MasterSeries Sales Team
3 Castle Street
Carrickfergus
Co. Antrim BT38 7BE

Tel : 028 9036 5950

Job ref	: **Job Ref**
Sheet	: **Sheet Ref / 1 -**
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STRUCTURAL CALCULATIONS

Using

MASTERSERIES STEEL DESIGN

STRUCTURAL ENGINEERS

MASTERSERIES SALES TEAM

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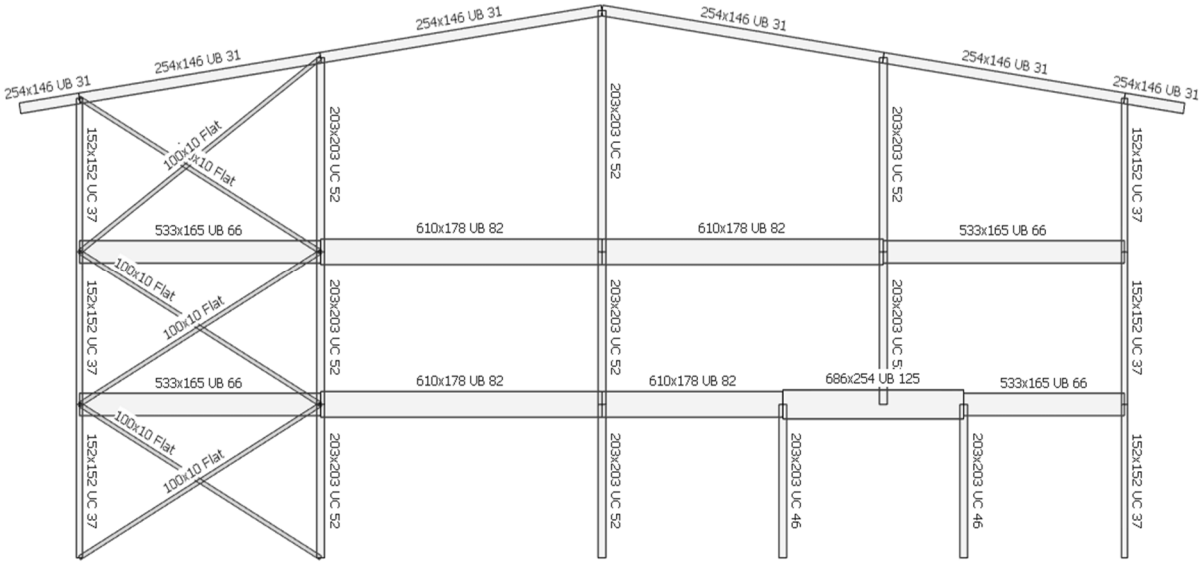

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Job ref	: **Job Ref**
Sheet	: **Sheet Ref / 2 -**
Made By	:
Date	**: 21 June 2015/ Version 2015.04**
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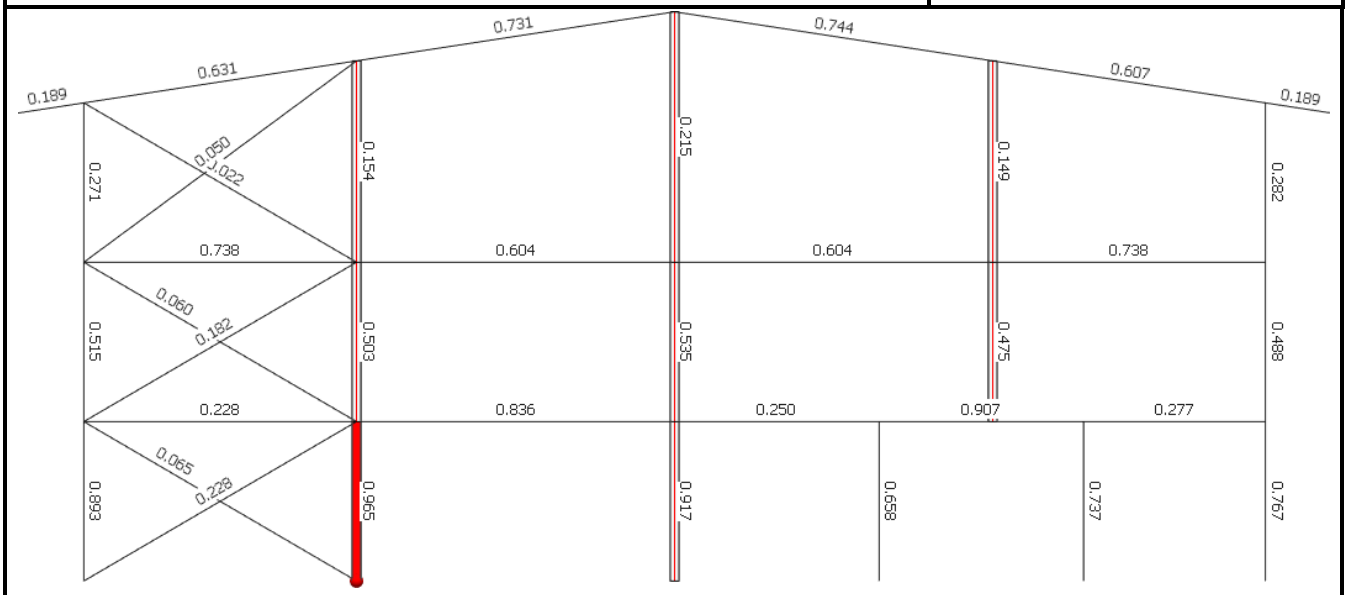
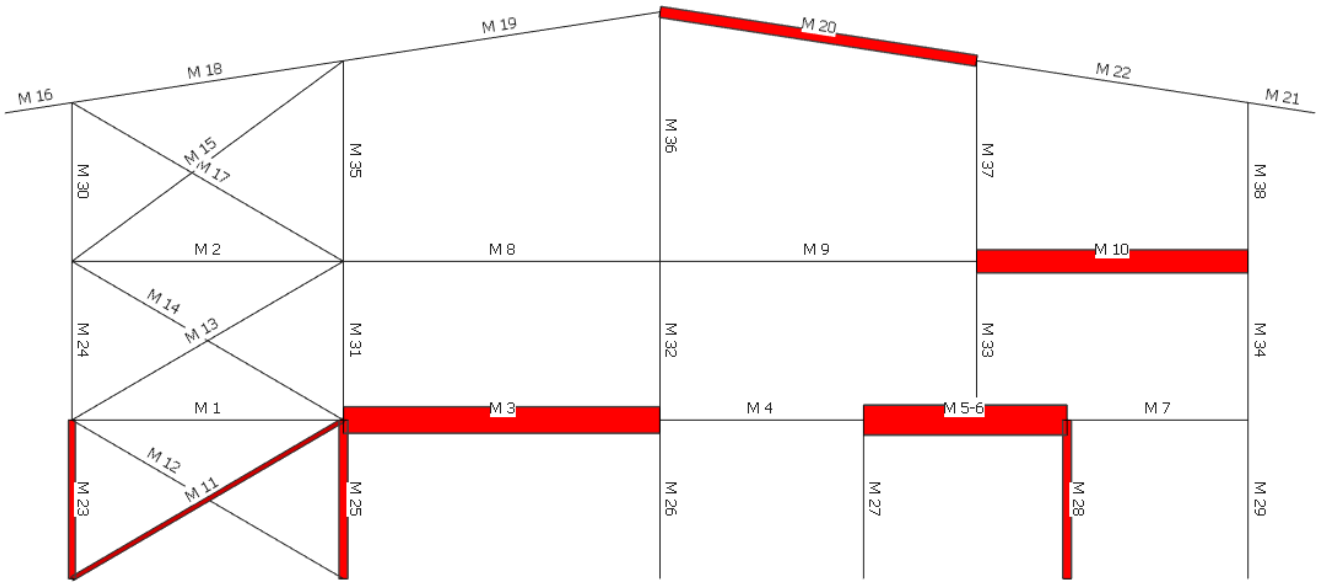


Steel Design Utilisation Factors



Member Numbers With Critical Member in each Section Size highlighted

MasterSeries Sales

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Job ref	: **Job Ref**
Sheet	: **Sheet Ref / 3 -**
Made By	:
Date	**: 21 June 2015/ Version 2015.04**
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Design Summary with Critical Member in each Section Size highlighted in Bold

List of Checks – Results Summary			
M Id 28	**00.000 to 06.946 m, in Case 002**	**0.228**	**S-T Pass**
M Id 1	00.000 to 03.500 m, in Case 001	0.228	App-G Pass
M Id 5	00.000 to 06.000 m, in Case 002	0.738	A+M Pass
M Id 5	00.000 to 03.500 m, in Case 001	0.738	App-G Pass
M Id 2	**00.000 to 07.000 m, in Case 007**	**0.836**	**A+M Pass**
M Id 2	00.000 to 03.500 m, in Case 001	0.738	App-G Pass
M Id 3	00.000 to 04.500 m, in Case 007	0.250	A+M Pass
M Id 3	00.000 to 03.500 m, in Case 001	0.250	App-G Pass
M Id 4	**00.000 to 04.500 m, in Case 007**	**0.907**	**A+M Pass**
M Id 4	00.000 to 04.500 m, in Case 001	0.907	App-G Pass
M Id 36	00.000 to 04.000 m, in Case 006	0.277	A+M Pass
M Id 36	00.000 to 03.500 m, in Case 001	0.277	App-G Pass
M Id 6	00.000 to 07.000 m, in Case 002	0.604	A+M Pass
M Id 6	00.000 to 03.500 m, in Case 001	0.604	App-G Pass
M Id 7	00.000 to 07.000 m, in Case 007	0.604	A+M Pass
M Id 7	00.000 to 03.500 m, in Case 001	0.604	App-G Pass
M Id 8	00.000 to 06.000 m, in Case 007	0.738	A+M Pass
M Id 8	**00.000 to 03.500 m, in Case 001**	**0.738**	**App-G Pass**
M Id 28	00.000 to 06.946 m, in Case 002	0.228	S-T Pass
M Id 29	00.000 to 06.946 m, in Case 007	0.065	S-T Pass
M Id 30	00.000 to 06.946 m, in Case 002	0.182	S-T Pass
M Id 31	00.000 to 06.946 m, in Case 007	0.060	S-T Pass
M Id 32	00.000 to 07.454 m, in Case 002	0.050	S-T Pass
M Id 34	00.000 to 01.500 m, in Case 003	0.184	A+M Pass
M Id 34	00.000 to 01.500 m, in Case 003	0.189	App-G Pass
M Id 33	00.000 to 06.946 m, in Case 007	0.022	S-T Pass
M Id 9	00.000 to 01.800 m, in Case 002	0.472	A+M Pass
	01.800 to 03.600 m, in Case 003	0.472	A+M Pass
	03.600 to 05.400 m, in Case 002	0.472	A+M Pass
	05.400 to 06.071 m, in Case 003	0.631	A+M Pass
M Id 9	00.000 to 03.500 m, in Case 002	0.080	App-G Pass
M Id 10	00.000 to 01.800 m, in Case 003	0.631	A+M Pass
	01.800 to 03.600 m, in Case 007	0.608	A+M Pass
	03.600 to 05.400 m, in Case 007	0.608	A+M Pass
	05.400 to 07.082 m, in Case 002	0.731	A+M Pass
M Id 10	00.000 to 03.500 m, in Case 007	0.643	App-G Pass
M Id 11	00.000 to 01.800 m, in Case 002	0.732	A+M Pass
	01.800 to 03.600 m, in Case 007	0.665	A+M Pass
	03.600 to 05.400 m, in Case 007	0.665	A+M Pass
	05.400 to 07.082 m, in Case 003	0.665	A+M Pass
M Id 11	**00.000 to 03.500 m, in Case 007**	**0.744**	**App-G Pass**
M Id 35	00.000 to 01.500 m, in Case 007	0.184	A+M Pass
M Id 35	00.000 to 01.500 m, in Case 007	0.189	App-G Pass
M Id 12	00.000 to 01.800 m, in Case 003	0.597	A+M Pass
	01.800 to 03.600 m, in Case 007	0.472	A+M Pass
	03.600 to 05.400 m, in Case 007	0.472	A+M Pass
	05.400 to 06.071 m, in Case 006	0.472	A+M Pass
M Id 12	00.000 to 03.500 m, in Case 007	0.607	App-G Pass
M Id 13	**00.000 to 03.500 m, in Case 003**	**0.893**	**A+M Pass**
M Id 18	00.000 to 03.500 m, in Case 007	0.515	A+M Pass
M Id 14	**00.000 to 03.500 m, in Case 002**	**0.965**	**A+M Pass**
M Id 15	00.000 to 03.500 m, in Case 007	0.917	A+M Pass
M Id 37	00.000 to 03.500 m, in Case 002	0.658	A+M Pass
M Id 38	**00.000 to 03.500 m, in Case 007**	**0.737**	**A+M Pass**
M Id 17	00.000 to 03.500 m, in Case 007	0.767	A+M Pass
M Id 23	00.000 to 03.500 m, in Case 003	0.271	A+M Pass
M Id 19	00.000 to 03.500 m, in Case 002	0.503	A+M Pass
M Id 20	00.000 to 03.500 m, in Case 007	0.535	A+M Pass
M Id 21	00.000 to 03.500 m, in Case 002	0.475	A+M Pass
M Id 22	00.000 to 03.500 m, in Case 006	0.488	A+M Pass
M Id 24	00.000 to 04.423 m, in Case 002	0.154	A+M Pass
M Id 25	00.000 to 05.500 m, in Case 002	0.215	A+M Pass
M Id 26	00.000 to 04.423 m, in Case 007	0.149	A+M Pass
M Id 27	00.000 to 03.500 m, in Case 003	0.282	A+M Pass

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Job ref	: **Job Ref**
Sheet	: **Sheet Ref / 4 -**
Made By	:
Date	: **21 June 2015/ Version 2015.04**
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DETAILED DESIGN OUTPUT WITH FOR THE CRITICAL MEMBERS IN ABOVE TABLE

STRUT AND TIE (MEMBER)
Member Id 28 @ Level 1 : Bracing

Classification and Effective Area (EN 1993: 2006)

Section (7.85 kg/m)	100x10 Flat 7.85 [S 355]		
Class = Fn(b/t,f_y)	10, 355	(Axial: Non-Slender)	Class 3
Effective Properties	Area=10 cm², $W_{pl.y}$=16.67(25) cm³, $W_{pl.z}$=2.5 cm³		
Auto Design Load Cases	1-3 & 6-7		

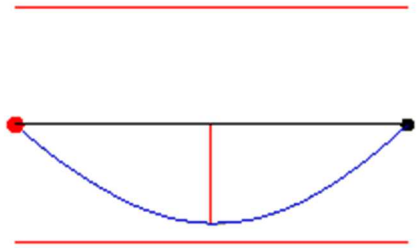
Axially Loaded Member in Tension : 6.2.3 (Case 2)

Web Holes	1 No. 22 mm Holes	2.2 cm²	
Tc = 0.9A_{net}.fu/γ_{M2}	0.9x7.8x470/1.1 (No bearing / block tearing design)	299.946 kN	
F (Tie)/Tc	68.288 / 299.946	0.228	OK

AXIAL WITH MOMENTS (MEMBER)
Member Id 2 @ Level 1 in Load Case 7 : int beam

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

D1 D 077.010 (kN/m³)
D1 UDLY -025.000 (kN/m)
L1 UDLY -028.000 (kN/m)
D1 PTRY -030.000 2.100 5.500 -030.000



Member Forces in Load Case 7 and Maximum Deflection from Load Case 5							
Span No.	Axial Force (kN)	Shear Force (kN)		Bending Moment (kNm)		Maximum Moment (kNm)	Maximum Deflection (mm @ m)
		End 1	End 2	Node 1	Node 2		
3	9	3.651C	331.866	0.000	651.379	19.850	
10	3.651C	-343.669	0.000	@ 3.573	@ 3.500		

Classification and Effective Area (EN 1993: 2006)

Section (81.81 kg/m)	610x178 UB 82 [S 355]		
Class = Fn(b/T,d/t,f_y,N,M_y,M_z)	6.95, 54.76, 355, 3.65, 651.33, 0	(Axial: Non-Slender)	Class 1
Auto Design Load Cases	1-3 & 6-7		

Local Capacity Check

$V_{y.Ed}/V_{pl.y.Rd}$	6.26 / 1295.523 =	0.005	Low Shear
$M_{c.y.Rd}$ = f_y.$W_{pl.y}$/ γ_{M0}	355 x 2194/1	778.87 kN.m	
$N_{pl.Rd}$ = A_g.f_y/ γ_{M0}	104.22 x 355/1 =	3699.81 kN	
n = $N_{Ed}/N_{pl.Rd}$	3.651 / 3699.81 =	0.001	OK
$W_{pl.N.y}$ = Fn($W_{pl.y}$, A_{vy},)	2194, 63.209, 0.001	2194 cm³	
$M_{N.y.Rd}$ = $W_{pl.N.y}$.f_y/ γ_{M0}	2194 x 355/1	778.87 kN.m	
($M_{y.Ed}/M_{N.y.Rd+(M\_(z.Ed}/M_{N.z.Rd}$)	$(651.231/778.87)^2+(0)^1$=	0.699	OK

Compression Resistance N.b.Rd

Ley = Ky.Ly	1x7 =	7	
λy = √A.fy/Ncr	√104.22x355/23631.68	0.396	
$N_{b.y.Rd}$ = Area.c.f_y/ γ_{M1}	104.22x0.954x355/10/1 =	3529.267 kN	Curve a

Lateral Buckling Check M.b.Rd

$M_{b.Rd}$ = $M_{c.y.Rd}$	Fully Restrained	778.87 kN.m	

Buckling Resistance

$U_{N.y}$ = $N_{Ed}/(c_y.N_{Rk}/\gamma_{M1})$	3.651 / 3529.267	0.001	OK

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Job ref	: **Job Ref**
Sheet	: **Sheet Ref / 5 -**
Made By	:
Date	: **21 June 2015/ Version 2015.04**
Checked	:
Approved	:

$U_{N.z} = N_{Ed}/(c_z.N_{Rk}/\gamma_{M1})$	3.651 / 3699.81	0.001	OK
$U_{M.y} = M_{y.Ed}/(c_{LT}.M_{y.Rk}/\gamma_{M1})$	651.231 / 778.87	0.836	OK
$U_{M.z} = M_{z.Ed}/(M_{z.Rk}/\gamma_{M1})$	0 / 77.39	0.000	OK
$k_yy=C_{my}\{1+(\lambda_y-0.2)U_{N.y}\}$		0.950	
$k_zz=C_{mz}\{1+(2\lambda_z-0.6)U_{N.z}\}$		0.999	
$k_yz=0.6\ k_zz$		0.600	
$k_zy=0.6\ k_yy$		0.570	
$U_{Ny}+k_yy.U_{M.y}+k_yz.U_{M.z}$	0.001+0.950x0.836+0.600x0.000	0.796	OK
$U_{Nz}+k_zy.U_{M.y}+k_zz.U_{M.z}$	0.001+0.570x0.836+0.999x0.000	0.478	OK

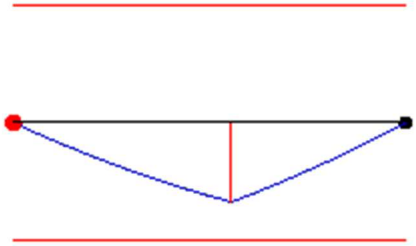
Deflection Check - Load Case 5

Deflection Limits (Internal Beams)	In-span δ ≤ 7000/360 = 19.4 mm Live (Case 4)	7.46 mm	OK
	In-span δ ≤ 7000/250 = 28 mm D+L (Case 5)	19.85 mm	OK

AXIAL WITH MOMENTS (MEMBER)
Member Id 4 @ Level 1 in Load Case 7 : Tran beam

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

D1 D 077.010 (kN/m³)
D1 UDLY -025.000 (kN/m)
L1 UDLY -028.000 (kN/m)



Member Forces in Load Case 7 and Maximum Deflection from Load Case 5							
Span No.	Axial Force (kN)	Shear Force (kN)		Bending Moment (kNm)		Maximum Moment (kNm)	Maximum Deflection (mm @ m)
		End 1	End 2	Node 1	Node 2		
	11	3.814C	467.251	0.000	926.254	4.670	
13	2.713C	-540.526	0.000	@ 2.500	@ 2.300		

Classification and Effective Area (EN 1993: 2006)

Section (125.0 kg/m)	686x254 UB 125 [S 355]		
Class = Fn(b/T,d/t,f_y,N,M_y,M_z)	7.81, 52.57, 345, 3.81, 926.25, 0	(Axial: Non-Slender)	Class 1
Auto Design Load Cases	1-3 & 6-7		

Local Capacity Check

$V_{y.Ed}/V_{pl.y.Rd}$	385.718 / 1679.499 =	0.23	Low Shear
$M_{c.y.Rd} = f_y.W_{pl.y}/\gamma_{M0}$	345 x 3994.1/1	1377.964 kN.m	
$N_{pl.Rd} = A_g.f_y/\gamma_{M0}$	159.47 x 345/1 =	5501.715 kN	
$n = N_{Ed}/N_{pl.Rd}$	2.713 / 5501.715 =	0.000	OK
$W_{pl.N.y} = Fn(W_{pl.y}, A_{vy},)$	3994.1, 84.318, 0	3994.1 cm³	
$M_{N.y.Rd} = W_{pl.N.y}.f_y/\gamma_{M0}$	3994.1 x 345/1	1377.964 kN.m	
$(M_{y.Ed}/M_{N.y.Rd+(M\_(z.Ed}/M_{N.z.Rd})$	$(926.254/1377.964)^2+(0)^1$=	0.452	OK

Compression Resistance N.b.Rd

Ley = Ky.Ly	1x4.5 =	4.5	
$\lambda y = \sqrt{A.fy/Ncr}$	√159.47x345/120781.6	0.213	
$N_{b.y.Rd} = Area.c.f_y/\gamma_{M1}$	159.47x0.997x345/10/1 =	5485.486 kN	Curve a
Lez = Kz.Lz	1x4.5 =	4.5	
$\lambda z = \sqrt{A.fy/Ncrz}$	√159.47x345/4500.6	1.106	
$N_{b.z.Rd} = Area.c.f_y/\gamma_{M1}$	159.47x0.532x345/10/1 =	2925.405 kN	Curve b
Let = Kt.Lx	1x4.5 =	4.5	
$\lambda T = \sqrt{A.fy/NcrT}$	√159.47x345/7606.48	0.85	
$N_{b.T.Rd}$= Area.c.fy/ γ_{M1}	159.47x0.693x345/10/1 =	3811.846 kN	Curve b

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

C_1= fn(M_1, M_2, M_o, ~y,~m)	0.5, 0.5, 854.9, 0.863, 300.000	1.127	Uniform
$C_{mLT}=0.95+0.05\alpha_h$	M_h = 0.54, M_s = 855.42, ~y = 0.863, α_s = 0.001	0.95	Table B.3

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Job ref	: **Job Ref**
Sheet	: **Sheet Ref / 6 -**
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Checked	:
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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 = 855.38, ~y = 1.000, α_s = 0.000	0.95	Table B.3

Lateral Buckling Check M.b.Rd

Le = 1.00 L	1 x 4.5 =	4.5 m	
M_{cr} = Fn(C_1,L_e,I_z,I_t,I_w,E)	1.127, 4.500, 4397, 116.2, 4.786, 210000	1826.678 kN.m	
λ_{LT} = √ W.f_y/M_{cr}	√ 3994.1 x 345 / 1826.678	0.869	
c_{LT} = Fn(λ_{LT}, λ_{LT}5950)	0.869, 0.913	0.721	Curve c
c_{LT}.mod = Fn(c_{LT},λ_{LT},k_c,f)	0.721, 0.869, 0.942, 0.971	0.742	6.3.2.3
$M_{b.Rd}$ = c $W_{pl.y}$.f_y ≤ $M_{c.y.Rd}$	0.742 x 3994 x 345 ≤ 1377.964 =	1022.422 kN.m	

Buckling Resistance

$U_{N.y}$ = N_{Ed}/(c_y.N_{Rk}/γ_{M1})	3.814 / 5485.486	0.001	OK
$U_{N.z}$ = N_{Ed}/(c_z.N_{Rk}/γ_{M1})	3.814 / 2925.405	0.001	OK
$U_{M.y}$ = $M_{y.Ed}$/(c_{LT}.$M_{y.Rk}$/γ_{M1})	926.254 / 1022.422	0.906	OK
$U_{M.z}$ = $M_{z.Ed}$/($M_{z.Rk}$/γ_{M1})	0 / 187.128	0.000	OK
k_yy=C_{my}{1+(λ_y-0.2)$U_{N.y}$}		0.950	
k_zz=C_{mz}{1+1.4$U_{N.z}$}		1.002	
k_yz=0.6 k_zz		0.601	
k_zy= 1- {0.1λ_z/(C_{mLT}-0.25)}$U_{N.z}$		1.000	
U_{Ny}+k_yy.$U_{M.y}$+k_yz.$U_{M.z}$	0.001+0.950x0.906+0.601x0.000	0.861	OK
U_{Nz}+k_zy.$U_{M.y}$+k_zz.$U_{M.z}$	0.001+1.000x0.906+1.002x0.000	0.907	OK

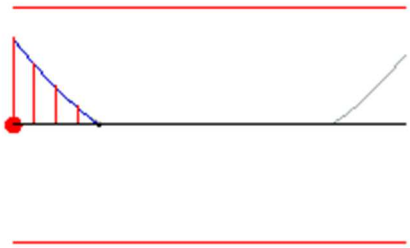
Deflection Check - Load Case 5

In-span δ ≤ Span/360	4.67 ≤ 4500 / 360	4.67 mm	OK

APPENDIX-BB STABILITY (MEMBER) :
Member Id 11 @ Level 1 : Rafters
Between 0.000 and 1.546 m, in Load Case 7

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

D1 D 077.010 (kN/m³)
D1 UDLY -012.000 (kN/m)
L1 UDLY -004.500 (kN/m)



Member Forces in Load Case 7

Span No.	Axial Force (kN)	Shear Force (kN)		Bending Moment (kNm)		Maximum Moment (kNm)	Maximum Deflection (mm @ m)
		End 1	End 2	Node 1	Node 2		
20	22	3.151T	83.482	-101.811	50.860	0.000	
24	21.721C	-78.187	-83.013	@ 3.683	@ 2.300		

Classification and Effective Area (EN 1993: 2006)

Section (31.14 kg/m)	254x146 UB 31 [S 355]		
Class = Fn(b/T,d/t,f_y,N,M_y,M_z)	8.49, 36.5, 355, -3.15, 101.81, 0	(Axial: Non-Slender)	Class 1
Auto Design Load Cases	1-3 & 6-7		

Tension Side Lateral Restraint Spacing Check, Lm

L_m = fn(N_{Ed}, A, C_1, $W_{pl,y}$, I_T)	21.7, 39.7,1.75, 393.1, 8.6	1.730m	
L_m > s	1.730>1.546 - Effect of tension side lateral restraints considered		

Compression Resistance N.b.Rd

i_s = Fn(i_y, i_z, a)	105.5, 33.6, 150.8	187.1 mm	
N_{crT} = Fn(E, I_z, I_t, I_w, L_t, a, i_s)	210, 448, 9, 0.066, 1546, 151, 187	4355.8 kN	
λT = √A.fy/NcrT	√39.67x355/ 10 /4355.77	0.569	
$N_{b.T.Rd}$= Area.c.fy/ γ_{M1}	39.67x0.853x355/10/1 =	1200.569 kN	Curve b

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Job ref	: **Job Ref**
Sheet	: **Sheet Ref / 7 -**
Made By	:
Date	: **21 June 2015/ Version 2015.04**
Checked	:
Approved	:

Modification Factor for Linear Moment Gradients Cm BB.3.3.1

N_{crE} = Fn(E, I_z, L_t)	210, 448, 1546	3888.4 kN	
~h = N_{crE} / N_{crT}	3888.4 / 4355.8	0.893	
βt=Fn(M1,M2)	0.0, 0.0	0.000	
C_m=Fn(~h,βt,B_0,B_1,B_2)	0.893, 0.000, 0.527, 0.375, 0.099	1.899	BB.13

Lateral Buckling Resistance Moment Mb

M_{cr0} = fn(N_{crT}, i_s, a)	4355.8, 0.187, 0.151	505.515 kN.m	
$M_{cr0,Lim}$ = fn(s, E, I_z, I_T, I_w)	1.546, 210, 448.4, 8.552, 0.06588	498.983 kN.m	
M_{cr0} = Min(M_{cr0}, $M_{cr0,Lim}$)	Min(0.506, 0.499)	498.983 kN.m	
c=fn(h_{max}/h_{min}, L_h/L_y, h/t_f)	1, 1, 29.233	1	BB.17
M_{cr} = c^2 • C_m • M_{cr0}	1.000^2 • 1.899 • 499.0	947.587 kN.m	
λ_{LT} = √ W.f_y/M_{cr}	√ 393.1 x 355 / 1826.678	0.384	
c_{LT} = Fn(λ_{LT}, Φ_{LT}, β)	0.384, 0.898, 0.750	1.000	Curve d
c_{LT}.mod = Fn(c_{LT},λ_{LT},k_c,f)	1.000, 0.384, 0.942, 0.971	1.000	6.3.2.3
$M_{b.Rd}$ = c $W_{pl.y}$.f_y	1.000 x 393.1 x 355	139.551 kN.m	

Combined Axial and Bending 6.62

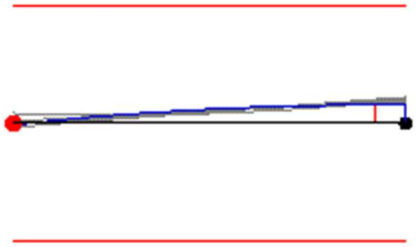
k_{zy}	1- {0.1/(C_{mLT}-0.25)}• $N_{Ed}/N_{b,z,Rd}$	0.995	
$N_{Ed}/N_{b,z,Rd}$+k_{zy}.$M_{y,Ed}/M_{b,Rd}$	21.721 / 1200.569 + 0.995 x 101.811 / 139.551 =	0.744	OK

AXIAL WITH MOMENTS (MEMBER)

Member Id 13 @ Level 1 in Load Case 3 : External Columnns

Member Loading and Member Forces
Loading Combination : 1 UT + 1.35 D1 + 1.5 L1 + 0.75 W2

D1 D 077.010 (kN/m³)
W2 UDLX -004.800 (kN/m)



Member Forces in Load Case 3 and Maximum Deflection from Load Case 5							
Span No.	Axial Force (kN)	Shear Force (kN)		Bending Moment (kNm)		Maximum Moment (kNm)	Maximum Deflection (mm @ m)
		End 1	End 2	Node 1	Node 2		
23	1	552.343C	-6.216	1.348	-3.338	0.133	
2	550.629C	6.384	3.068	@ 1.610	@ 2.205		

Additional Nominal Moments

M_{yUp}	-19.989 kN.m

Classification and Effective Area (EN 1993: 2006)

Section (36.98 kg/m)	152x152 UC 37 [S 355]		
Class = Fn(b/T,d/t,f_y,N,M_y,M_z)	6.71, 15.45, 355, 552.34, 17.08, 0	(Axial: Non-Slender)	Class 1
Auto Design Load Cases	1-3 & 6-7		

Local Capacity Check

$V_{y.Ed}/V_{pl.y.Rd}$	5.415 / 292.395 =	0.019	Low Shear
$M_{c.y.Rd}$ = f_y.$W_{pl.y}$/ γ_{M0}	355 x 308.8/1	109.624 kN.m	
$N_{pl.Rd}$ = A_g.f_y/ γ_{M0}	47.11 x 355/1 =	1672.405 kN	
n = $N_{Ed}/N_{pl.Rd}$	552.343 / 1672.405 =	0.330	OK
$W_{pl.N.y}$ = Fn($W_{pl.y}$, A_{vy},)	308.8, 14.266, 0.33	235.84 cm³	
$M_{N.y.Rd}$ = $W_{pl.N.y}$.f_y/ γ_{M0}	235.84 x 355/1	83.725 kN.m	
($M_{y.Ed}/M_{N.y.Rd+(M\_(z.Ed}/M_{N.z.Rd}$)	$(17.08/83.725)^2+(0)^{1.651}$=	0.042	OK

Compression Resistance N.b.Rd

Ley = Ky.Ly	1x3.5 =	3.5	
λy = √A.fy/Ncr	√47.11x355/3741.54	0.669	
$N_{b.y.Rd}$ = Area.c.f_y/ γ_{M1}	47.11x0.801x355/10/1 =	1339.741 kN	Curve b

MasterSeries Sales Team
3 Castle Street
Carrickfergus
Co. Antrim BT38 7BE

Tel : 028 9036 5950

Job ref	: **Job Ref**
Sheet	: **Sheet Ref / 8 -**
Made By	:
Date	: **21 June 2015/ Version 2015.04**
Checked	:
Approved	:

Lez = Kz.Lz	1x3.5 =	3.5	
λz = √A.fy/Ncrz	√47.11x355/1196.37	1.184	
$N_{b.z.Rd}$ = Area.c.f_y/ γ_{M1}	47.11x0.442x355/10/1 =	738.636 kN	Curve c
Let = Kt.Lx	1x3.5 =	3.5	
λT = √A.fy/NcrT	√47.11x355/3591.12	0.682	
$N_{b.T.Rd}$= Area.c.fy/ γ_{M1}	47.11x0.736x355/10/1 =	1230.116 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)	1.3, -16.9, -5.5, -0.079, 0.325	1.279	Uniform
C_{mLT}=Max(0.2+0.8α_s, 0.4)	M_h = -16.92, M_s = -13.3, ~y = -0.079, α_s = 0.786	0.829	Table B.3
C_{mz}=Max(0.6+0.4~y, 0.4)	M = 0, ~y = 1.000	1	Table B.3
C_{my}=Max(0.6+0.4~y, 0.4)	M = -16.92, ~y = -0.080	0.568	Table B.3

Lateral Buckling Check M.b.Rd

Le = 1.00 L	1 x 3.5 =	3.5 m	
M_{cr} = Fn(C_1,L_e,I_z,I_t,I_w,E)	1.279, 3.500, 707.1, 19.18, 0.03984, 210000	208.596 kN.m	
λ_{LT} = √ W.f_y/M_{cr}	√ 308.8 x 355 / 208.596	0.725	
c_{LT} = Fn(λ_{LT}, λ_{LT}5950)	0.725, 0.744	0.857	Curve b
c_{LT}.mod = Fn(c_{LT},λ_{LT},k_c,f)	0.857, 0.725, 0.884, 0.943	0.909	6.3.2.3
$M_{b.Rd}$ = c $W_{pl.y}$.f_y ≤ $M_{c.y.Rd}$	0.909 x 308.8 x 355 ≤ 109.624 =	99.656 kN.m	

Buckling Resistance

$U_{N.y}$ = N_{Ed}/(c_y.N_{Rk}/γ_{M1})	552.343 / 1339.741	0.412	OK
$U_{N.z}$ = N_{Ed}/(c_z.N_{Rk}/γ_{M1})	552.343 / 738.636	0.748	OK
$U_{M.y}$ = $M_{y.Ed}$/(c_{LT}.$M_{y.Rk}$/γ_{M1})	17.08 / 99.656	0.171	OK
$U_{M.z}$ = $M_{z.Ed}$/($M_{z.Rk}$/γ_{M1})	0 / 49.558	0.000	OK
k_yy={C_{my}{1+(λ_y-0.2)$U_{N.y}$}		0.678	
k_zz={C_{mz}{1+1.4$U_{N.z}$}		2.047	
k_yz=0.6 k_zz		1.228	
k_zy= 1- {0.1λ_z/(C_{mLT}-0.25)}$U_{N.z}$		0.847	
U_{Ny}+k_yy.$U_{M.y}$+k_yz.$U_{M.z}$	0.412+0.678x0.171+1.228x0.000	0.528	OK
U_{Nz}+k_zy.$U_{M.y}$+k_zz.$U_{M.z}$	0.748+0.847x0.171+2.047x0.000	0.893	OK

Deflection Check - Load Case 5

In-span δ ≤ Span/360	0.13 ≤ 3500 / 360	0.13 mm	OK

AXIAL WITH MOMENTS (MEMBER)

Member Id 14 @ Level 1 in Load Case 2 : Int Col

Member Loading and Member Forces
Loading Combination : 1 UT + 1.35 D1 + 1.5 L1 + 0.75 W1

D1 D 077.010 (kN/m³)

Member Forces in Load Case 2 and Maximum Deflection from Load Case 5							
Span No.	Axial Force (kN)	Shear Force (kN)		Bending Moment (kNm)		Maximum Moment (kNm)	Maximum Deflection (mm @ m)
		End 1	End 2	Node 1	Node 2		
25	3	1295.818C	3.086	-12.976		0.040	
9	1293.406C	3.086	3.661		@ 1.820		

Additional Nominal Moments

M_{yUp}	-6.813 kN.m

Classification and Effective Area (EN 1993: 2006)

Section (52.03 kg/m)	203x203 UC 52 [S 355]		
Class = Fn(b/T,d/t,f_y,N,M_y,M_z)	8.17, 20.35, 355, 1295.82, 12.98, 0	(Axial: Non-Slender)	Class 1

MasterSeries Sales Team
3 Castle Street
Carrickfergus
Co. Antrim BT38 7BE

Tel : 028 9036 5950

Job ref	: **Job Ref**
Sheet	: **Sheet Ref / 9 -**
Made By	:
Date	: **21 June 2015/ Version 2015.04**
Checked	:
Approved	:

Auto Design Load Cases	1-3 & 6-7		

Local Capacity Check

$V_{y.Ed}/V_{pl.y.Rd}$	3.086 / 384.145 =	0.008	Low Shear
$M_{c.y.Rd} = f_y.W_{pl.y}/ \gamma_{M0}$	355 x 567.4/1	201.427 kN.m	
$N_{pl.Rd} = A_g.f_y/ \gamma_{M0}$	66.28 x 355/1 =	2352.94 kN	
$n = N_{Ed}/N_{pl.Rd}$	1295.818 / 2352.94 =	0.551	OK
$W_{pl.N.y}$ = Fn($W_{pl.y}$, A_{vy},)	567.4, 18.743, 0.551	287.95 cm³	
$M_{N.y.Rd} = W_{pl.N.y}.f_y/ \gamma_{M0}$	287.95 x 355/1	102.222 kN.m	
$(M_{y.Ed}/M_{N.y.Rd+(M\_(z.Ed}/M_{N.z.Rd})$	$(12.976/102.222)^2+(0)^{2.754}$=	0.016	OK

Compression Resistance N.b.Rd

Ley = Ky.Ly	1x3.5 =	3.5	
$\lambda y = \sqrt{A.fy/Ncr}$	√66.28x355/8902.44	0.514	
$N_{b.y.Rd} = Area.c.f_y/ \gamma_{M1}$	66.28x0.878x355/10/1 =	2065.688 kN	Curve b
Lez = Kz.Lz	1x3.5 =	3.5	
$\lambda z = \sqrt{A.fy/Ncrz}$	√66.28x355/3012.49	0.884	
$N_{b.z.Rd} = Area.c.f_y/ \gamma_{M1}$	66.28x0.61x355/10/1 =	1434.157 kN	Curve c
Let = Kt.Lx	1x3.5 =	3.5	
$\lambda T = \sqrt{A.fy/NcrT}$	√66.28x355/5069.15	0.681	
$N_{b.T.Rd}$= Area.c.fy/ γ_{M1}	66.28x0.736x355/10/1 =	1732.307 kN	Curve c

Equivalent Uniform Moment Factor C1

C_1= fn(M_1, M_2, ~y)	-13.0, -10.5, 0.807	1.098	Not Loaded
C_{mLT}=Max(0.6+0.4~y, 0.4)	M = -12.97, ~y = 0.807	0.923	Table B.3
C_{mz}=Max(0.6+0.4~y, 0.4)	M = 0, ~y = 1.000	1	Table B.3
C_{my}=Max(0.6+0.4~y, 0.4)	M = -12.98, ~y = 0.807	0.923	Table B.3

Lateral Buckling Check M.b.Rd

Le = 1.00 L	1 x 3.5 =	3.5 m	
M_{cr} = Fn(C_1,L_e,I_z,I_t,I_w,E)	1.098, 3.500, 1781, 31.76, 0.1666, 210000	442.141 kN.m	
$\lambda_{LT} = \sqrt{W.f_y/M_{cr}}$	√ 567.4 x 355 / 442.141	0.675	
c_{LT} = Fn(λ_{LT}, λ_{LT}5950)	0.675, 0.638	0.882	Curve b
c_{LT}.mod = Fn(c_{LT},λ_{LT},k_c,f)	0.882, 0.675, 0.954, 0.978	0.902	6.3.2.3
$M_{b.Rd} = c\ W_{pl.y}.f_y \le M_{c.y.Rd}$	0.902 x 567.4 x 355 ≤ 201.427 =	181.666 kN.m	

Buckling Resistance

$U_{N.y} = N_{Ed}/(c_y.N_{Rk}/\gamma_{M1})$	1295.818 / 2065.688	0.627	OK
$U_{N.z} = N_{Ed}/(c_z.N_{Rk}/\gamma_{M1})$	1295.818 / 1434.157	0.904	OK
$U_{M.y} = M_{y.Ed}/(c_{LT}.M_{y.Rk}/\gamma_{M1})$	12.976 / 181.666	0.071	OK
$U_{M.z} = M_{z.Ed}/(M_{z.Rk}/\gamma_{M1})$	0 / 93.791	0.000	OK
$k_yy=C_{my}\{1+(\lambda_y-0.2)U_{N.y}\}$		1.105	
$k_zz=C_{mz}\{1+(2\lambda_z-0.6)U_{N.z}\}$		2.056	
$k_yz=0.6\ k_zz$		1.234	
k_zy= 1- {0.1/(C_{mLT}-0.25)}$U_{N.z}$		0.866	
$U_{Ny}+k_yy.U_{M.y}+k_yz.U_{M.z}$	0.627+1.105x0.071+1.234x0.000	0.706	OK
$U_{Nz}+k_zy.U_{M.y}+k_zz.U_{M.z}$	0.904+0.866x0.071+2.056x0.000	0.965	OK

Deflection Check - Load Case 5

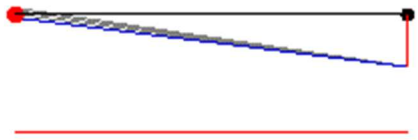
In-span δ ≤ Span/360	0.04 ≤ 3500 / 360	0.04 mm	OK

AXIAL WITH MOMENTS (MEMBER)

Member Id 38 @ Level 1 in Load Case 7 : Single Cols

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

D1 D 077.010 (kN/m³)



MasterSeries Sales Team
3 Castle Street
Carrickfergus
Co. Antrim BT38 7BE

Tel : 028 9036 5950

Job ref	: **Job Ref**
Sheet	: **Sheet Ref / 10 -**
Made By	:
Date	**: 21 June 2015/ Version 2015.04**
Checked	**:**
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Member Forces in Load Case 7 and Maximum Deflection from Load Case 5

Span No.	Axial Force (kN)	Shear Force (kN)		Bending Moment (kNm)		Maximum Moment (kNm)	Maximum Deflection (mm @ m)
		End 1	End 2	Node 1	Node 2		
28	6	695.906C	-0.883	4.510		0.026	
13	693.769C	-0.883	0.000		@ 1.470		

Additional Nominal Moments

M_{yUp}	78.077 kN.m		

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, 695.91, 78.08, 0	(Axial: Non-Slender)	Class 2
Auto Design Load Cases	1-3 & 6-7		

Local Capacity Check

$V_{y.Ed}/V_{pl.y.Rd}$	0.883 / 347.898 =	0.003	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	
$n = N_{Ed}/N_{pl.Rd}$	695.906 / 2084.915 =	0.334	OK
$W_{pl.N.y}$ = Fn($W_{pl.y}$, A_{vy},)	497.4, 16.974, 0.334	375.99 cm³	
$M_{N.y.Rd} = W_{pl.N.y}.f_y/ \gamma_{M0}$	375.99 x 355/1	133.478 kN.m	
($M_{y.Ed}/M_{N.y.Rd+(M\_(z.Ed}/M_{N.z.Rd}$)	$(78.077/133.478)^2+(0)^{1.669}$=	0.342	OK

Compression Resistance N.b.Rd

Ley = Ky.Ly	1x3.5 =	3.5	
$\lambda y = \sqrt{A.fy/Ncr}$	√58.73x355/7733.31	0.519	
$N_{b.y.Rd} = Area.c.f_y/ \gamma_{M1}$	58.73x0.876x355/10/1 =	1825.440 kN	Curve b
Lez = Kz.Lz	1x3.5 =	3.5	
$\lambda z = \sqrt{A.fy/Ncrz}$	√58.73x355/2624.36	0.891	
$N_{b.z.Rd} = Area.c.f_y/ \gamma_{M1}$	58.73x0.605x355/10/1 =	1261.932 kN	Curve c
Let = Kt.Lx	1x3.5 =	3.5	
$\lambda T = \sqrt{A.fy/NcrT}$	√58.73x355/4037.02	0.719	
$N_{b.T.Rd}$= Area.c.fy/ γ_{M1}	58.73x0.713x355/10/1 =	1486.786 kN	Curve c

Equivalent Uniform Moment Factor C1

C_1= fn(M_1, M_2, ~y)	4.5, 78.1, 0.058	1.690	Not Loaded
C_{mLT}=Max(0.6+0.4~y, 0.4)	M = 78.06, ~y = 0.058	0.623	Table B.3
C_{mz}=Max(0.6+0.4~y, 0.4)	M = 0, ~y = 1.000	1	Table B.3
C_{my}=Max(0.6+0.4~y, 0.4)	M = 78.08, ~y = 0.058	0.623	Table B.3

Lateral Buckling Check M.b.Rd

Le = 1.00 L	1 x 3.5 =	3.5 m	
M_{cr} = Fn(C_1,L_e,I_z,I_t,I_w,E)	1.690, 3.500, 1551, 22.15, 0.1429, 210000	561.569 kN.m	
$\lambda_{LT} = \sqrt{ W.f_y/M_{cr}}$	√ 497.4 x 355 / 561.569	0.561	
c_{LT} = Fn(λ_{LT}, λ_{LT}5950)	0.561, 0.657	0.934	Curve b
c_{LT}.mod = Fn(c_{LT},λ_{LT},k_c,f)	0.934, 0.561, 0.769, 0.898	1.000	6.3.2.3
$M_{b.Rd} = c\ W_{pl.y}.f_y \le M_{c.y.Rd}$	1.000 x 497.4 x 355 ≤ 176.577 =	176.577 kN.m	

Buckling Resistance

$U_{N.y} = N_{Ed}/(c_y.N_{Rk}/\gamma_{M1})$	695.906 / 1825.44	0.381	OK
$U_{N.z} = N_{Ed}/(c_z.N_{Rk}/\gamma_{M1})$	695.906 / 1261.932	0.551	OK
$U_{M.y} = M_{y.Ed}/(c_{LT}.M_{y.Rk}/\gamma_{M1})$	78.077 / 176.577	0.442	OK
$U_{M.z} = M_{z.Ed}/(M_{z.Rk}/\gamma_{M1})$	0 / 81.97	0.000	OK
$k_yy=C_{my}\{1+(\lambda_y-0.2)U_{N.y}\}$		0.699	
$k_zz=C_{mz}\{1+(2\lambda_z-0.6)U_{N.z}\}$		1.652	
$k_yz=0.6\ k_zz$		0.991	
$k_zy=0.6\ k_yy$		0.419	
$U_{Ny}+k_yy.U_{M.y}+k_yz.U_{M.z}$	0.381+0.699x0.442+0.991x0.000	0.690	OK
$U_{Nz}+k_zy.U_{M.y}+k_zz.U_{M.z}$	0.551+0.419x0.442+1.652x0.000	0.737	OK

Deflection Check - Load Case 5

In-span δ ≤ Span/360	0.03 ≤ 3500 / 360	0.03 mm	OK

MasterSeries Sales Team
3 Castle Street
Carrickfergus
Co. Antrim BT38 7BE

Tel : 028 9036 5950

Job ref	: **Job Ref**
Sheet	: **Sheet Ref / 11 -**
Made By	:
Date	**: 21 June 2015/ Version 2015.04**
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ELASTIC CRITICAL LOAD FACTOR

Load Case 002 : (Ultimate) Dead plus Live plus Wind on Side		
h, H_{Ed}, V_{Ed}, δ	3.500, 81.57, 5507.51, 4.720mm	
α_{cr}	Critical Level 1 - Member 23, $(h.H_{Ed}/(\delta.V_{Ed})$	10.982
Load Case 003 : (Ultimate) Dead Plus Live Plus Wind On Gable		
h, H_{Ed}, V_{Ed}, δ	3.500, 24.03, 5508.03, 2.653mm	
α_{cr}	Critical Level 1 - Member 29, $(h.H_{Ed}/(\delta.V_{Ed})$	5.756
Sway Amplfication Factor†	$1 / (1 - 1 / \alpha_{cr})$	1.210
Load Case 006 : Dead plus Live (Ultimate) + Notional @ 0°		
h, H_{Ed}, V_{Ed}, δ	3.500, 24.03, 5507.51, 2.638mm	
α_{cr}	Critical Level 1 - Member 23, $(h.H_{Ed}/(\delta.V_{Ed})$	5.789
Sway Amplfication Factor†	$1 / (1 - 1 / \alpha_{cr})$	1.209
Load Case 007 : Dead plus Live (Ultimate) + Notional @ 180°		
h, H_{Ed}, V_{Ed}, δ	3.500, 23.89, 5508.02, 2.046mm	
α_{cr}	Critical Level 1 - Member 29, $(h.H_{Ed}/(\delta.V_{Ed})$	7.417
Sway Amplfication Factor†	$1 / (1 - 1 / \alpha_{cr})$	1.156

SWAY STABILITY SUMMARY

Load Case	α_{cr}	Sway Amp†	Plastic LF	Method	From Case	P-~D ON	Verdict
001 : Dead plus Live (Ultimate)	Not calculated						
002 : (Ultimate) Dead plus Live pl	10.98	1.000	N.A	HNL Method	002	Yes	OK
003 : (Ultimate) Dead Plus Live Pl	5.756	1.210	N.A	HNL Method	003	Yes	OK
006 : Dead plus Live (Ultimate) +	5.789	1.209	N.A	HNL Method	006	Yes	OK
007 : Dead plus Live (Ultimate) +	7.417	1.156	N.A	HNL Method	007	Yes	OK

Second-Order Elastic Analysis has been used in the following cases

Load Case 001 : Load Case 001 : Dead plus Live (Ultimate)
Load Case 002 : Load Case 002 : (Ultimate) Dead plus Live plus Wind on Side
Load Case 003 : Load Case 003 : (Ultimate) Dead Plus Live Plus Wind On Gable
Load Case 006 : Load Case 006 : Dead plus Live (Ultimate) + Notional @ 0°
Load Case 007 : Load Case 007 : Dead plus Live (Ultimate) + Notional @ 180°

†Note that Sway Amplifaction factors are given for information purposes only and are not applied to the forces in the frame.
In all cases of sway sensitivity it is recommended to carry out appropriate P-Delta analysis.