rd}=\min(F_{rd.u.1}, F_{rd.u.2}, F_{rd.u.3})$	Min (186.20, 249.50, 411.05)	186.2 kN	OK

Check 12: Beam Web Tension

$F_{Rd}=t_{bw}\bullet h_{peff}\bullet F_{ubw} / \gamma_{Mu}$	11.0 • 139 • 410 / 1.1	569.1 kN	OK

Check 13: Welds

n/a	Covered by Check 2		OK

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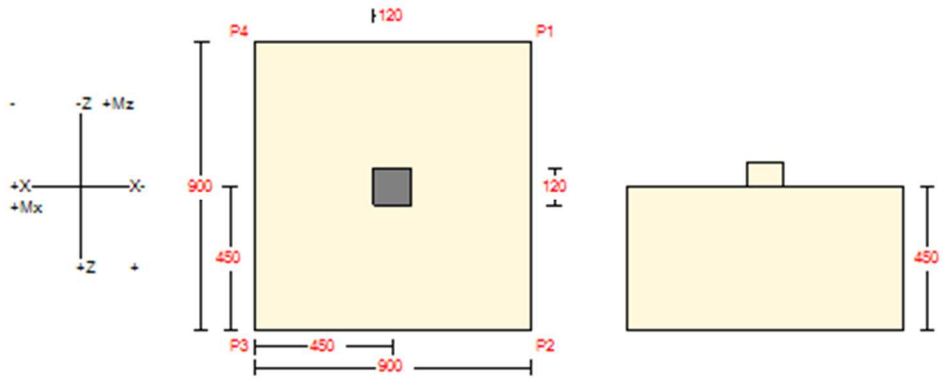

Basic Properties

Design to	EC 2: 2004 - Using UK values	
Fy, Fcu, Covers T, B, S	460 N/mm², 35 N/mm², 50 mm, 50 mm, 50 mm	
Gross: Area, Area1, Z zz, Z xx	0.81, 0.014, 0.122, 0.122	
Conc Den, LFsrv , LFult	23.4, 1.0, 1.0	
Surcharge = Sur_{ext} + h_0 • γ_{soil}	0.0 = 0.0 + 0.0 • 18.0	
SWP = SWP_0 + γ_{soil}• (h_0+ D)	83 = 75 + 18 x (0.000 + 0.450)	

Mass Concrete Pad Design

x-x projections	390, 390	450 mm	OK
z-z projections	390, 390	450 mm	OK

Critical Serviceability : 8 : Dead Plus Wind

Fpad = Den•d•Area•LF	23.4 x 0.45 x 0.81 x 1.00	8.5 kN	
Fcol = F	1.5 +	1.5 kN	
Fres = F + Fpad	1.5 + 8.5	10.0 kN	
Mxx res = Mxx + Vz•D + Fcol•exx	1.1 + (2.0 x 0.45) + (1.5 x 0.0)	2.0 kN.m	
Effective B (Be) = 3•(B/2 -(Mxx/Fres))	3 x (450 - (2.0 / 10.0))	763 mm	
Reduced Properties A, Zd, Zb	0.687, 0.103, 0.087		

Pressure

Mxx res = Mxx res - (B-Be)/2 • Fres	2.0 - (0.9-0.763)/2 x 10.0	1.3 kN.m	
Pmax = Fn(Pa, Pzz, Pxx, p1-4)	14.6, ±0.0, ±14.6, 29.2, 0.0, 0.0, 29.2	29.2 kN/m²	OK
Check for up-lift	Le 900 >=900 Be 763 < 900		Caution

FOS Overturning

Mxx Rest = (F)•e+(pad)•L/2	(2) x 0.450 + (9) x 0.450	5 kN.m	
FOS OT xx = Mxx Rest / Mxx ot	5 / 2	2.30 > 1.5	OK

FOS Sliding

FOS Sliding = 0.30 • F / Fv	(0.30 x 10) / 2	1.52 > 1.5	OK

Combined Axial & Horizontal loads

F/P_v+ Fv/P_h<1 (BS 8004: 2.3.2.4.7)	10.0 / 67.2 + 2.0 / 3.0 = 0.15 + 0.66	0.81	OK

Uplift Design

Uplift detected.	0.372 m of Negative Pressure in Case 5 : Dead Plus Wind Design Top Steel to resist (SelfWeight + Surcharge) x 1.4	
Load W = (d x den + sur) x 1.4	(450x23.4 + 0.0) x 1.4	14.74 kN/m²

Z-Z Axis Section Capacities in Reversal

No Top Steel. Use Fcu Ten =	Min(2, 0.05 x Fcu) = Min(2, 0.05 x 35)	1.75 N/mm²
Z zz = B x D² / 6	900 x 450² / 6	0.03m³
Mu zz = Z x Fcu ten	0.03 x 1.75 x 1000	53.16 kN.m

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X-X Axis Section Capacities in Reversal

No Top Steel. Use Fcu Ten =	Min(2, 0.05 x Fcu) = Min(2, 0.05 x 35)	1.75 N/mm²	
Z xx = B x D² / 6	900 x 450² / 6	0.03m³	
Mu xx = Z x Fcu ten	0.03 x 1.75 x 1000	53.16 kN.m	

Moments at Column Face

M zz = Fn(W,Wd,Lmax)	14.74, 900, 390	1.01 kN.m	OK
M xx = Fn(W,Wd,Lmax)	14.74, 900, 390	1.01 kN.m	OK

Shear at Column Face

v zz = Fn(W,Lmax,d)	14.74, 390, 400	0.01 N/mm²	OK
v xx = Fn(W,Lmax,d)	14.74, 390, 400	0.01 N/mm²	OK

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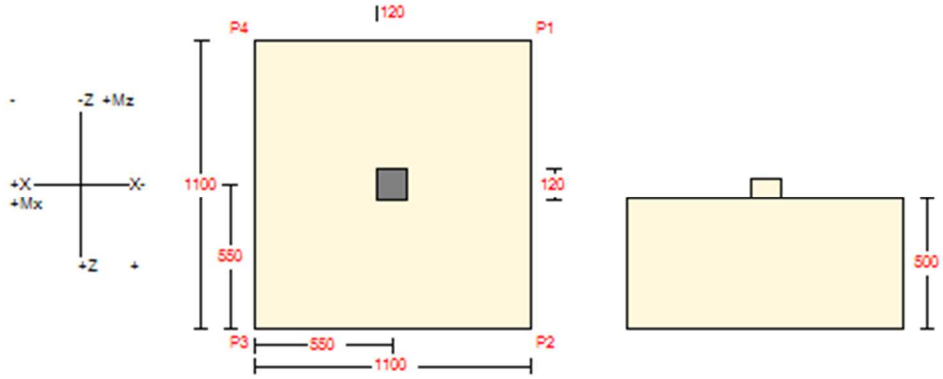

Basic Properties

Design to	EC 2: 2004 - Using UK values
Fy, Fcu, Covers T, B, S	460 N/mm², 35 N/mm², 50 mm, 50 mm, 50 mm
Gross: Area, Area1, Z zz, Z xx	1.21, 0.014, 0.222, 0.222
Conc Den, LFsrv , LFult	23.4, 1.0, 1.0
Surcharge = Sur_{ext} + h_0 • γ_{soil}	0.0 = 0.0 + 0.0 • 18.0
SWP = SWP_0 + γ_{soil}• (h_0+ D)	84 = 75 + 18 x (0.000 + 0.500)

Mass Concrete Pad Design

x-x projections	490, 490	500 mm	OK
z-z projections	490, 490	500 mm	OK

Critical Serviceability : 8 : Dead Plus Wind

Fpad = Den•d•Area•LF	23.4 x 0.5 x 1.21 x 1.00	14.2 kN	
Fcol = F	2.1 +	2.1 kN	
Fres = F + Fpad	2.1 + 14.2	16.2 kN	
Mxx res = Mxx + Vz•D + Fcol•exx	1.7 + (3.2 x 0.5) + (2.1 x 0.0)	3.3 kN.m	
Effective B (Be) = 3•(B/2 -(Mxx/Fres))	3 x (550 - (3.3 / 16.2))	1038 mm	
Reduced Properties A, Zd, Zb	1.141, 0.209, 0.197		

Pressure

Mxx res = Mxx res - (B-Be)/2 • Fres	3.3 - (1.1-1.038)/2 x 16.2	2.8 kN.m	
Pmax = Fn(Pa, Pzz, Pxx, p1-4)	14.2, ±0.0, ±14.2, 28.4, 0.0, 0.0, 28.4	28.4 kN/m²	OK
Check for up-lift	Le 1100 >=1100 Be 1038 < 1100		Caution

FOS Overturning

Mxx Rest = (F)•e+(pad)•L/2	(2) x 0.550 + (14) x 0.550	9 kN.m	
FOS OT xx = Mxx Rest / Mxx ot	9 / 3	2.69 > 1.5	OK

FOS Sliding

FOS Sliding = 0.30 • F / Fv	(0.30 x 16) / 3	1.53 > 1.5	OK

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Combined Axial & Horizontal loads

$F/P_v + Fv/P_h < 1$ (BS 8004: 2.3.2.4.7)	16.2 / 101.6 + 3.2 / 4.9 = 0.16 + 0.65	0.81	OK

Uplift Design

Uplift detected.	0.307 m of Negative Pressure in Case 5 : Dead Plus Wind Design Top Steel to resist (SelfWeight + Surcharge) x 1.4		
Load W = (d x den + sur) x 1.4	(500x23.4 + 0.0) x 1.4	16.38 kN/m²	

Z-Z Axis Section Capacities in Reversal

No Top Steel. Use Fcu Ten =	Min(2, 0.05 x Fcu) = Min(2, 0.05 x 35)	1.75 N/mm²	
Z zz = B x D² / 6	1100 x 500² / 6	0.05m³	
Mu zz = Z x Fcu ten	0.05 x 1.75 x 1000	80.21 kN.m	

X-X Axis Section Capacities in Reversal

No Top Steel. Use Fcu Ten =	Min(2, 0.05 x Fcu) = Min(2, 0.05 x 35)	1.75 N/mm²	
Z xx = B x D² / 6	1100 x 500² / 6	0.05m³	
Mu xx = Z x Fcu ten	0.05 x 1.75 x 1000	80.21 kN.m	

Moments at Column Face

M zz = Fn(W,Wd,Lmax)	16.38, 1100, 490	2.16 kN.m	OK
M xx = Fn(W,Wd,Lmax)	16.38, 1100, 490	2.16 kN.m	OK

Shear at Column Face

v zz = Fn(W,Lmax,d)	16.38, 490, 450	0.02 N/mm²	OK
v xx = Fn(W,Lmax,d)	16.38, 490, 450	0.02 N/mm²	OK

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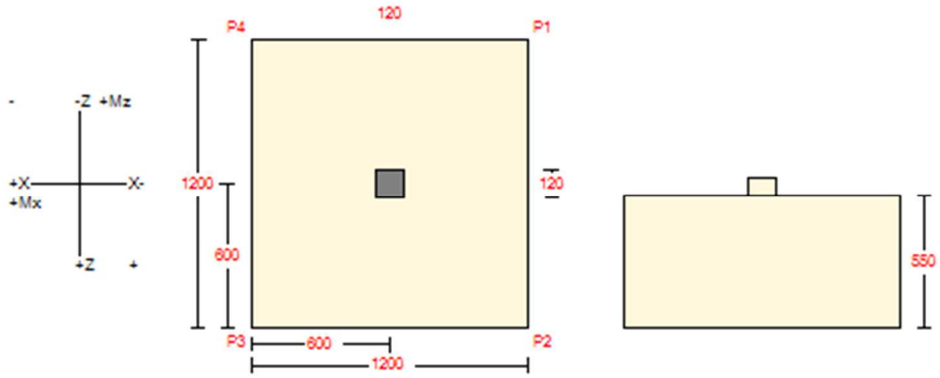

Basic Properties

Design to	EC 2: 2004 - Using UK values
Fy, Fcu, Covers T, B, S	460 N/mm², 35 N/mm², 50 mm, 50 mm, 50 mm
Gross: Area, Area1, Z zz, Z xx	1.44, 0.014, 0.288, 0.288
Conc Den, LFsrv , LFult	23.4, 1.0, 1.0
Surcharge = Sur_{ext} + h_0 • γ_{soil}	0.0 = 0.0 + 0.0 • 18.0
SWP = SWP_0 + γ_{soil}• (h_0+ D)	85 = 75 + 18 x (0.000 + 0.550)

Mass Concrete Pad Design

x-x projections	540, 540	550 mm	OK
z-z projections	540, 540	550 mm	OK

Critical Serviceability : 7 : Dead Plus Live Plus Wind

Fpad = Den•d•Area•LF	23.4 x 0.55 x 1.44 x 1.00	18.5 kN	
Fcol = F	74.7 +	74.7 kN	
Fres = F + Fpad	74.7 + 18.5	93.2 kN	
Mxx res = Mxx + Vz•D + Fcol•exx	1.5 + (2.8 x 0.55) + (74.7 x 0.0)	3.1 kN.m	
Effective B (Be) = Fn(Mzz,Fres,B)	3.1, 93.2, 1200	1200 mm	

Pressure

Pmax = Fn(Pa, Pzz, Pxx, p1-4)	64.7, ±0.0, ±10.6, 75.3, 54.1, 54.1, 75.3	75.3 kN/m²	OK

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Check for up-lift	Le 1200 >=1200 Be 1200 >=1200		OK

FOS Overturning

Mxx Rest = (F)•e+(pad)•L/2	(75) x 0.600 + (19) x 0.600	56 kN.m	
FOS OT xx = Mxx Rest / Mxx ot	56 / 3	18.31 > 1.5	OK

FOS Sliding

FOS Sliding = 0.30 • F / Fv	(0.30 x 93) / 3	10.02 > 1.5	OK

Combined Axial & Horizontal loads

$F/P_v + Fv/P_h < 1$ (BS 8004: 2.3.2.4.7)	93.2 / 122.4 + 2.8 / 28.0 = 0.76 + 0.10	0.86	OK

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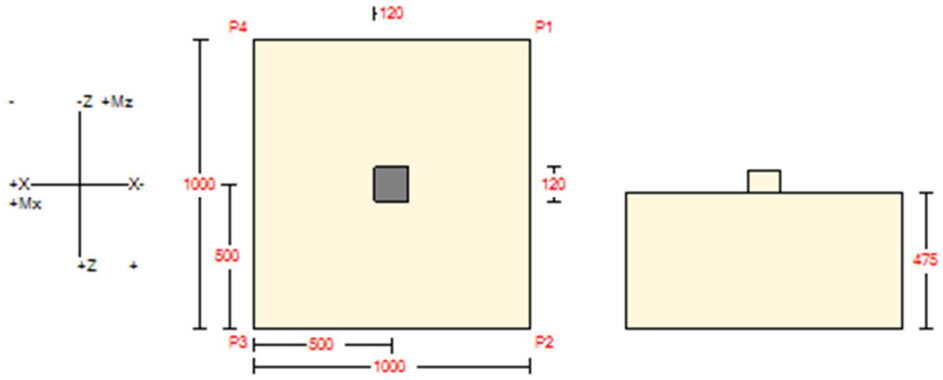

Basic Properties

Design to	EC 2: 2004 - Using UK values
Fy, Fcu, Covers T, B, S	460 N/mm², 35 N/mm², 50 mm, 50 mm, 50 mm
Gross: Area, Area1, Z zz, Z xx	1.0, 0.014, 0.167, 0.167
Conc Den, LFsrv , LFult	23.4, 1.0, 1.0
Surcharge = Sur_{ext} + h_0 • γ_{soil}	0.0 = 0.0 + 0.0 • 18.0
SWP = SWP_0 + γ_{soil}• (h_0+ D)	84 = 75 + 18 x (0.000 + 0.475)

Mass Concrete Pad Design

x-x projections	440, 440	475 mm	OK
z-z projections	440, 440	475 mm	OK

Critical Serviceability : 8 : Dead Plus Wind

Fpad = Den•d•Area•LF	23.4 x 0.475 x 1.0 x 1.00	11.1 kN	
Fcol = F	1.6 +	1.6 kN	
Fres = F + Fpad	1.6 + 11.1	12.7 kN	
Mxx res = Mxx + Vz•D + Fcol•exx	1.3 + (2.3 x 0.475) + (1.6 x 0.0)	2.4 kN.m	
Effective B (Be) = 3•(B/2 -(Mxx/Fres))	3 x (500 - (2.4 / 12.7))	937 mm	
Reduced Properties A, Zd, Zb	0.937, 0.156, 0.146		

Pressure

Mxx res = Mxx res - (B-Be)/2 • Fres	2.4 - (1.0-0.937)/2 x 12.7	2.0 kN.m	
Pmax = Fn(Pa, Pzz, Pxx, p1-4)	13.5, ±0.0, ±13.5, 27.0, 0.0, 0.0, 27.0	27.0 kN/m²	OK
Check for up-lift	Le 1000 >=1000 Be 937 < 1000		Caution

FOS Overturning

Mxx Rest = (F)•e+(pad)•L/2	(2) x 0.500 + (11) x 0.500	6 kN.m	
FOS OT xx = Mxx Rest / Mxx ot	6 / 2	2.67 > 1.5	OK

FOS Sliding

FOS Sliding = 0.30 • F / Fv	(0.30 x 13) / 2	1.63 > 1.5	OK

Combined Axial & Horizontal loads

$F/P_v + Fv/P_h < 1$ (BS 8004: 2.3.2.4.7)	12.7 / 84.0 + 2.3 / 3.8 = 0.15 + 0.61	0.76	OK

MasterSeries User Company

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Job ref	: **Job Ref**
Sheet	: **Sheet Ref / 27 -**
Made By	:
Date	**: 26 Feb 2018/ Version 2017.14**
Checked	**:**
Approved	:

Uplift Design

Uplift detected.	0.285 m of Negative Pressure in Case 5 : Dead Plus Wind Design Top Steel to resist (SelfWeight + Surcharge) x 1.4		
Load W = (d x den + sur) x 1.4	(475x23.4 + 0.0) x 1.4	15.56 kN/m²	

Z-Z Axis Section Capacities in Reversal

No Top Steel. Use Fcu Ten =	Min(2, 0.05 x Fcu) = Min(2, 0.05 x 35)	1.75 N/mm²	
Z zz = B x D² / 6	1000 x 475² / 6	0.04m³	
Mu zz = Z x Fcu ten	0.04 x 1.75 x 1000	65.81 kN.m	

X-X Axis Section Capacities in Reversal

No Top Steel. Use Fcu Ten =	Min(2, 0.05 x Fcu) = Min(2, 0.05 x 35)	1.75 N/mm²	
Z xx = B x D² / 6	1000 x 475² / 6	0.04m³	
Mu xx = Z x Fcu ten	0.04 x 1.75 x 1000	65.81 kN.m	

Moments at Column Face

M zz = Fn(W,Wd,Lmax)	15.56, 1000, 440	1.51 kN.m	OK
M xx = Fn(W,Wd,Lmax)	15.56, 1000, 440	1.51 kN.m	OK

Shear at Column Face

v zz = Fn(W,Lmax,d)	15.56, 440, 425	0.02 N/mm²	OK
v xx = Fn(W,Lmax,d)	15.56, 440, 425	0.02 N/mm²	OK