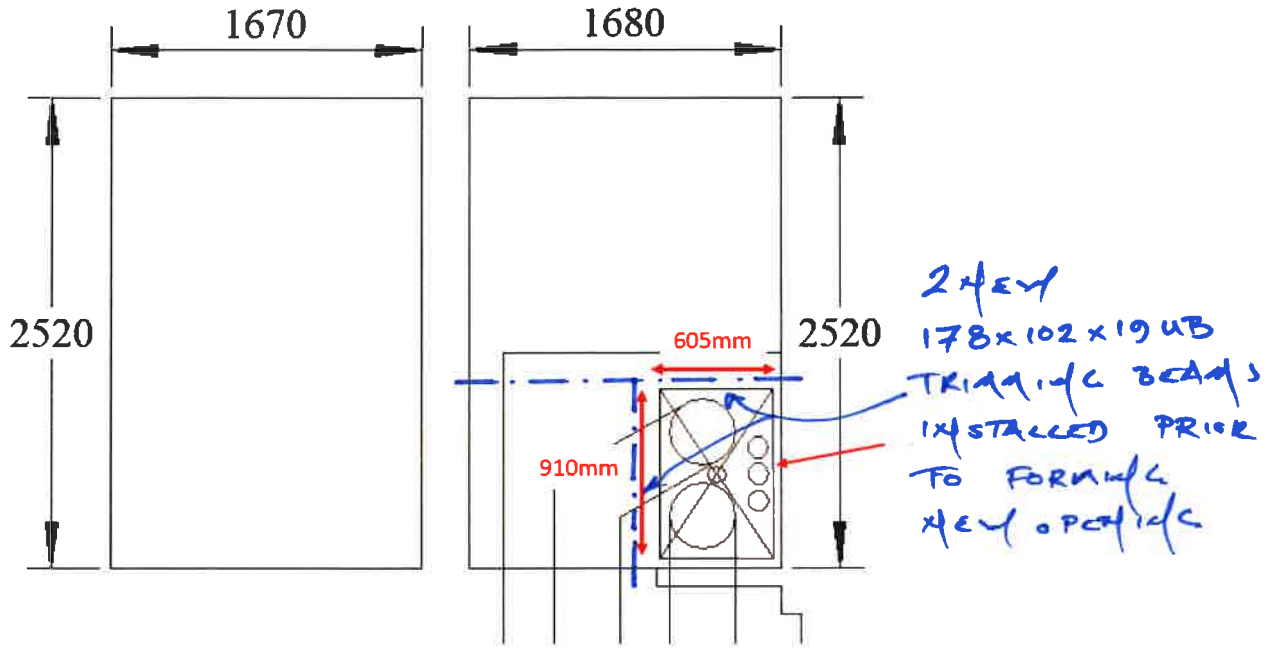
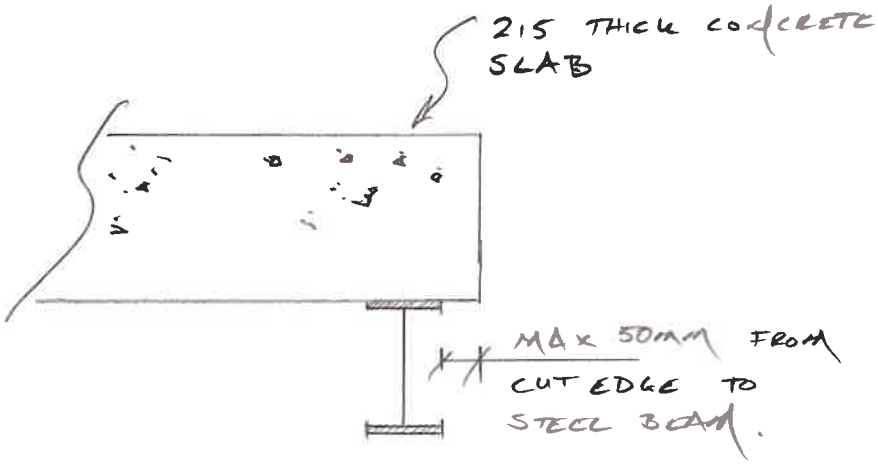
Marston & Grundy LLP Consulting Engineers Priory House, Ellesmere Avenue Marple, Stockport SK6 7AN	Project: MCFC – Pitch Replacement				Job Ref.: 12554	
	Section: Disabled WC Modifications				Sheet no./rev.: SK01	
	Calc. by	Date	Chk'd by	Date	App'd by	Date

Disabled Toilet (L) Disabled Toilet (R)





BEAMS TO SIT ON 150 DEEP x 215 WIDE x BLOCK THICKNESS CONCRETE PADSTONES IN BLOCKWORK

FOR CONNECTION DETAIL SEE CALC SHEET PAGE 5.

STEEL TO BE PRIMED WITH ZINC RICH PRIMER TO A MIN DFT OF 75 MICRONS.

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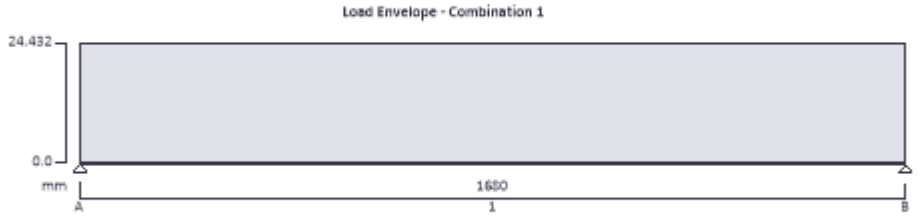
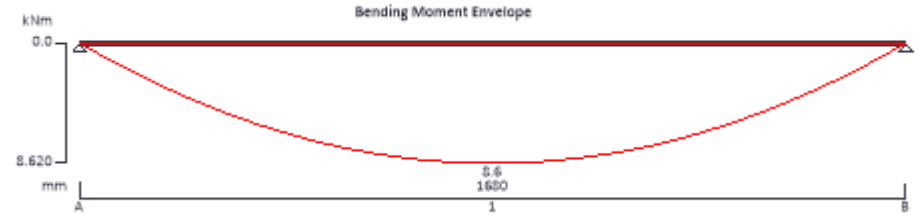
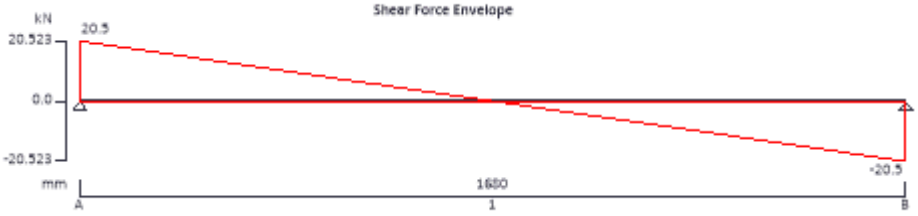
STEEL BEAM ANALYSIS & DESIGN (EN1993-1-1:2005)

In accordance with EN1993-1-1:2005 incorporating Corrigenda February 2006 and April 2009 and the UK national annex

TEDDS calculation version 3.0.14







Support conditions

Support A	Vertically restrained
	Rotationally free
Support B	Vertically restrained
	Rotationally free

Applied loading

Beam loads	imposed - Variable full UDL 9.37 kN/m
	slab - Permanent full UDL 7.5 kN/m
	swt - Permanent self weight of beam × 1

Load combinations

Load combination 1	Support A	Permanent × 1.35
		Variable × 1.50
		Permanent × 1.35
		Variable × 1.50
	Support B	Permanent × 1.35
		Variable × 1.50

Analysis results

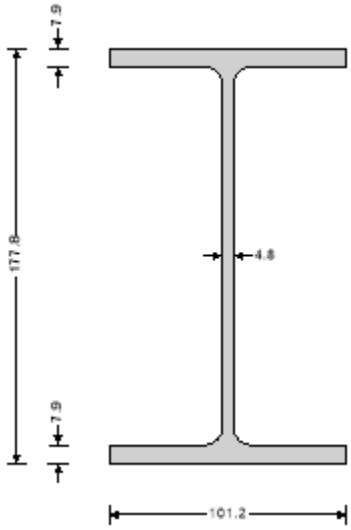
Maximum moment	M_{max} = **8.6** kNm	M_{min} = **0** kNm
Maximum shear	V_{max} = **20.5** kN	V_{min} = **-20.5** kN
Deflection	δ_{max} = **0.6** mm	δ_{min} = **0** mm
Maximum reaction at support A	R_{A_max} = **20.5** kN	R_{A_min} = **20.5** kN
Unfactored permanent load reaction at support A	$R_{A\_Permanent}$ = **6.5** kN	
Unfactored variable load reaction at support A	$R_{A\_Variable}$ = **7.9** kN	
Maximum reaction at support B	R_{B_max} = **20.5** kN	R_{B_min} = **20.5** kN
Unfactored permanent load reaction at support B	$R_{B\_Permanent}$ = **6.5** kN	
Unfactored variable load reaction at support B	$R_{B\_Variable}$ = **7.9** kN	

Section details

Section type	**UKB 178x102x19 (Tata Steel Advance)**
Steel grade	**S355**

EN 10025-2:2004 - Hot rolled products of structural steels

Nominal thickness of element	t = max(t_f, t_w) = **7.9** mm
Nominal yield strength	f_y = **355** N/mm^2
Nominal ultimate tensile strength	f_u = **470** N/mm^2
Modulus of elasticity	E = **210000** N/mm^2



Partial factors - Section 6.1

Resistance of cross-sections	γ_{M0} = **1.00**
Resistance of members to instability	γ_{M1} = **1.00**
Resistance of tensile members to fracture	γ_{M2} = **1.10**

Lateral restraint

Span 1 has lateral restraint at supports only

Effective length factors

Effective length factor in major axis	K_y = **1.000**
Effective length factor in minor axis	K_z = **1.000**
Effective length factor for torsion	$K_{LT.A}$ = **1.000**
	$K_{LT.B}$ = **1.000**

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Classification of cross sections - Section 5.5

$\varepsilon = \sqrt{[235\ N/mm^2 / f_y]} =$ **0.81**

Internal compression parts subject to bending - Table 5.2 (sheet 1 of 3)

Width of section: $c = d =$ **146.8** mm

$c / t_w = 37.6 \times \varepsilon <= 72 \times \varepsilon$ Class 1

Outstand flanges - Table 5.2 (sheet 2 of 3)

Width of section: $c = (b - t_w - 2 \times r) / 2 =$ **40.6** mm

$c / t_f = 6.3 \times \varepsilon <= 9 \times \varepsilon$ Class 1

Section is class 1

Check shear - Section 6.2.6

Height of web: $h_w = h - 2 \times t_f =$ **162** mm

Shear area factor: $\eta =$ **1.000**

$h_w / t_w < 72 \times \varepsilon / \eta$

Shear buckling resistance can be ignored

Design shear force: $V_{Ed} = \max(abs(V_{max}), abs(V_{min})) =$ **20.5** kN

Shear area - cl 6.2.6(3): $A_v = \max(A - 2 \times b \times t_f + (t_w + 2 \times r) \times t_f, \eta \times h_w \times t_w) =$ **985** mm^2

Design shear resistance - cl 6.2.6(2): $V_{c,Rd} = V_{pl,Rd} = A_v \times (f_y / \sqrt{[3]}) / \gamma_{M0} =$ **201.9** kN

PASS - Design shear resistance exceeds design shear force

Check bending moment major (y-y) axis - Section 6.2.5

Design bending moment: $M_{Ed} = \max(abs(M_{s1\_max}), abs(M_{s1\_min})) =$ **8.6** kNm

Design bending resistance moment - eq 6.13: $M_{c,Rd} = M_{pl,Rd} = W_{pl.y} \times f_y / \gamma_{M0} =$ **60.8** kNm

Slenderness ratio for lateral torsional buckling

Correction factor - Table 6.6: $k_c =$ **0.94**

$C_1 = 1 / k_c^2 =$ **1.132**

Curvature factor: $g = \sqrt{[1 - (I_z / I_y)]} =$ **0.948**

Poissons ratio: $\nu =$ **0.3**

Shear modulus: $G = E / [2 \times (1 + \nu)] =$ **80769** N/mm^2

Unrestrained length: $L = 1.0 \times L_{s1} =$ **1680** mm

Elastic critical buckling moment: $M_{cr} = C_1 \times \pi^2 \times E \times I_z / (L^2 \times g) \times \sqrt{[I_w / I_z + L^2 \times G \times I_t / (\pi^2 \times E \times I_z)]} =$ **124.3** kNm

Slenderness ratio for lateral torsional buckling: $\bar{\lambda}_{LT} = \sqrt{(W_{pl.y} \times f_y / M_{cr})} =$ **0.699**

Limiting slenderness ratio: $\bar{\lambda}_{LT,0} =$ **0.4**

$\bar{\lambda}_{LT} > \bar{\lambda}_{LT,0}$ *- Lateral torsional buckling cannot be ignored*

Design resistance for buckling - Section 6.3.2.1

Buckling curve - Table 6.5: b

Imperfection factor - Table 6.3: $\alpha_{LT} =$ **0.34**

Correction factor for rolled sections: $\beta =$ **0.75**

LTB reduction determination factor: $\phi_{LT} = 0.5 \times [1 + \alpha_{LT} \times (\bar{\lambda}_{LT} - \bar{\lambda}_{LT,0}) + \beta \times \bar{\lambda}_{LT}^2] =$ **0.734**

LTB reduction factor - eq 6.57: $\chi_{LT} = \min(1 / [\phi_{LT} + \sqrt{(\phi_{LT}^2 - \beta \times \bar{\lambda}_{LT}^2)}], 1, 1 / \bar{\lambda}_{LT}^2) =$ **0.870**

Modification factor: $f = \min(1 - 0.5 \times (1 - k_c) \times [1 - 2 \times (\bar{\lambda}_{LT} - 0.8)^2], 1) =$ **0.971**

Modified LTB reduction factor - eq 6.58: $\chi_{LT,mod} = \min(\chi_{LT} / f, 1) =$ **0.896**

Design buckling resistance moment - eq 6.55: $M_{b,Rd} = \chi_{LT,mod} \times W_{pl.y} \times f_y / \gamma_{M1} =$ **54.5** kNm

PASS - Design buckling resistance moment exceeds design bending moment

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Check vertical deflection - Section 7.2.1

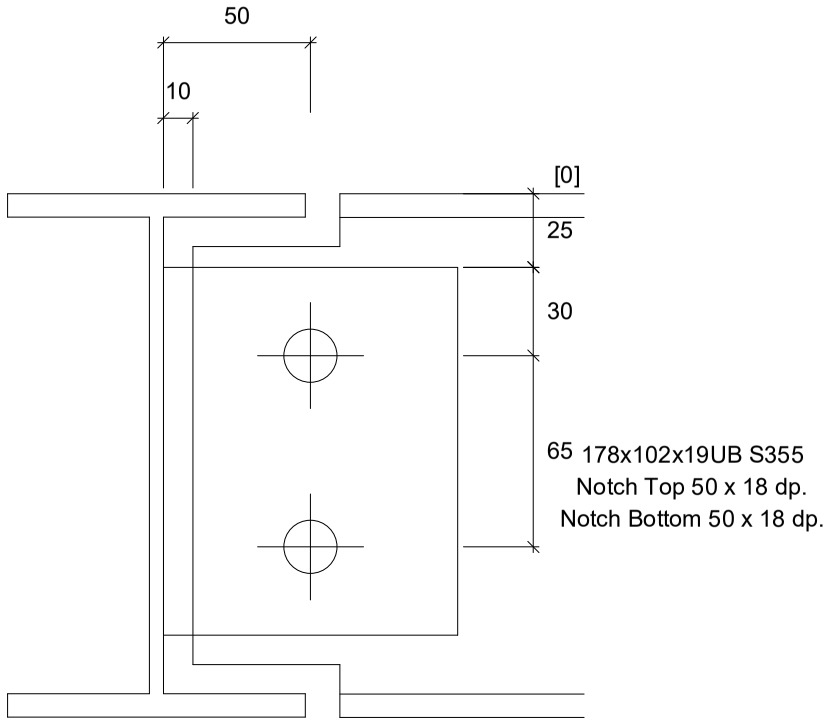
Consider deflection due to permanent and variable loads

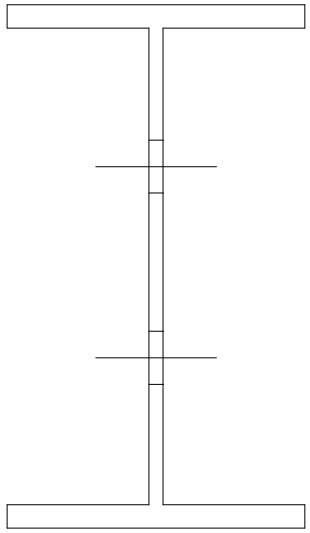
Limiting deflection $\delta_{lim} = L_{s1} / 360 =$ **4.7** mm

Maximum deflection span 1 $\delta = \max(\text{abs}(\delta_{max}), \text{abs}(\delta_{min})) =$ **0.621** mm

PASS - Maximum deflection does not exceed deflection limit

Combination: Dead+Super
Shear 25.00 kN. + 0.00 kN. on 0 bolt row(s) other side of joint





178x102x19UB S355
100 x 10 FL x 125 LG Fin Plate
M16 Grade 8.8 bolts 18 dia holes
8 mm. profile fillet weld
All fittings grade S275

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Printed	: 01/06/2021	Sheet	: 5
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Title	: MCFC New Pitch Trimmer beam connection		
Designed	: md	Checked	:

Summary of results	**Unity**	**Verdict**	**Comment**
Bolts in shear	0.38	Passed	
Fin plate bearing	0.23	Passed	
Fin plate shear gross	0.16	Passed	
Fin plate shear net	0.13	Passed	
Fin plate shear block	0.14	Passed	
Fin plate bending	0.17	Passed	
Fin plate buckling (LTB)	0.17	Passed	
Fin plate weld	0.90	Passed	
Beam web bearing	0.44	Passed	
Beam web shear gross	0.20	Passed	
Beam web shear net	0.23	Passed	
Beam web shear block	0.24	Passed	
Beam web shear/bending at connection	0.22	Passed	
Beam web shear/bending at notch	0.26	Passed	
Beam web shear/bending interaction	0.35	Passed	
Supporting beam web punching shear	0.23	Passed	
Supporting beam web shear	0.20	Passed	
Ductility	0.87	Passed	
Rotation		N/A	Up to 12.475 m. span. See calc
Local stability of notch	0.00	Passed	
Overall		**Passed**	

Structural integrity

Tension capacity = 121.5 kN. 486% of V_{Ed}
Beam web in bearing governs