AXIOM
STRUCTURES

85 CAMDEN MEWS,
LONDON NW1

STRUCTURAL CALCULATIONS

PART B

INTERNAL ALTERATIONS

Notes:

Beams designed to BS and to limit deflection to

min. span/ 300 under service Dead + Imposed loads

Span/ 360 – 500 under Imposed load

Unless noted otherwise for enhancement performance

Existing Brickwork = assumed allowable service bearing pressure of 0.43N/mm2 in padstone designs unless noted otherwise.

Frames to provide stability at the rear are designed to limit sway of H / 300

Job Number: 15005
Date issue: January 2019
Prepared by: KK / AP

AXIOM STRUCTURES

RR1 - Timber Roof Rafters

Span max =	2.95 m				
Slope =	26 degree				
Loading:					
Dead =	0.9 kN/m2	including self weigth of joists			
Imposed (snow) =	0.6 kN/m2				
From Tedds use =	**47x150 C24 at 400 c/c**	max. clear span =	3.8 m		
	Screw fix and birds mouth to beams and supporting wall.				

Uplift to RR1 - Timber Roof Rafters:

qs =			0.70					
Cp = Cpe + Cpi =			-0.80	(-0.6-0.2)				
Fw = qs x Cp =			-0.56 kN/m2					
Uplift at edge x 2.95/2 =	1.475	x	-0.56	x	1.4	=	-1.2 kN/m	
Res = SWt Roof x 2.95/2 =	1.475	x	0.25	x	1	=	0.4 kN/m	
					uplift	**=**	**-0.8 kN/m**	
use min. 2x5dia screws x 100mm long = 2x14.4N x 35mm						=	1.008 kN	
(standard penetration)	K52 =		1.25				1.26 KN short term	
	per m = 1/0.4 =		2.50 joists per meter				3.15 kN/m	Uplift OK

CJ1 - Ceiling Joists

Span =	2.2 m			
Loading:				
Dead =	0.30 kN/m2	including self weigth of joists		
Imposed =	0.3 kN/m2			
From Trada Tables for =	**47x120 C24 at 400 c/c**	max. clear span =	2.65 m	OK

TP1 - Timber Purlin:

Loading:

	L		DL+IL kN/m2		
New RR x 3.2m/2	1.6	x	1.5	=	2.4 kN/m
Ceiling x 2.1/2	1.1	x	0.6	=	0.6 kN/m
Swt					0.1 kN/m
			Sum sls:		3.1 kN/m

L = span	2.9 m
w =	3.1 kN/m
Msls = ql^2/8 =	3.2 kNm
EIreq (Lx0.003) =	326.0 kNm2
R sls =	4.5 kN

Design:

Grade C24

		Bending		Shear	
E =	7200 N/mm2	smII =	7.5 N/mm2	tII =	0.71 N/mm2
N =	1	K2 =	1		
b =	75 mm	K3 =	1.25 medium term		
N x b =	**75** mm	K7 =	1.03 depth		
h =	**225** mm	K8 =	1.1 load-sharing		
		K9 =	1 number for E		
A =	16875 mm2				
Ixx = bh3/12 =	71191406 mm4				
Iyy = b3h/12 =	7910156 mm4				
Zxx = bh2/6 =	632813 mm3				
		Mrd = Z*sm*K2*K3*K7*K8		Vc,rd = 2/3*A*tIIsv*K2*K3*K8	
K9*EI =	512.58 kNm2	Mrd =	6.7 kNm	Vc,rd =	11.0 kN
	OK		OK		OK

Use: 1x75x225C24

TP2 - Timber Purlin:

Loading:

	L		DL+IL kN/m2		
New RR x 3.2m/2	1.6	x	1.5	=	2.4 kN/m
Swt					0.1 kN/m
			Sum sls:		2.5 kN/m

L = span	2.0 m
w =	2.5 kN/m
Msls = ql^2/8 =	1.2 kNm
EIreq (Lx0.003) =	85.1 kNm2
R sls =	2.5 kN

Design:

Grade C24

		Bending		Shear	
E =	7200 N/mm2	smII =	7.5 N/mm2	tII =	0.71 N/mm2
N =	1	K2 =	1		
b =	75 mm	K3 =	1.25 medium term		
N x b =	**75** mm	K7 =	1.03 depth		
h =	**225** mm	K8 =	1.1 load-sharing		
		K9 =	1 number for E		
A =	16875 mm2				
Ixx = bh3/12 =	71191406 mm4				
Iyy = b3h/12 =	7910156 mm4				
Zxx = bh2/6 =	632813 mm3				
		Mrd = Z*sm*K2*K3*K7*K8		Vc,rd = 2/3*A*tIIsv*K2*K3*K8	
K9*EI =	512.58 kNm2	Mrd =	6.7 kNm	Vc,rd =	11.0 kN
	OK		OK		OK

Use: 1x75x225C24

P1 - Timber Post to support Timber Purlin

F sls = TP1 + TP2=	6.9 kN
M sls = 0.05 x F sls	0.35 kNm

Try: 2x47x97C16

Le = 1 m

Grade C16

		Bending		Shear	
E =	5800 N/mm2	smII =	5.3 N/mm2	tII =	0.67 N/mm2
		K2 =	1		
b = y	94 mm	K3 =	1.25 medium term		
h = x	97 mm	K7 =	1.13 depth		
		K8 =	1 load-sharing		
N =	2	K9 =	1.14 number for E		
A =	9118 mm2				
Ixx = bh3/12 =	7149272 mm4				
Iyy = b3h/12 =	6713887 mm4				
Zxx = bh2/6 =	147408 mm3	K9*EI =	47.27 kNm2		

Compression:

scII =	6.8 N/mm2	(compression parallel to grain)
Ley =	1000 mm	
iyy = (Iyy/A)^0.5	27.1 mm	
E/scII =	682	
lambda y = le/I =	37	
K12 =	**0.803**	

Pc = sc*A*K2*K3*K12*K8

Pcy = 62.2 kN

Mc = Z*sm*K2*K3*K7*K8		Vc = 2/3*A*t*K2*K3*K8	
Mc =	1.1 kNm	Vc =	5.1 kN

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Interaction check compression with bending:

F =	6.9 kN				
M =	0.3 kNm				
Keu = Euler Coeficient = 1-(1.5*S-c,a*K12)/s-e					
S-c,a = F/A =	0.76 N/mm2				
K12 =	0.803		Keu =	0.98	
S-e = pi^2*Emin/(lambda)^2 =		42.2 N/mm2			
	Check:	Mmax/Mc*Keu =		0.32	
		F/Pcy =		0.11	
		Mmax/Mc*Keu + F/Pcy =		0.43	< 1
				OK	**Use: 2x47x97C16**

<u>LT - Timber Lintel:</u>

Loading:

	L		DL+IL kN/m2		
New RR x 2.7m/2	1.4	x	1.5	=	2.0 kN/m
Swt					0.1 kN/m
			Sum sls:		2.1 kN/m

L = span	2.8 m
w =	2.1 kN/m
Msls = ql^2/8 =	2.0 kNm
EIreq (Lx0.003) =	197.7 kNm2
R sls =	2.9 kN

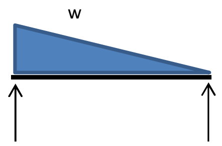
Design:

Grade C24

		Bending		Shear	
E =	7200 N/mm2	smII =	7.5 N/mm2	tII =	0.71 N/mm2
N =	2	K2 =	1		
b =	47 mm	K3 =	1.25 medium term		
N x b =	**94** mm	K7 =	1.08 depth		
h =	**150** mm	K8 =	1.1 load-sharing		
		K9 =	1.14 number for E		
A =	14100 mm2				
Ixx = bh3/12 =	26437500 mm4				
Iyy = b3h/12 =	10382300 mm4				
Zxx = bh2/6 =	352500 mm3				
		Mrd = Z*sm*K2*K3*K7*K8		Vc,rd = 2/3*A*tIIsv*K2*K3*K8	
K9*EI =	217.00 kNm2	Mrd =	3.9 kNm	Vc,rd =	9.2 kN
	OK		OK		OK

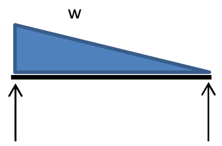
Use: 2x47x150C24

Hip rafter 1- check simply supported:



Loading - maximum w:		DL+IL		DL+IL	
Roof x (1.1m/2+ 0.8m/2)	0.95 x		1.4 =	1.33	
		Sum:		1.33	kN/m
L=	1.5	m			
from Tedds refer to next page:	**use 2x47x150C24**				
Ra sls=	0.65	kN			
Rb sls=	0.33	kN			

Hip rafter 2- check simply supported:



Loading - maximum w:		DL+IL		DL+IL	
Roof x (2.9m/2+ 1.9m/2)	2.4 x		1.4 =	3.36	
		Sum:		3.36	kN/m
L=	3.8	m			
from Tedds refer to next page:	**use 2x47x200C24**		**or alternatively flitch beam 2x47x150C24 with full depth 10thk MS Plate**		
Ra sls=	4.6	kN	Connect to steel beam via Maxi Speedy joists hangers SWL=5.9kN, hence OK		
Rb sls=	2.5	kN			

RB1 - Steel beam

Loadings:

	L		DL+IL kN/m2		DL+IL	
RR x 2.9m/2	1.5	x	1.5	=	2.2	kN/m
Ceiling x 0.4	0.4	x	0.6	=	0.2	kN/m
swt					0.3	
			Sum sls:		2.7	kN/m

L = span =	2.2	m							
Muls =	2.5	kNm	**for 152x152x23 UC**		**S355**				
			Mb = (le = 3 m)	45.4	kNm	(from Blue Book)		OK	
Ireq = (def<l/360)	66	cm4	Ixx =	1250	cm4	(from Blue Book)			
			def =	0.32	mm	L/def =	6807	OK	
Rsls A =	3.0	kN							
Ruls A=	4.5	kN							

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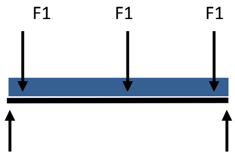
RB2 - Steel beam

Loadings:

	L		DL+IL		DL+IL
RR x 0.7m/2	0.4	x	1.5	=	0.5 kN/m
Ceiling x 0.4	0.4	x	0.6	=	0.2 kN/m
Swt included					
			Sum sls:		0.8 kN/m



Point load:

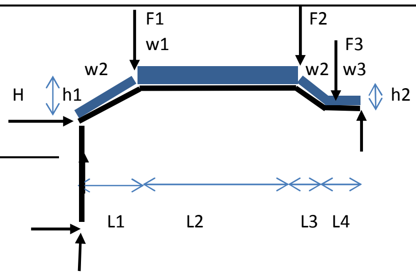
F1=TP2 = at 0, 1.1m, 2.2m= 2.5 kN

L = span = 2.2 m

Rsls A (from Tedds) = 4.9 kN
Rsls B (from Tedds) = 4.9 kN

From Tedds calculation refer to next page min needed section is = 152x152x23UC S355

RB3 - Cranked steel beam



Loadings:

	L		DL+IL kN/m2		DL+IL
w1:					
RR x 4m/2	2.0	x	1.5	=	3.0 kN/m
Ceiling x 2.2m/2	1.1	x	0.6	=	0.7 kN/m
Swt included					
			Sum sls:		3.7 kN/m
	2.0				

	L		DL+IL kN/m2		DL+IL
w2:					
RR x 0.4m	0.4	x	1.5	=	0.6 kN/m
Swt included					
			Sum sls:		0.6 kN/m

	L		DL+IL kN/m2		DL+IL
w2:					
RR x 2m	1.0	x	1.5	=	1.5 kN/m
Swt included					
			Sum sls:		1.5 kN/m

Point load:

F1= RB1 + Hip Rafter= 9.6 kN
F2=RB2 + Hip Rafter= 11.5 kN
F3= Hip Rafter= 6.6 kN

Wind load:

H= 0.75kN/m2 x 0.5 x 4.1m x 2.2m= 3.4 kN

L1= 2.1 m
L2= 2.6 m
L3= 0.65 m
L4= 1.4 m

h1= 1.10 m
h2= 0.45 m

L=total= actual length= 7.2 m

Refer to Robot use: **203x203x60 UC S355**

Muls =	56 kNm			
Mb = (le = 4.7m) =	168 kNm	(from Blue Book)	M/Mb =	0.33 < 1.0 OK
def (from Robot)	13.8 mm		L/def =	522 > 360 OK

Ra sls= vertical= 22.7 kN
Ra sls = horizontal= 5.6 kN

Rb sls= 23.5 kN
Rb sls = horizontal= 5.6 kN

Column below RB3 check:

Fsls =	22.7 kN				
Fuls =	34.1 kN				
Muls = Robot=	16.8 kNm				
sway def= (IL+WIND)=	4.5 mm	H/def=	444.4 > 300	hence OK	

Try 100x100x10.0 SHS S355:

Le =	2.0 m	
Pcy =	807.0 kN	(Blue book)
Mcx=	33.0 kNm	

<u>Interaction check compression with bending</u>

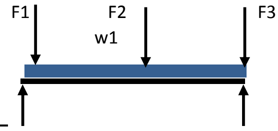
Mxx/Mbs + Myy/Mcy + F/Pcy =	Mxx/Mbs =	0.00				
	Myy/Mcy =	0.51				
	F/Pcy =	0.04	0.55	<	1.0	OK

Use100x100x10.0 SHS S355

<u>Connection 4M20 bolts at 100 crs =</u>

Mc for 4M20 8.8 grade = 2 x 137 kN x 0.10= 27.4 kNm > 16.8 kNm OK

<u>**RB4=CB1=**</u>



Loadings:

	L		DL+IL kN/m2		DL+IL
w1:					
RR x 0.4m	0.4	x	1.5	=	0.6 kN/m
Swt incuded					
			Sum sls:		0.6 kN/m

Point load:

F1= RB3+ TP1 + TP2=	30.4 kN
F2=TP1 + TP2=	6.9 kN
F3= RB3+ TP1 + TP2=	30.4 kN
L=span==	2.4 m

<u>**Refer to Robot use:**</u> **203x203x46 UC S355**

Rsls A (from Tedds) =	35.1 kN
Rsls B (from Tedds) =	35.1 kN

Lpad reqire 100w = 0.25 m (0.95 N/mm2 = brick stress sls) **use min 300mm bearin either end**

From Tedds calculation refer to next page min needed section is = 203x203x46 UC S355

FJ1 - Timber Floor Joists

Span =	3.2 m				
Loading:					
Dead =	0.75 kN/m2	including self weigth of joists			
Imposed =	1.5 kN/m2				
From Trada Tables for =	**47x200 C24 at 400 c/c**	max. clear span =	3.8 m	OK	

Trimmer in stair void:

Loading:

	L		DL+IL		
Stairs x 2m/2	1	x	2.3	=	2.3 kN/m
1st Floor x 0.4m	0.4	x	2.3	=	0.9 kN/m
Stud wall x 2.7m	2.7	x	0.5	=	1.4 kN/m
swt					1.0 kN/m
			Sum sls:		3.3 kN/m

L= span	3.2 m	(as measured on plan)
w=	3.3 kn/m	
Msls = ql^2/8 =	4.2 knm	
Eireq (Lx0.0003) =	465.0 knm2	
R=	5.2 kn	

Design:

Grade C24

		Bending		Shear	
E =	7200 n/mm2	smII =	7.5 N/mm2	tII =	0.71 N/mm2
		K2 =	1		
b =	**94** mm	K3 =	1		
h =	**200** mm	K7 =	1.05 depth		
		K8 =	1.1 load-sharing		
N =	2	K9 =	1.14 number for E		
A =	18800 mm2				
Ixx=bh3/12 =	62666667 mm4				
Iyy = b3h/12 =	13843067 mm4				
Zxx =bh2/6 =	626666.67 mm3				
		Mc= Z*sm*K2*K3*K7*K8		Vc= 2/3*A*tIIsv*K2*K3*K8	
K9*EI =	514.37 kNm2	Mc =	5.4 kNm	Vc =	9.8 kN
	OK		OK		OK

Use 2x47x200C24

1B1-1- Steel beam

Loadings:

	L		DL+IL kN/m2		DL+IL
1st Floor x 4.4m/2	2.2	x	2.3	=	5.1 kN/m
swt					0.3
			Sum sls:		5.4 kN/m

L = span =	2.9 m					
Muls =	8.5 kNm		**for 152x152x23 UC**	**S355**		
		Mb = (le = 3 m)		45.4 kNm	(from Blue Book)	OK
Ireq = (def<l/360)	299 cm4		Ixx =	1250 cm4	(from Blue Book)	
			def =	1.93 mm	L/def = 1505	OK
Rsls A =	7.8 kN					
Lpad reqire 100w =	0.05 m	(0.95 N/mm2 = brick stress sls)			**use PS1: 150d x 100w x 330l**	
Ruls A=	11.7 kN					

1B1-2- Steel beam

Loadings:

	L		DL+IL kN/m2		DL+IL	
1st Floor x 2.1m/2	1.1	x	2.3	=	2.4	kN/m
Timber wall x 3m	3	x	0.9	=	2.7	kN/m
swt					0.3	
			Sum sls:		5.4	kN/m

L = span =	3.0 m				
Muls =	9.1 kNm	**for 152x152x23 UC**	**S355**		
		Mb = (le = 3 m)	45.4 kNm	(from Blue Book)	OK
Ireq = (def<l/360)	335 cm4	Ixx =	1250 cm4	(from Blue Book)	
		def =	2.23 mm	L/def = 1346	OK
Rsls A =	8.1 kN				
Lpad reqire 100w =	0.06 m	(0.95 N/mm2 = brick stress sls)		**use PS1: 150d x 100w x 330l**	
Ruls A=	12.2 kN				

1B1-3- Steel beam

Loadings:

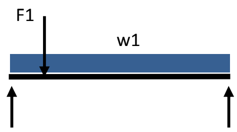
	L		DL+IL kN/m2		DL+IL	
1st Floor x 3m/2	1.5	x	2.3	=	3.5	kN/m
swt					0.3	
			Sum sls:		3.8	kN/m

L = span =	2.1 m				
Muls =	3.1 kNm	**for 152x152x23 UC**	**S355**		
		Mb = (le = 3 m)	45.4 kNm	(from Blue Book)	OK
Ireq = (def<l/360)	80 cm4	Ixx =	1250 cm4	(from Blue Book)	
		def =	0.37 mm	L/def = 5667	OK
Rsls A =	3.9 kN				
Lpad reqire 100w =	0.03 m	(0.95 N/mm2 = brick stress sls)		**use PS1: 150d x 100w x 330l**	
Ruls A=	5.9 kN				

1B6-1 - Steel beam

Loadings w1:

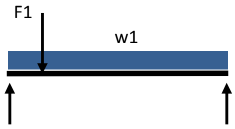
	L		DL+IL		DL+IL	
1st Floor x 0.4m/2	0.2	x	2.3	=	0.5	kN/m
Swt included						
			Sum sls:		0.5	kN/m



F1 at 0.45m= non - structural chimney=	10.3 kN	*2.5kN/m2 x 1m x 4.1m
L =span=	2.9 m	
Rsls A (from Tedds)=	9.8 kN	
Rsls B (from Tedds) =	2.7 kN	

From Tedds calculation refer to next page min needed section is = 150x90x24PFC S355

1B6-2 - Steel beam



Loadings w1:

	L		DL+IL		DL+IL	
1st Floor x 2.9m/2	1.5	x	2.3	=	3.3	kN/m
Swt included						
		Sum sls:			3.3	kN/m

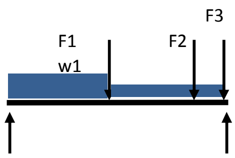
F1 at 0.2m= non - structural chimney= 6.2 kN *2.5kN/m2 x 0.6m x 4.1m

L =span= 2.0 m

Rsls A (from Tedds)= 10.9 kN
Rsls B (from Tedds) = 5.6 kN

From Tedds calculation refer to next page min needed section is = 150x90x24PFC S355

1B2- Steel beam



Loadings w1 (0-3.2m):

	L		DL+IL		DL+IL	
1st Floor x 5m/2	2.5	x	2.3	=	5.8	kN/m
Swt included						
		Sum sls:			5.8	kN/m

Loadings w2 (3.2m-6.7m):

	L		DL+IL		DL+IL	
1st Floor x 2.9m/2	1.5	x	2.3	=	3.3	kN/m
Swt included						
		Sum sls:			3.3	kN/m

F1 at 3.2m= 1B1-3= 5.9 kN 0.3DL + 0.7IL
F2 at 6.1m = 1B6-1 + 1B6-2= 20.7 kN 0.8DL + 0.2IL
F3 at 6.7m = RB3= 22.7 kN 0.6DL + 0.4IL

Rsls A (from Tedds)= 24.2 kN
Rsls B (from Tedds) = 59.8 kN

From Tedds calculation refer to next page min needed section is = 203x203x60UC S355

Lpad reqire 100w = 0.93 m (0.43 N/mm2 = existing brick stress sls)

use PS3: 300deep x 100wide x950long

Check def for DL+0.1IL :

Loadings w1 (0-3.2m):

	L		DL+0.1IL		DL+IL	
1st Floor x 5m/2	2.5	x	0.9	=	2.3	kN/m
Swt included						
			Sum sls:		2.3	kN/m	mm

Loadings w2 (3.2m-6.7m):

	L		DL+0.1IL		DL+IL	
1st Floor x 2.9m/2	1.5	x	0.9	=	1.3	kN/m
Swt included						
			Sum sls:		1.3	kN/m

	DL+0.1IL	
F1 at 3.2m= 1B1-3=	2.2	kN
F2 at 6.1m = 1B6-1 + 1B6-2=	17.0	kN
F3 at 6.7m = RB3=	14.5	kN

def= Tedds= 8.20 mm

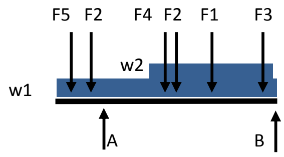
Natural frequency od beam:

f=18/(def)^0.5 6.29 Hz > 5Hz ok

1B3 - Steel beam

Loadings:

	L		DL+IL		DL+IL	
w1:						
1st Floor x 0.4m	0.4	x	2.3	=	0.9	kN/m
Swt included						
			Sum sls:		0.9	kN/m



	L		DL+IL		DL+IL	
w2:(2.5m - 4.6m)						
Cavity wall x 2.8	2.8	x	4.1	=	11.5	kN/m
Swt included						
			Sum sls:		11.5	kN/m

Point load:

F1=1B2 at 0.3m and 3m=	24.2	kN
F2= 1B1-1 at 2.3m=	7.8	kN
F3= 1B1-2 at 4.6m=	8.1	kN
F4= CB1 at 2.3m=	35.1	kN
F5=1B1-1 at 0.1m=	7.8	kN

L (A-B)= 0.8 m

L (B-C)= 3.8 m

Rsls A (from Tedds) = 83.5 kN

Rsls B (from Tedds) = 54.4 kN

Lpad reqire 200w = 0.19 m (0.95 N/mm2 = brick stress sls)

use min 250mm bearing on wall

From Tedds calculation refer to next page min needed section is = 203UC46 S355

Column under 1B3 check:

Fsls =	83.5	kN
Fuls =	125.3	kN
M nom= 0.15 x Fuls=	18.8	kNm

Try 200x100x8.0 RHS S355:

Le =	2.8	m	
Pcy =	1150.0	kN	(Blue book)
Mb =	95.0	kNm	
Mcy=	61.1	kNm	

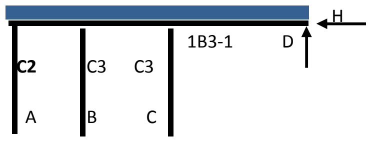
Interaction check compression with bending

Mxx/Mbs + Myy/Mcy + F/Pcy =	Mxx/Mbs =	0.00				
	Myy/Mcy =	0.31				
	F/Pcy =	0.11	0.42	<	1.0	OK

Use 200x100x8.0 RHS S355

Goal Frame:

Loadings:	L/H		DL+IL			
Roof x 3m/2	1.5	x	1.5	=	2.3	kN/m
1st Floor x 3.1m/2	1.6	x	2.3	=	3.6	kN/m
9" Wall x 2.3m	2.3	x	5	=	11.5	kN/m
Swt included						
			Sum sls:		17.3	kN/m



Wind load:
WL = 0.7 x A — Main house = 5.0x 8.5/2 = 21.3 m2

H = 14.9 kN

Span1 = L (A-B)= 2.7 m
Span2= L (B-C)= 1.2 m
Span2= L (C-D)= 1.8 m

Height of frame = 2.8 m

Head beam 1B3-1=1B5:

Refer to Robot use: **203x133x30UB S355**

Muls=	17.5 kNm		M/Mb =	0.25 < 1.0	OK
Mb = (le = 3m) =	69.2 kNm	(from Blue Book)			
Vuls=	38.3 kN				
Pv=	282 kN	(from Blue Book)	Vuls/Pv=	0.14 < 1.0	OK
def = DL+IL=	2 mm		L/def =	1350 > 360	OK
Rsls A (from Tedds)=	21.2 kN				
Rsls B (from Tedds) =	37.6 kN				
Rsls C(from Tedds)=	26.4 kN				
Rsls D (from Tedds)=	13.5 kN				
Lpad reqire 100w =	0.21 m	(0.43 N/mm2 = ex. brick stress sls)			

use 250mm bearing

From Tedds calculation refer to next page min needed section is = 203UB30 S355

Column C3:

Fsls = 37.6 kN
Fuls = 56.4 kN

Muls= from Robot= 22.6 kNm

sway def= (IL+WIND)= 9.0 mm — H/def= 311.1 > 300 hence OK

Try 200x100x8.0 RHS S355:
Le = 2.8 m
Pcy = 1150.0 kN (Blue book)
Mb = 95.0 kNm

Interaction check compression with bending
Mxx/Mbs + Myy/Mcy + F/Pcy = Mxx/Mbs = 0.24
Myy/Mcy = 0.00
F/Pcy = 0.05 — 0.29 < 1.0 OK

Use 200x100x8.0 RHS S355

Connection 4M20 bolts at 300 crs =

Ps (shear capacity)=	4 x 91.9kN = (blue book)	367.6 kN	>	20.8 kN	OK
Mc for 4M20 8.8 grade =	2 x 137 kN x 0.30 =	82.2 kNm	>	22.6 kNm	OK

Column C2:

Fsls =	8.1 kN								
Fuls =	12.2 kN								
Muls= from Robot=	12.0 kNm								

Try 203x203x46 S355

Le =	2.8 m	
Pcy =	1470.0 kN	(Blue book)
Mb =	143.0 kNm	

Interaction check compression with bending

Mxx/Mbs + Myy/Mcy + F/Pcy =	Mxx/Mbs	0.08					
	Myy/Mcy	0.00	=				
	F/Pcy =	0.01	=	0.09	<	1	OK

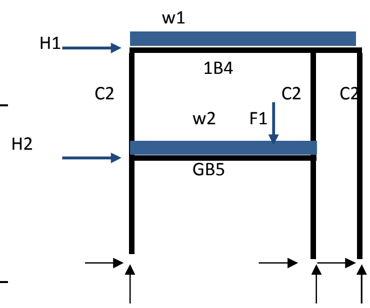
Use: 203UC46 S355

Connection 4M20 bolts at 200 crs =

Ps (shear capacity)=	4 x 91.9kN = (blue book)	367.6 kN	>	0.0 kN	OK
Mc for 4M20 8.8 grade =	2 x 137 kN x 0.20 =	54.8 kNm	>	12.0 kNm	OK

Steel Box Frame:

Loadings w1:	L/H		DL+IL		
Roof x 3m/2	1.5	x	1.5	=	2.3 kN/m
1st Floor x 3m/2	1.5	x	2.3	=	3.5 kN/m
215thk blockwork wall x 2m	2.0	x	3.9	=	7.8 kN/m
Swt included					
		Sum sls:			13.5 kN/m



Loadings w2:	L/H		DL+IL		
MD x 3m/2	1.5	x	7.3	=	11.0 kN/m
215thk blockwork wall x 0.3m	0.3	x	3.9	=	1.2 kN/m
Swt included					
		Sum sls:			12.1 kN/m

F1 = blockwork pier= 4.875 kN 0.5m x 2.5m x 3.9kN/m2

Wind load:

WL1 = 0.7 x A	Main house = 3x 8.5/2 =	12.8 m2
H1 =	8.9 kN	
WL2 = 0.7 x A	Main house = 1.5x 8.5/2 =	6.4 m2
H1 =	4.5 kN	

Span of frame =	4.2 m
Height of frame 1=	3.1 m
Height of frame 2=	2.9 m

Head beam 1B4:

	L	DL+IL kN/m2		DL+IL	
Loadings: as per above			=	13.5	kN/m
		Sum sls:		13.5	kN/m

L = span = 4.2 m

Refer to Robot use: **203x203x46 UC S355**

Muls =	22.1 kNm	1.2(DL+IL+WIND)			
Mb = (le = 3.5m) =	135 kNm	(from Blue Book)	M/Mb =	0.16 < 1.0	OK
def (from Robot)	2 mm		L/def =	2100 > 360	OK

Check Connection Beam 1B4 with Column C2

Muls= 22.1 kNm

Connection 4M20 bolts at 180 crs =

Ps (shear capacity)=	4 x 91.9kN =	(blue book)	367.6 kN	>	12.5 kN	OK
Mc for 4M20 8.8 grade =	2 x 137 kN x 0.18 =		49.32 kNm	>	22.1 kNm	OK

Provide 15mm thick Steel end plate S275, 6CFW

Column C2:

Fsls =	65.5 kN	
Fuls =	98.3 kN	
Muls= (from Robot)=	27.0 kNm	
def at top=	12.2 mm	
H1+H2/def=	487.7 >300	hence OK
def at middle=	9.2 mm	
H2/def=	315.2 >300	hence OK

Try 203x203x46UC S355:

Le =	3.1 m	
Pcy =	1470.0 kN	(Blue book)
Mb =	143.0 kNm	

Interaction check compression with bending

Mxx/Mbs + Myy/Mcy + F/Pcy =	Mxx/Mbs =	0.19				
	Myy/Mcy =	0.00				
	F/Pcy =	0.07	0.26	<	1	OK

Foundation pad under column C2

Fsls =	65.5 kN			
Pad swt =	4.0 kN			
Sum sls:	69.5 kN			
Area of foundations = sum / 150GBP	=	0.46 m2	Bmin =	0.68 m
Use B = A =	**=**	**1 m**		
Area	=	1.00 m2	OK	

Column C2 - RH side:

Fsls =	3.0 kN	
Fuls =	5.0 kN	
Muls= (from Robot)=	1.5 kNm	

Try 203x203x46UC S355:

Le =	6.0 m	
Pcy =	648.0 kN	(Blue book)
Mb =	99.0 kNm	

Interaction check compression with bending

Mxx/Mbs + Myy/Mcy + F/Pcy =	Mxx/Mbs =	0.02				
	Myy/Mcy =	0.00				
	F/Pcy =	0.01	0.02	<	1	OK

Check Connection Beam GB5 with Column C2

Muls=	38.7 kNm
Vuls =	33.4 kNm

Connection 2x3 M20 Bolts =

Ps (shear capacity)=	6 x 91.9N =	(blue book)	551.4 kN	>	33.4 kN	OK

Mc for 4 M20 8.8 grade (assumed - bottom line of bolts not working in this condition)=

Pnom (tension capaity)=	110 kN				
Mc= [2xPnom x 0.090x 90/225 + 2xPnom x 0.225]=		57.42 kNm	>	38.7 kNm	OK

Fc/t = M/a =	201.56 kN			
a = 203mm - 2 x 11mm / 2 =	192 mm			
Clamping width =	203 mm			
Steel grade and class =	**355** Mpa			
Flange compresion capacity =	792.7 kN	>	201.56 kN	**OK**

Check Ground Beam GB5:

Refer to Robot use: **203x203x46 UC S355**

Muls = from Robot=	39.0 kNm				
Mb = (le = 3.5m) =	135 kNm	(from Blue Book)	M/Mb =	0.29 < 1.0	OK
def (from Robot)	1 mm		L/def =	4200 > 360	OK

AXIOM STRUCTURES

GB1 Steel beam

Loadings:

	L		DL+IL kN/m2		DL+IL	
GF 150thk MD x 4.6m/2	2.3	x	7.3	=	16.8	kN/m
Stud wall x 2.8m	2.8	x	0.9	=	2.5	kN/m
swt					0.4	
			Sum sls:		19.7	kN/m

L = span = 2.9 m

Muls = 31.1 kNm — **for 152x152x30 UC S355**

Mb = (le = 3 m) 68.3 kNm (from Blue Book) OK

Ireq = (def<l/360) 1101 cm4 — Ixx = 1750 cm4 (from Blue Book)

def = 5.06 mm L/def = 573 OK

Rsls A = 28.6 kN

Ruls A= 42.9 kN <-- onto Padstone extended to retaining wall

GB2 Steel beam:

Loadings:

	L		DL+IL kN/m2		DL+IL	
GF 150thk MD x 5.4m/2	2.7	x	7.3	=	19.7	kN/m
swt					0.9	
			Sum sls:		20.6	kN/m

L = span = 6.7 m

Muls = 173.5 kNm — **for 254x254x89 UC S355**

Mb = (le = 7 m) 285 kNm (from Blue Book) OK

Ireq = (def<l/360) 14195 cm4 — Ixx = 14300 cm4 (from Blue Book)

def = 18.45 mm L/def = 363 OK

Rsls A = 69.0 kN

Ruls A= 103.6 kN <-- onto Padstone extended to retaining wall

GB4 Steel beam below cavity wall on gl. C:

Loadings:

	L		DL+IL kN/m2		DL+IL	
New RR x 3m/2	1.5	x	1.5	=	2.3	kN/m
Internal leaf of blockwork x 5.0	5	x	2.1	=	10.5	kN/m
GF 150thk MD x 0.4m	0.4	x	7.3	=	2.9	kN/m
swt					0.3	
			Sum sls:		16.0	kN/m

L = span = 3.0 m

Muls = 26.9 kNm — **for 200x90x30PFC S355**

Mb = (le = 3 m) 52.6 kNm (from Blue Book) OK

Ireq = (def<l/360) 987 cm4 — Ixx = 2520 cm4 (from Blue Book)

def = 3.26 mm L/def = 920 OK

Rsls A = 24.0 kN

Ruls A= 35.9 kN <-- Connected to steel beams/ columns with min 2M16 8.8 bolts

Ps= 2 x 58.9kN= 117.8kN

GB4 Steel beam below cavity wall on gl. B:

Loadings:

	L		DL+IL kN/m2		DL+IL	
Internal leaf of blockwork x 5.6	5.6	x	2.1	=	11.8	kN/m
GF 150thk MD x 0.4m	0.4	x	7.3	=	2.9	kN/m
swt					0.3	
			Sum sls:		15.0	kN/m

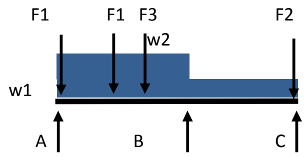
L = span = 2.0 m

Muls = 11.2 kNm **for 200x90x30PFC S355**
Mb = (le = 2 m) 739 kNm (from Blue Book) OK

Ireq = (def<l/360) 274 cm4 Ixx = 2520 cm4 (from Blue Book)
def = 0.60 mm L/def = 3311 OK

Rsls A = 15.0 kN
Ruls A= 22.5 kN <-- Connected to steel beams/ columns with min 2M16 8.8 bolts
Ps= 2 x 58.9kN= 117.8kN

GB3 - Steel beam



Loadings w1:

	L		DL+IL		DL+IL	
GF 150thk MD x 0.4m	0.4	x	7.3	=	2.9	kN/m
Swt included						
			Sum sls:		2.9	kN/m

Loadings w2 (0-1.6m):

	L		DL+IL		DL+IL	
Cavity wall x 3.1	3.1	x	4.1	=	12.7	kN/m
CB-1 / 3m=					11.7	kN/m
Swt included						
			Sum sls:		24.4	kN/m

Loadings w3 (3.2m-3.9m):

	L		DL+IL		DL+IL	
Cavity wall x 3.1	3.1	x	4.1	=	12.7	kN/m
Swt included						
			Sum sls:		12.7	kN/m

F1= GB1 and GB2 at 3.2m and 0.8m= 97.6 kN
F2= GB1 and 1B3 at 5.3m= 83.0 kN
F3= 1B3 at 1.6m= 83.5 kN

L (A-B)= 1.9 m
L (B-C)= 3.9 m

Rsls A (from Tedds)= 47.8 kN onto padstone extended to basement

Rsls B (from Tedds) = 289.3 kN Onto C1

Rsls C(from Tedds)= 92.0 kN onto padstone extended to basement

From Tedds calculation refer to next page min needed section is = 203UC46 S355

<u>Column C1 to support GB3</u>

Fsls =	289.3	kN
Fuls =	434.0	kN
Mnom = (0.1m + 0.15m/2) x GB3/3	25.3	kNm

Try 152x152x37UC S355

Le =	2.8	m	
Pcy =	920.0	kN	(Blue book)
Mb =	88.7	kNm	

<u>Interaction check compression with bending</u>

Mxx/Mbs + Myy/Mcy + F/Pcy =	Mxx/Mbs	0.29					
	Myy/Mcy	0.00	=				
	F/Pcy =	0.47	=	0.76	<	1	OK

Use: 152x152x37UC S355

<u>Foundation pad under column C1</u>

Fsls =	289.3	kN
Pad swt =	9.0	kN
Sum sls:	298.3	kN

Area of foundations = sum / 150GBP	=	1.99	m2		Bmin =	1.41	m
Use B = A =	**=**	**1.5**	**m**				
Area	=	2.25	m2	OK			

<u>L1 Concrete external lintel</u>

L = clear span = 1.9 m

	L/H		DL+IL kN/m2		DL+IL	
Internal brick/block x 0.4	0.4	x	2.0	=	0.8	kN/m
New RR x 4.1/2	2.1	x	1.5	=	3.1	kN/m
			Sum sls:		3.9	kN/m

Manufacture size to order	Section	P100	P150	P220	P255	S10	R15	R15A	R22	R22A	S15	R21	R21A
	Profile	65 x 100	65 x 150	65 x 220	65 x 255	100 x 100	100 x 140	140 x 100	100 x 215	215 x 100	150 x 140	140 x 215	215 x 140
	CLEAR SPAN	Service Moment (kNm)											
		0.96	1.06	2.61	3.09	1.60	2.79	4.82	5.88	12.47	8.61	8.08	13.33
600	**300**	37.75	37.75	70.68	91.44	50.67	71.24	117.11	152.89	210.67	110.56	180.00	280.00
750	**450**	21.33	23.56	53.00	68.58	35.56	53.43	87.83	114.67	158.00	82.92	135.00	210.00
900	**600**	13.65	15.08	37.12	43.95	22.76	39.68	68.55	83.63	126.40	66.33	108.00	168.00
1050	**750**	9.48	10.47	25.78	30.52	15.80	27.56	47.60	58.07	105.33	55.28	79.80	131.65
1200	**900**	6.97	7.69	18.94	22.42	11.61	20.24	34.98	42.67	90.29	47.38	58.63	96.73
1350	**1050**	5.33	5.89	14.50	17.17	8.89	15.50	26.78	32.67	69.28	41.46	44.89	74.06
1500	**1200**	4.21	4.65	11.46	13.56	7.02	12.25	21.16	25.81	54.74	36.85	35.47	58.51
1650	**1350**	3.41	3.77	9.28	10.99	5.69	9.92	17.14	20.91	44.34	30.61	28.73	47.40
1800	**1500**	2.82	3.11	7.67	9.08	4.70	8.20	14.16	17.28	36.64	25.30	23.74	39.17
2100	**1650**	2.37	2.62	6.44	7.63	3.95	6.89	11.90	14.52	30.79	21.26	19.95	32.91
2100	**1800**	2.02	2.23	5.49	6.50	3.37	5.87	10.14	12.37	26.24	18.11	17.00	28.04
2400	**1950**	1.74	1.92	4.73	5.61	2.90	5.06	8.74	10.67	22.62	15.62	14.66	24.18

SUPREME R15A 140d x 100w; max clear 1.9 m = 8.74 kN/m OK **Use: SUPREME R15A 140d x 100w**

<u>WP windpost check with EA:</u>

H = wind post height 3.0 m

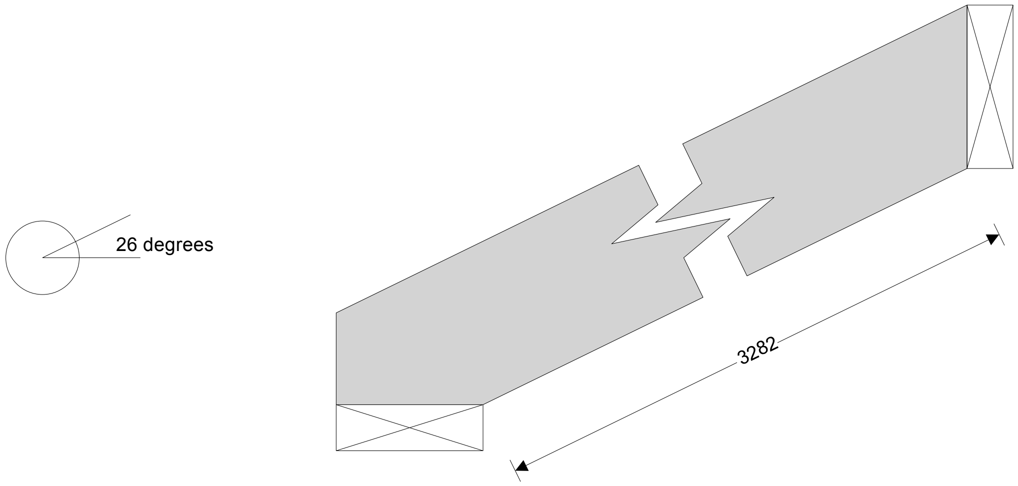
Wind load

p = wind load =	0.7	kN/m2
L = width of wind load = 3.6m/2 =	1.8	m
q = (p x L)/2 =	0.6	kN/m
Vuls = 1.4 x q x H x 0.5	1.3	kN
Muls,1 = 1.4 x q x H^2 / 8 =	1.0	kNm

From Tedds use 120x120x10 EA S275

TIMBER RAFTER DESIGN (BS5268-2:2002)

TEDDS calculation version 1.0.03



Rafter details

Breadth of timber sections	b = **47** mm
Depth of timber sections	h = **150** mm
Rafter spacing	s = **400** mm
Rafter slope	α = **26.0** deg
Clear span of rafter on horizontal	L_{clh} = **2950** mm
Clear span of rafter on slope	$L_{cl} = L_{clh} / \cos(\alpha)$ = **3282** mm
Rafter span	**Single span**
Timber strength class	**C24**

Section properties

Cross sectional area of rafter	$A = b \times h$ = **7050** mm^2
Section modulus	$Z = b \times h^2 / 6$ = **176250** mm^3
Second moment of area	$I = b \times h^3 / 12$ = **13218750** mm^4
Radius of gyration	$r = \sqrt{(I / A)}$ = **43.3** mm

Loading details

Rafter self weight	$F_j = b \times h \times \rho_{char} \times g_{acc}$ = **0.02** kN/m
Dead load on slope	F_d = **0.60** kN/m^2
Imposed load on plan	F_u = **0.60** kN/m^2
Imposed point load	F_p = **0.90** kN

Modification factors

Section depth factor	$K_7 = (300\text{ mm} / h)^{0.11}$ = **1.08**
Load sharing factor	K_8 = **1.10**

Consider long term load condition

Load duration factor	K_3 = **1.00**
Total UDL perpendicular to rafter	$F = F_d \times \cos(\alpha) \times s + F_j \times \cos(\alpha)$ = **0.237** kN/m
Notional bearing length	$L_b = F \times L_{cl} / [2 \times (b \times \sigma_{cp1} \times K_8 - F)]$ = **3** mm
Effective span	$L_{eff} = L_{cl} + L_b$ = **3285** mm

Check bending stress

Bending stress parallel to grain	σ_m = **7.500** N/mm^2

Permissible bending stress $\sigma_{m\_adm} = \sigma_m \times K_3 \times K_7 \times K_8 = \mathbf{8.904}$ N/mm^2

Applied bending stress $\sigma_{m\_max} = F \times L_{eff}^2 / (8 \times Z) = \mathbf{1.818}$ N/mm^2

PASS - Applied bending stress within permissible limits

Check compressive stress parallel to grain

Compression stress parallel to grain $\sigma_c = \mathbf{7.900}$ N/mm^2

Minimum modulus of elasticity $E_{min} = \mathbf{7200}$ N/mm^2

Compression member factor $K_{12} = \mathbf{0.55}$

Permissible compressive stress $\sigma_{c\_adm} = \sigma_c \times K_3 \times K_8 \times K_{12} = \mathbf{4.805}$ N/mm^2

Applied compressive stress $\sigma_{c\_max} = F \times L_{eff} \times (\cot(\alpha) + 3 \times \tan(\alpha)) / (2 \times A) = \mathbf{0.194}$ N/mm^2

PASS - Applied compressive stress within permissible limits

Check combined bending and compressive stress parallel to grain

Euler stress $\sigma_e = \pi^2 \times E_{min} / \lambda^2 = \mathbf{12.345}$ N/mm^2

Euler coefficient $K_{eu} = 1 - (1.5 \times \sigma_{c\_max} \times K_{12} / \sigma_e) = \mathbf{0.987}$

Combined axial compression and bending check $\sigma_{m\_max} / (\sigma_{m\_adm} \times K_{eu}) + \sigma_{c\_max} / \sigma_{c\_adm} = \mathbf{0.247} < \mathbf{1}$

PASS - Combined compressive and bending stresses are within permissible limits

Check shear stress

Shear stress parallel to grain $\tau = \mathbf{0.710}$ N/mm^2

Permissible shear stress $\tau_{adm} = \tau \times K_3 \times K_8 = \mathbf{0.781}$ N/mm^2

Applied shear stress $\tau_{max} = 3 \times F \times L_{eff} / (4 \times A) = \mathbf{0.083}$ N/mm^2

PASS - Applied shear stress within permissible limits

Check deflection

Permissible deflection $\delta_{adm} = 0.003 \times L_{eff} = \mathbf{9.856}$ mm

Bending deflection $\delta_b = 5 \times F \times L_{eff}^4 / (384 \times E_{mean} \times I) = \mathbf{2.523}$ mm

Shear deflection $\delta_s = 12 \times F \times L_{eff}^2 / (5 \times E_{mean} \times A) = \mathbf{0.081}$ mm

Total deflection $\delta_{max} = \delta_b + \delta_s = \mathbf{2.604}$ mm

PASS - Total deflection within permissible limits

Consider medium term load condition

Load duration factor $K_3 = \mathbf{1.25}$

Total UDL perpendicular to rafter $F = [F_u \times \cos(\alpha)^2 + F_d \times \cos(\alpha)] \times s + F_j \times \cos(\alpha) = \mathbf{0.431}$ kN/m

Notional bearing length $L_b = F \times L_{cl} / [2 \times (b \times \sigma_{cp1} \times K_8 - F)] = \mathbf{6}$ mm

Effective span $L_{eff} = L_{cl} + L_b = \mathbf{3288}$ mm

Check bending stress

Bending stress parallel to grain $\sigma_m = \mathbf{7.500}$ N/mm^2

Permissible bending stress $\sigma_{m\_adm} = \sigma_m \times K_3 \times K_7 \times K_8 = \mathbf{11.130}$ N/mm^2

Applied bending stress $\sigma_{m\_max} = F \times L_{eff}^2 / (8 \times Z) = \mathbf{3.307}$ N/mm^2

PASS - Applied bending stress within permissible limits

Check compressive stress parallel to grain

Compression stress parallel to grain $\sigma_c = \mathbf{7.900}$ N/mm^2

Minimum modulus of elasticity $E_{min} = \mathbf{7200}$ N/mm^2

Compression member factor $K_{12} = \mathbf{0.51}$

Permissible compressive stress $\sigma_{c\_adm} = \sigma_c \times K_3 \times K_8 \times K_{12} = \mathbf{5.508}$ N/mm^2

Applied compressive stress $\sigma_{c\_max} = F \times L_{eff} \times (\cot(\alpha) + 3 \times \tan(\alpha)) / (2 \times A) = \mathbf{0.353}$ N/mm^2

PASS - Applied compressive stress within permissible limits

Check combined bending and compressive stress parallel to grain

Euler stress $\sigma_e = \pi^2 \times E_{min} / \lambda^2 = \mathbf{12.325}$ N/mm^2

Euler coefficient $K_{eu} = 1 - (1.5 \times \sigma_{c\_max} \times K_{12} / \sigma_e) = \mathbf{0.978}$

Combined axial compression and bending check $\sigma_{m\_max} / (\sigma_{m\_adm} \times K_{eu}) + \sigma_{c\_max} / \sigma_{c\_adm} = \mathbf{0.368} < \mathbf{1}$

PASS - Combined compressive and bending stresses are within permissible limits

Check shear stress

Shear stress parallel to grain $\tau = \mathbf{0.710}$ N/mm^2

Permissible shear stress $\tau_{adm} = \tau \times K_3 \times K_8 = \mathbf{0.976}$ N/mm^2

Applied shear stress $\tau_{max} = 3 \times F \times L_{eff} / (4 \times A) = \mathbf{0.151}$ N/mm^2

PASS - Applied shear stress within permissible limits

Check deflection

Permissible deflection $\delta_{adm} = 0.003 \times L_{eff} = \mathbf{9.864}$ mm

Bending deflection $\delta_b = 5 \times F \times L_{eff}^4 / (384 \times E_{mean} \times I) = \mathbf{4.597}$ mm

Shear deflection $\delta_s = 12 \times F \times L_{eff}^2 / (5 \times E_{mean} \times A) = \mathbf{0.147}$ mm

Total deflection $\delta_{max} = \delta_b + \delta_s = \mathbf{4.744}$ mm

PASS - Total deflection within permissible limits

Consider short term load condition

Load duration factor $K_3 = \mathbf{1.50}$

Total UDL perpendicular to rafter $F = F_d \times \cos(\alpha) \times s + F_j \times \cos(\alpha) = \mathbf{0.237}$ kN/m

Notional bearing length $L_b = [F \times L_{cl} + F_p \times \cos(\alpha)] / [2 \times (b \times \sigma_{cp1} \times K_8 - F)] = \mathbf{6}$ mm

Effective span $L_{eff} = L_{cl} + L_b = \mathbf{3289}$ mm

Check bending stress

Bending stress parallel to grain $\sigma_m = \mathbf{7.500}$ N/mm^2

Permissible bending stress $\sigma_{m\_adm} = \sigma_m \times K_3 \times K_7 \times K_8 = \mathbf{13.355}$ N/mm^2

Applied bending stress $\sigma_{m\_max} = F \times L_{eff}^2/(8 \times Z) + F_p \times \cos(\alpha) \times L_{eff}/(4 \times Z) = \mathbf{5.595}$ N/mm^2

PASS - Applied bending stress within permissible limits

Check compressive stress parallel to grain

Compression stress parallel to grain $\sigma_c = \mathbf{7.900}$ N/mm^2

Minimum modulus of elasticity $E_{min} = \mathbf{7200}$ N/mm^2

Compression member factor $K_{12} = \mathbf{0.46}$

Permissible compressive stress $\sigma_{c\_adm} = \sigma_c \times K_3 \times K_8 \times K_{12} = \mathbf{6.057}$ N/mm^2

Applied compressive stress $\sigma_{c\_max} = F \times L_{eff} \times (\cot(\alpha) + 3 \times \tan(\alpha))/(2 \times A) + F_p \times \sin(\alpha)/A = \mathbf{0.251}$ N/mm^2

PASS - Applied compressive stress within permissible limits

Check combined bending and compressive stress parallel to grain

Euler stress $\sigma_e = \pi^2 \times E_{min} / \lambda^2 = \mathbf{12.320}$ N/mm^2

Euler coefficient $K_{eu} = 1 - (1.5 \times \sigma_{c\_max} \times K_{12} / \sigma_e) = \mathbf{0.986}$

Combined axial compression and bending check $\sigma_{m\_max} / (\sigma_{m\_adm} \times K_{eu}) + \sigma_{c\_max} / \sigma_{c\_adm} = \mathbf{0.466} < \mathbf{1}$

PASS - Combined compressive and bending stresses are within permissible limits

Check shear stress

Shear stress parallel to grain $\tau = \mathbf{0.710}$ N/mm^2

Permissible shear stress $\tau_{adm} = \tau \times K_3 \times K_8 = \mathbf{1.172}$ N/mm^2

Applied shear stress $\tau_{max} = 3 \times F \times L_{eff} / (4 \times A) + 3 \times F_p \times \cos(\alpha) / (2 \times A) = \mathbf{0.255}$ N/mm^2

PASS - Applied shear stress within permissible limits

Check deflection

Permissible deflection $\delta_{adm} = 0.003 \times L_{eff} = \mathbf{9.866}$ mm

Bending deflection $\delta_b = L_{eff}^3 \times (5 \times F \times L_{eff} / 384 + F_p \times \cos(\alpha) / 48) / (E_{mean} \times I) = \mathbf{6.731}$ mm

Shear deflection $\delta_s = 12 \times L_{eff} \times (F \times L_{eff} + 2 \times F_p \times \cos(\alpha))/(5 \times E_{mean} \times A) = \mathbf{0.249}$ mm

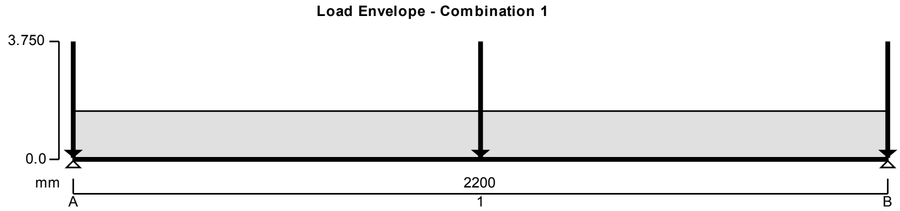
Total deflection $\delta_{max} = \delta_b + \delta_s = \mathbf{6.980}$ mm

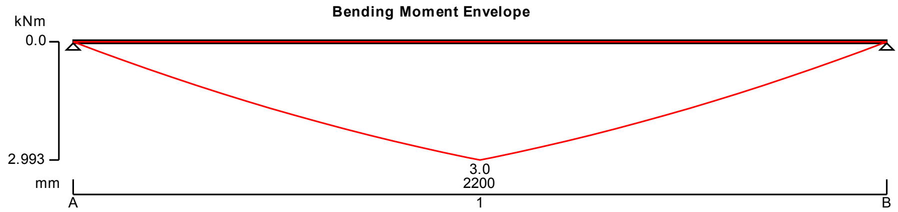
PASS - Total deflection within permissible limits

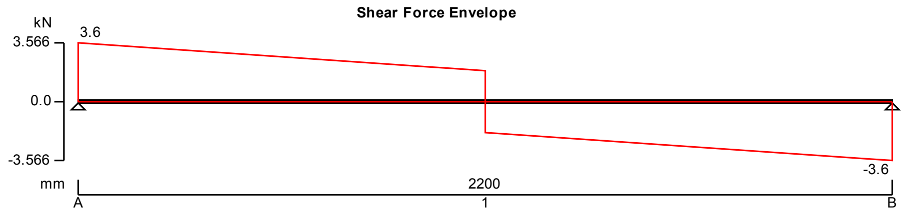
STEEL BEAM ANALYSIS & DESIGN (BS5950)

In accordance with BS5950-1:2000 incorporating Corrigendum No.1

TEDDS calculation version 3.0.05







Support conditions

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

Applied loading

Beam loads	Imposed self weight of beam × 1
	Imposed full UDL 0.8 kN/m
	Imposed point load 2.5 kN at 0 mm
	Imposed point load 2.5 kN at 1100 mm
	Imposed point load 2.5 kN at 2200 mm

Load combinations

Load combination 1	Support A	Dead × 1.50
		Imposed × 1.50
	Span 1	Dead × 1.50
		Imposed × 1.50
	Support B	Dead × 1.50

Imposed × 1.50

Analysis results

Maximum moment	M_{max} = **3** kNm	M_{min} = **0** kNm
Maximum shear	V_{max} = **3.6** kN	V_{min} = **-3.6** kN
Deflection	δ_{max} = **0.3** mm	δ_{min} = **0** mm
Maximum reaction at support A	R_{A_max} = **7.3** kN	R_{A_min} = **7.3** kN
Unfactored imposed load reaction at support A	$R_{A\_Imposed}$ = **4.9** kN	
Maximum reaction at support B	R_{B_max} = **7.3** kN	R_{B_min} = **7.3** kN
Unfactored imposed load reaction at support B	$R_{B\_Imposed}$ = **4.9** kN	

Section details

Section type **UC 152x152x23 (BS4-1)** Steel grade **S355**

Classification of cross sections - Section 3.5

Tensile strain coefficient ε = **0.88** Section classification **Semi-compact**

Shear capacity - Section 4.2.3

Design shear force F_v = **3.6** kN Design shear resistance P_v = **188.3** kN

PASS - Design shear resistance exceeds design shear force

Moment capacity - Section 4.2.5

Design bending moment M = **3** kNm Moment capacity low shear M_c = **60.5** kNm

Buckling resistance moment - Section 4.3.6.4

Buckling resistance moment M_b = **48.8** kNm M_b / m_{LT} = **48.8** kNm

PASS - Buckling resistance moment exceeds design bending moment

Check vertical deflection - Section 2.5.2

Consider deflection due to dead and imposed loads

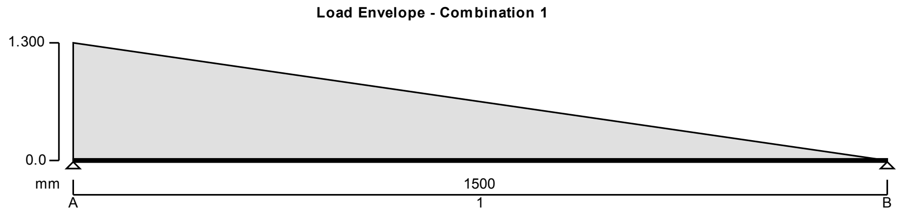
Limiting deflection δ_{lim} = **6.111** mm Maximum deflection δ = **0.339** mm

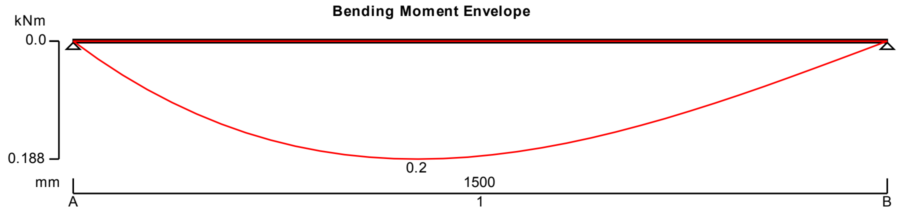
PASS - Maximum deflection does not exceed deflection limit

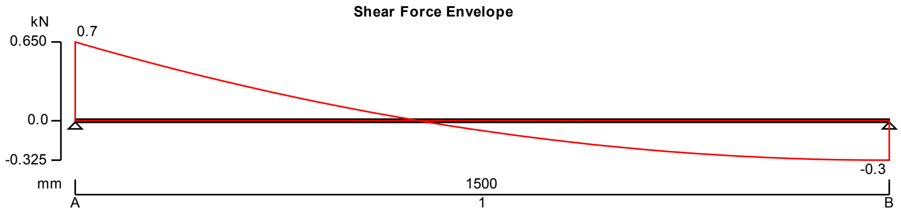
	Project				Job no.	
	85 Camden Mews				15005	
	Calcs for				Start page no./Revision	
	Hip rafter				1	
	Calcs by	Calcs date	Checked by	Checked date	Approved by	Approved date
	KL	17/12/2018	AP			

TIMBER BEAM ANALYSIS & DESIGN TO BS5268-2:2002

TEDDS calculation version 1.5.07







Applied loading

Beam loads

Imposed partial VDL 1.300 kN/m at 0 mm to 0.000 kN/m at 1500 mm

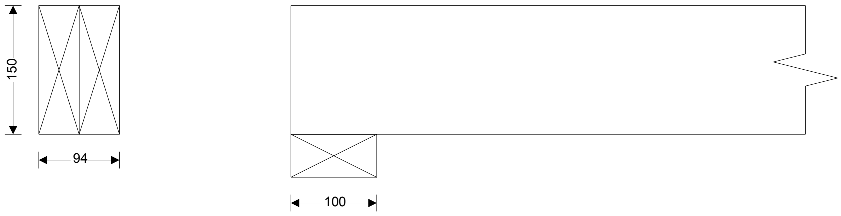
Load combinations

Load combination 1	Support A	Dead × 1.00
		Imposed × 1.00
	Span 1	Dead × 1.00
		Imposed × 1.00
	Support B	Dead × 1.00
		Imposed × 1.00

Analysis results

Maximum moment	M_{max} = **0.188** kNm	M_{min} = **0.000** kNm
Design moment	M = max(abs(M_{max}),abs(M_{min})) = **0.188** kNm	
Maximum shear	F_{max} = **0.650** kN	F_{min} = **-0.325** kN
Design shear	F = max(abs(F_{max}),abs(F_{min})) = **0.650** kN	
Total load on beam	W_{tot} = **0.975** kN	
Reactions at support A	R_{A_max} = **0.650** kN	R_{A_min} = **0.650** kN
Unfactored imposed load reaction at support A	$R_{A\_Imposed}$ = **0.650** kN	
Reactions at support B	R_{B_max} = **0.325** kN	R_{B_min} = **0.325** kN

Unfactored imposed load reaction at support B $R_{B\_Imposed}$ = **0.325** kN



Timber section details

Breadth of sections	b = **47** mm
Depth of sections	h = **150** mm
Number of sections in member	N = **2**
Overall breadth of member	b_b = N × b = **94** mm
Timber strength class	**C24**

Member details

Service class of timber	**1**
Load duration	**Medium term**
Length of bearing	L_b = **100** mm

Section properties

Cross sectional area of member	A = N × b × h = **14100** mm^2
Section modulus	Z_x = N × b × h^2 / 6 = **352500** mm^3
	Z_y = h × (N × b)^2 / 6 = **220900** mm^3
Second moment of area	I_x = N × b × h^3 / 12 = **26437500** mm^4
	I_y = h × (N × b)^3 / 12 = **10382300** mm^4
Radius of gyration	i_x = √(I_x / A) = **43.3** mm
	i_y = √(I_y / A) = **27.1** mm

Modification factors

Duration of loading - Table 17	K_3 = **1.25**
Bearing stress - Table 18	K_4 = **1.00**
Total depth of member - cl.2.10.6	K_7 = (300 mm / h)$^{0.11}$ = **1.08**
Load sharing - cl.2.10.11	K_8 = **1.10**
Minimum modulus of elasticity - Table 20	K_9 = **1.14**

Lateral support - cl.2.10.8

Ends held in position

Permissible depth-to-breadth ratio - Table 19	**3.00**
Actual depth-to-breadth ratio	h / (N × b) = **1.60**

PASS - Lateral support is adequate

Compression perpendicular to grain

Permissible bearing stress (no wane)	σ_{c_adm} = σ_{cp1} × K_3 × K_4 × K_8 = **3.300** N/mm^2
Applied bearing stress	σ_{c_a} = R_{A_max} / (N × b × L_b) = **0.069** N/mm^2
	σ_{c_a} / σ_{c_adm} = **0.021**

PASS - Applied compressive stress is less than permissible compressive stress at bearing

Bending parallel to grain

Permissible bending stress	σ_{m_adm} = σ_m × K_3 × K_7 × K_8 = **11.130** N/mm^2
Applied bending stress	σ_{m_a} = M / Z_x = **0.532** N/mm^2

$\sigma_{m\_a} / \sigma_{m\_adm}$ = **0.048**

PASS - Applied bending stress is less than permissible bending stress

Shear parallel to grain

Permissible shear stress $\tau_{adm} = \tau \times K_3 \times K_8$ = **0.976** N/mm^2

Applied shear stress $\tau_a = 3 \times F / (2 \times A)$ = **0.069** N/mm^2

τ_a / τ_{adm} = **0.071**

PASS - Applied shear stress is less than permissible shear stress

Deflection

Modulus of elasticity for deflection $E = E_{min} \times K_9$ = **8208** N/mm^2

Permissible deflection δ_{adm} = min(14 mm, 0.003 × L_{s1}) = **4.500** mm

Bending deflection δ_{b_s1} = **0.198** mm

Shear deflection δ_{v_s1} = **0.031** mm

Total deflection $\delta_a = \delta_{b\_s1} + \delta_{v\_s1}$ = **0.229** mm

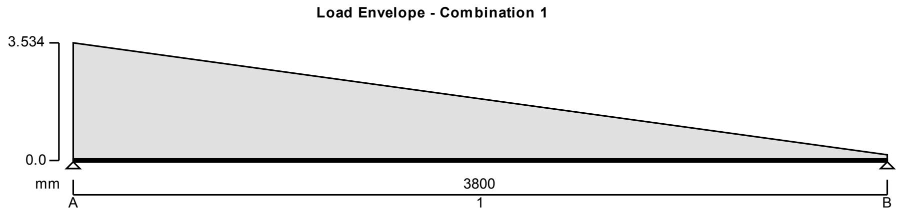
δ_a / δ_{adm} = **0.051**

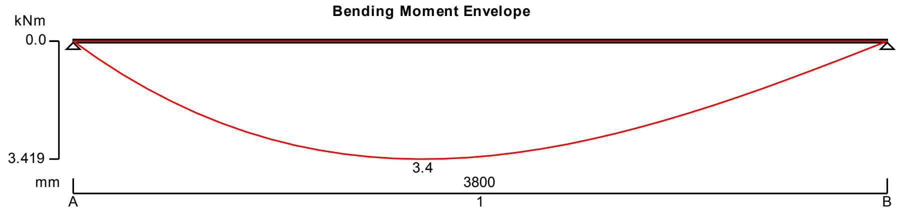
PASS - Total deflection is less than permissible deflection

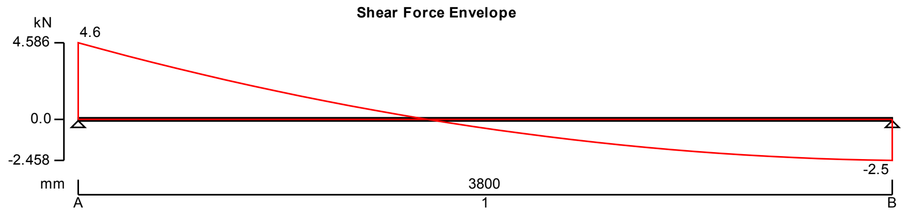
Tekla Tedds	Project 85 Camden Mews				Job no. 15005	
	Calcs for Hipped Beam				Start page no./Revision 1	
	Calcs by KK	Calcs date 17/12/2018	Checked by AP	Checked date	Approved by	Approved date

FLITCH BEAM ANALYSIS & DESIGN TO BS5268-2:2002

TEDDS calculation version 1.5.07







Applied loading

Beam loads

Imposed self weight of beam × 1

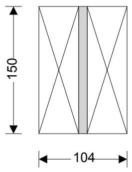
Imposed partial VDL 3.360 kN/m at 0 mm to 0.000 kN/m at 3800 mm

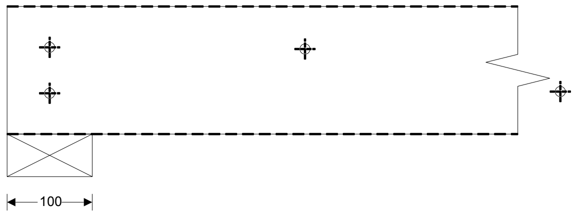
Load combinations

Load combination 1	Support A	Dead × 1.00
		Imposed × 1.00
	Span 1	Dead × 1.00
		Imposed × 1.00
	Support B	Dead × 1.00
		Imposed × 1.00

Analysis results

Design moment	M = **3.419** kNm	Design shear	F = **4.586** kN
Total load on beam	W_{tot} = **7.043** kN		
Reactions at support A	R_{A_max} = **4.586** kN	R_{A_min} = **4.586** kN	
Unfactored imposed load reaction at support A	$R_{A\_Imposed}$ = **4.586** kN		
Reactions at support B	R_{B_max} = **2.458** kN	R_{B_min} = **2.458** kN	
Unfactored imposed load reaction at support B	$R_{B\_Imposed}$ = **2.458** kN		





Timber section details

Breadth of section b = **47** mm | Depth of section h = **150** mm

Number of sections N = **2**

Timber strength class **C24**

Steel section details

Breadth of steel plate b_s = **10** mm | Depth of steel plate h_s = **150** mm

Number of steel plates in beam N_s = **1** | Steel stress p_y = **165** N/mm^2

Bolt diameter ϕ_b = **12** mm | Maximum bolt spacing S_{max} = **300** mm

Member details

Service class of timber **1** | Load duration **Medium term**

Length of bearing L_b = **100** mm

The beam is part of a load-sharing system consisting of four or more members

Lateral support - cl.2.10.8

Permiss.depth-to-breadth ratio **3.00** | Actual depth-to-breadth ratio **1.44**

PASS - Lateral support is adequate

Check bearing stress

Permissible bearing stress σ_{c_adm} = **3.300** N/mm^2 | Applied bearing stress σ_{c_a} = **0.488** N/mm^2

PASS - Applied compressive stress is less than permissible compressive stress at bearing

Bending parallel to grain

Permiss. timber bending stress σ_{m_adm} = **11.130** N/mm^2 | Applied timber bending stress σ_{m_a} = **3.212** N/mm^2

PASS - Timber bending stress is less than permissible timber bending stress

Permiss. steel bending stress p_y = **165.000** N/mm^2 | Applied steel bending stress $\sigma_{m\_a\_s}$ = **78.361** N/mm^2

PASS - Steel bending stress is less than permissible steel bending stress

Shear parallel to grain

Permissible shear stress τ_{adm} = **0.976** N/mm^2 | Applied shear stress τ_a = **0.162** N/mm^2

PASS - Applied shear stress is less than permissible shear stress

Deflection

Permissible deflection δ_{adm} = **11.400** mm | Total deflection δ_a = **6.277** mm

PASS - Total deflection is less than permissible deflection

Flitch plate bolting requirements

Bolts required at beam end N_{be} = **2.000** | Bolts required to beam length N_{bl} = **1.357**

- Provide a minimum of 2 No.12 mm diameter bolts at each support
- Provide 12 mm diameter bolts at a maximum of 300 mm centres along the length of the beam

Minimum bolt spacings

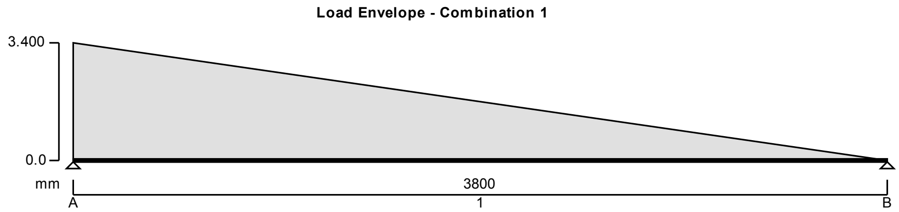
Minimum end spacing S_{end} = **48** mm | Minimum edge spacing S_{edge} = **48** mm

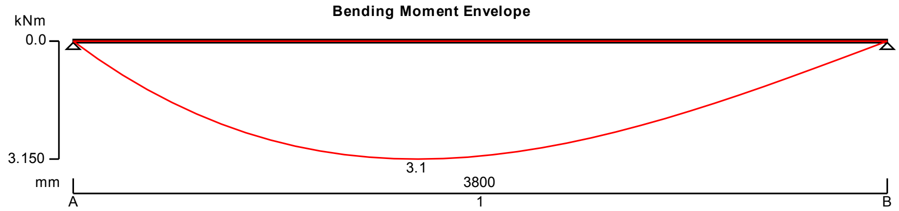
Minimum bolt spacing S_{bolt} = **48** mm

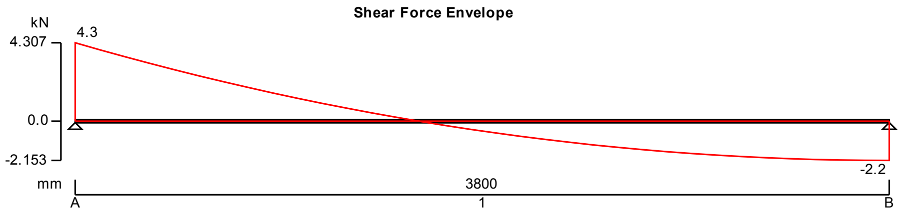
Minimum washer diameter ϕ_w = **36** mm | Minimum washer thickness t_w = **3.0** mm

TIMBER BEAM ANALYSIS & DESIGN TO BS5268-2:2002

TEDDS calculation version 1.5.07







Applied loading

Beam loads

Imposed partial VDL 3.400 kN/m at 0 mm to 0.000 kN/m at 3800 mm

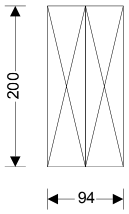
Load combinations

Load combination 1	Support A	Dead × 1.00
		Imposed × 1.00
	Span 1	Dead × 1.00
		Imposed × 1.00
	Support B	Dead × 1.00
		Imposed × 1.00

Analysis results

Maximum moment	M_{max} = **3.150** kNm	M_{min} = **0.000** kNm
Design moment	M = max(abs(M_{max}),abs(M_{min})) = **3.150** kNm	
Maximum shear	F_{max} = **4.307** kN	F_{min} = **-2.153** kN
Design shear	F = max(abs(F_{max}),abs(F_{min})) = **4.307** kN	
Total load on beam	W_{tot} = **6.460** kN	
Reactions at support A	R_{A_max} = **4.307** kN	R_{A_min} = **4.307** kN
Unfactored imposed load reaction at support A	$R_{A\_Imposed}$ = **4.307** kN	
Reactions at support B	R_{B_max} = **2.153** kN	R_{B_min} = **2.153** kN

Unfactored imposed load reaction at support B $R_{B\_Imposed}$ = **2.153** kN



Timber section details

Breadth of sections b = **47** mm

Depth of sections h = **200** mm

Number of sections in member N = **2**

Overall breadth of member $b_b = N \times b$ = **94** mm

Timber strength class **C24**

Member details

Service class of timber **1**

Load duration **Medium term**

Length of bearing L_b = **100** mm

Section properties

Cross sectional area of member $A = N \times b \times h$ = **18800** mm^2

Section modulus $Z_x = N \times b \times h^2 / 6$ = **626667** mm^3

$Z_y = h \times (N \times b)^2 / 6$ = **294533** mm^3

Second moment of area $I_x = N \times b \times h^3 / 12$ = **62666667** mm^4

$I_y = h \times (N \times b)^3 / 12$ = **13843067** mm^4

Radius of gyration $i_x = \sqrt{(I_x / A)}$ = **57.7** mm

$i_y = \sqrt{(I_y / A)}$ = **27.1** mm

Modification factors

Duration of loading - Table 17 K_3 = **1.25**

Bearing stress - Table 18 K_4 = **1.00**

Total depth of member - cl.2.10.6 $K_7 = (300 \text{ mm} / h)^{0.11}$ = **1.05**

Load sharing - cl.2.10.11 K_8 = **1.10**

Minimum modulus of elasticity - Table 20 K_9 = **1.14**

Lateral support - cl.2.10.8

Ends held in position

Permissible depth-to-breadth ratio - Table 19 **3.00**

Actual depth-to-breadth ratio $h / (N \times b)$ = **2.13**

PASS - Lateral support is adequate

Compression perpendicular to grain

Permissible bearing stress (no wane) $\sigma_{c\_adm} = \sigma_{cp1} \times K_3 \times K_4 \times K_8$ = **3.300** N/mm^2

Applied bearing stress $\sigma_{c\_a} = R_{A\_max} / (N \times b \times L_b)$ = **0.458** N/mm^2

$\sigma_{c\_a} / \sigma_{c\_adm}$ = **0.139**

PASS - Applied compressive stress is less than permissible compressive stress at bearing

Bending parallel to grain

Permissible bending stress $\sigma_{m\_adm} = \sigma_m \times K_3 \times K_7 \times K_8$ = **10.783** N/mm^2

Applied bending stress $\sigma_{m\_a} = M / Z_x = \mathbf{5.026}$ N/mm^2

$\sigma_{m\_a} / \sigma_{m\_adm} = \mathbf{0.466}$

PASS - Applied bending stress is less than permissible bending stress

Shear parallel to grain

Permissible shear stress $\tau_{adm} = \tau \times K_3 \times K_8 = \mathbf{0.976}$ N/mm^2

Applied shear stress $\tau_a = 3 \times F / (2 \times A) = \mathbf{0.344}$ N/mm^2

$\tau_a / \tau_{adm} = \mathbf{0.352}$

PASS - Applied shear stress is less than permissible shear stress

Deflection

Modulus of elasticity for deflection $E = E_{min} \times K_9 = \mathbf{8208}$ N/mm^2

Permissible deflection $\delta_{adm} = \min(14\text{ mm}, 0.003 \times L_{s1}) = \mathbf{11.400}$ mm

Bending deflection $\delta_{b\_s1} = \mathbf{8.989}$ mm

Shear deflection $\delta_{v\_s1} = \mathbf{0.392}$ mm

Total deflection $\delta_a = \delta_{b\_s1} + \delta_{v\_s1} = \mathbf{9.381}$ mm

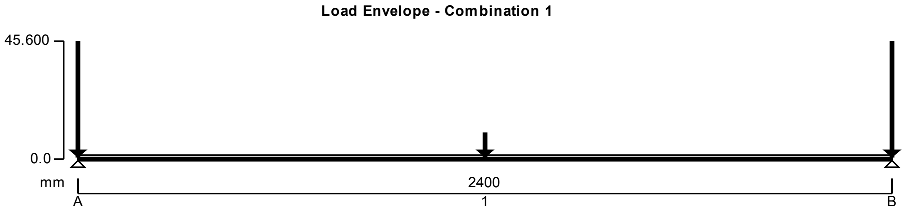
$\delta_a / \delta_{adm} = \mathbf{0.823}$

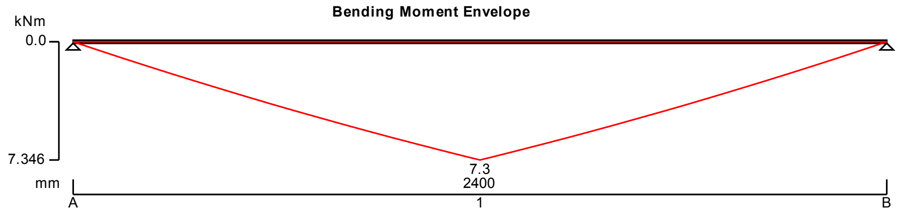
PASS - Total deflection is less than permissible deflection

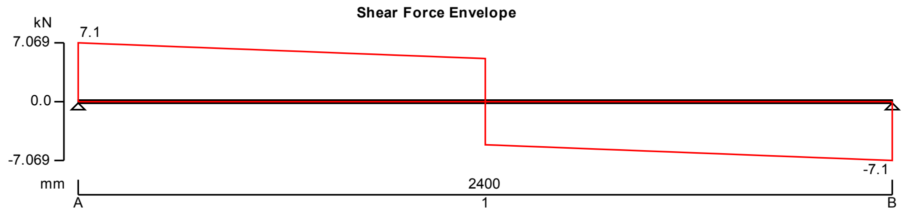
STEEL BEAM ANALYSIS & DESIGN (BS5950)

In accordance with BS5950-1:2000 incorporating Corrigendum No.1

TEDDS calculation version 3.0.05







Support conditions

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

Applied loading

Beam loads	Imposed self weight of beam × 1
	Imposed full UDL 0.6 kN/m
	Imposed point load 30.4 kN at 0 mm
	Imposed point load 6.9 kN at 1200 mm
	Imposed point load 30.4 kN at 2400 mm

Load combinations

Load combination 1	Support A	Dead × 1.50
		Imposed × 1.50
	Span 1	Dead × 1.50
		Imposed × 1.50
	Support B	Dead × 1.50

Imposed × 1.50

Analysis results

Maximum moment	M_{max} = **7.3** kNm	M_{min} = **0** kNm
Maximum shear	V_{max} = **7.1** kN	V_{min} = **-7.1** kN
Deflection	δ_{max} = **0.3** mm	δ_{min} = **0** mm
Maximum reaction at support A	R_{A_max} = **52.7** kN	R_{A_min} = **52.7** kN
Unfactored imposed load reaction at support A	$R_{A\_Imposed}$ = **35.1** kN	
Maximum reaction at support B	R_{B_max} = **52.7** kN	R_{B_min} = **52.7** kN
Unfactored imposed load reaction at support B	$R_{B\_Imposed}$ = **35.1** kN	

Section details

Section type **UC 203x203x46 (BS4-1)** Steel grade **S355**

Classification of cross sections - Section 3.5

Tensile strain coefficient ε = **0.88** Section classification **Semi-compact**

Shear capacity - Section 4.2.3

Design shear force F_v = **7.1** kN Design shear resistance P_v = **311.6** kN

PASS - Design shear resistance exceeds design shear force

Moment capacity - Section 4.2.5

Design bending moment M = **7.3** kNm Moment capacity low shear M_c = **174.1** kNm

Buckling resistance moment - Section 4.3.6.4

Buckling resistance moment M_b = **155.3** kNm M_b / m_{LT} = **155.3** kNm

PASS - Buckling resistance moment exceeds design bending moment

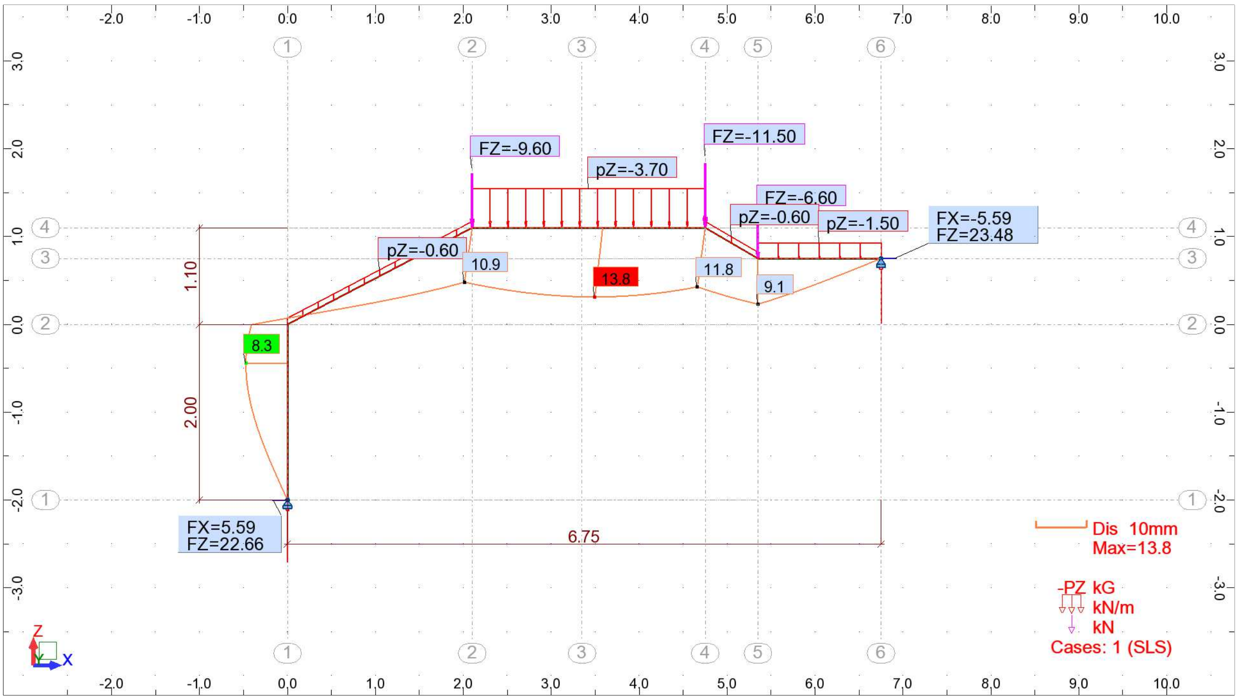
Check vertical deflection - Section 2.5.2

Consider deflection due to dead and imposed loads

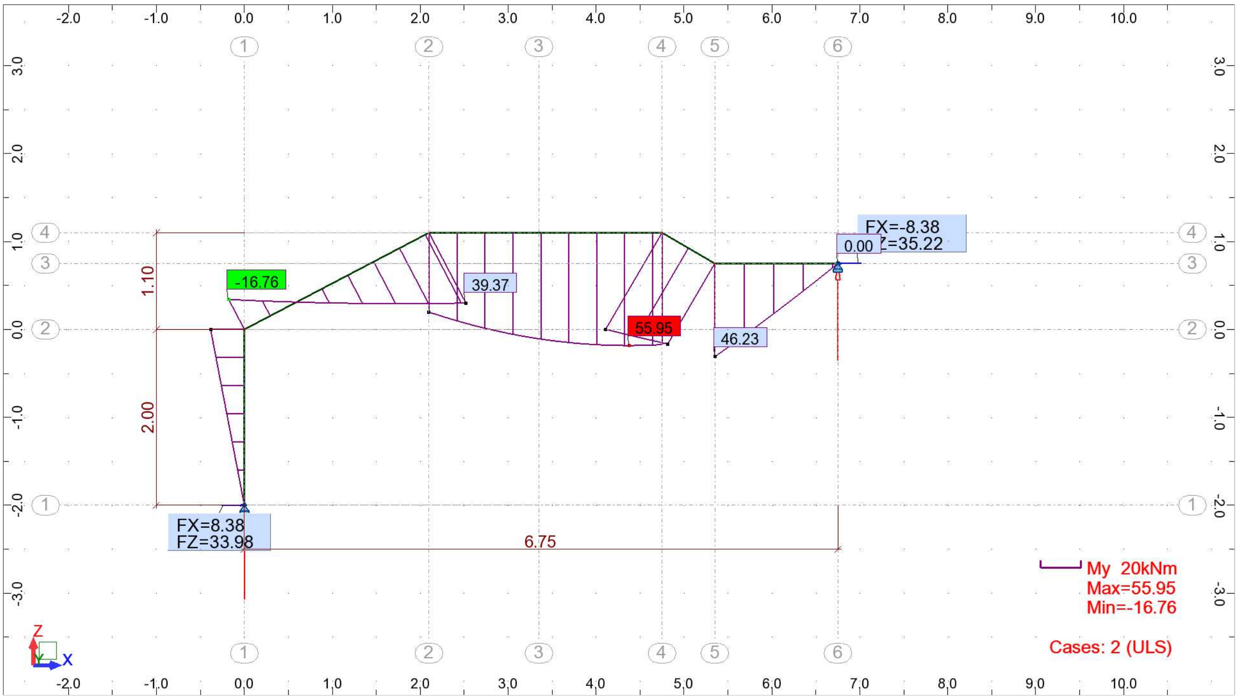
Limiting deflection δ_{lim} = **6.667** mm Maximum deflection δ = **0.261** mm

PASS - Maximum deflection does not exceed deflection limit

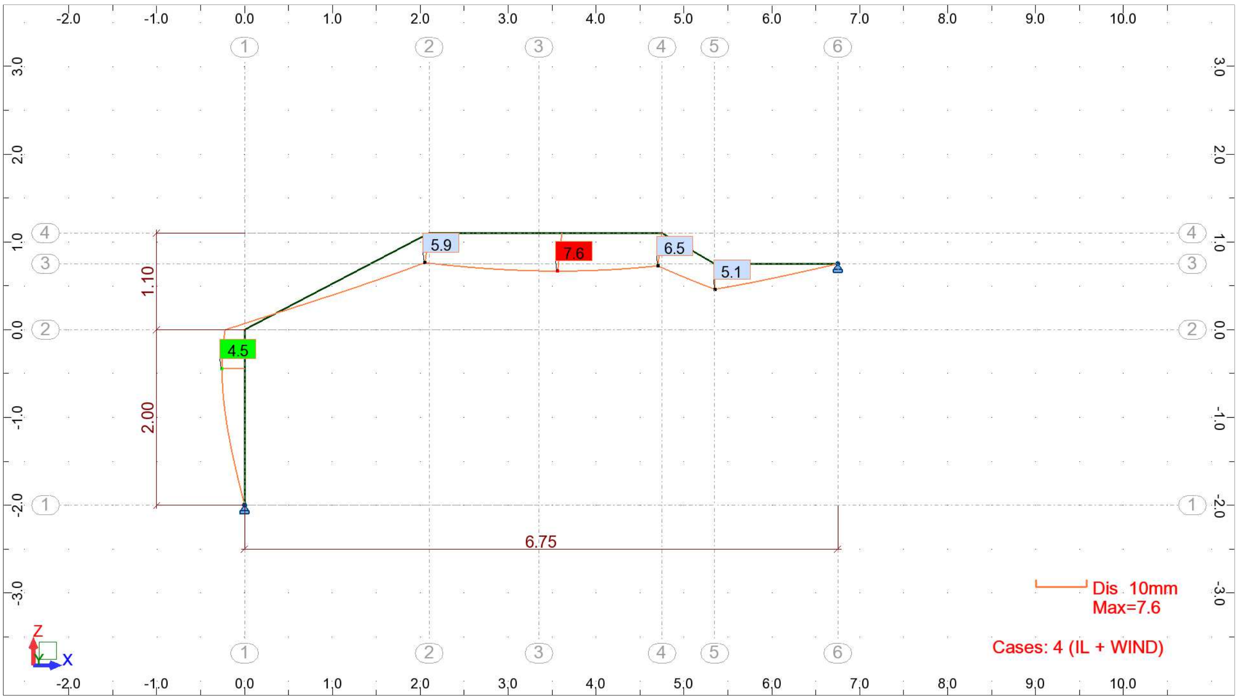
View - Exact deformation(s),Reaction forces(kN), Cases: 1 (SLS)



View - MY,Reaction forces(kN), Cases: 2 (ULS)



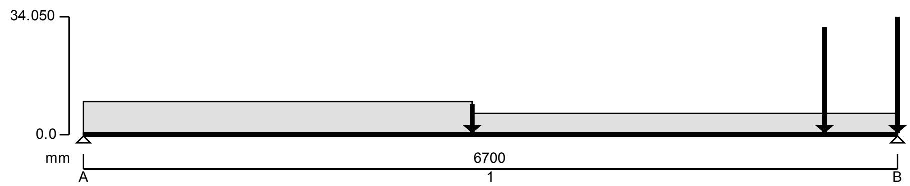
View - Exact deformation(s), Cases: 4 (IL + WIND)

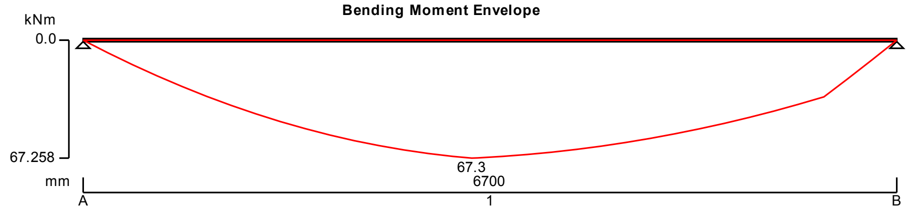


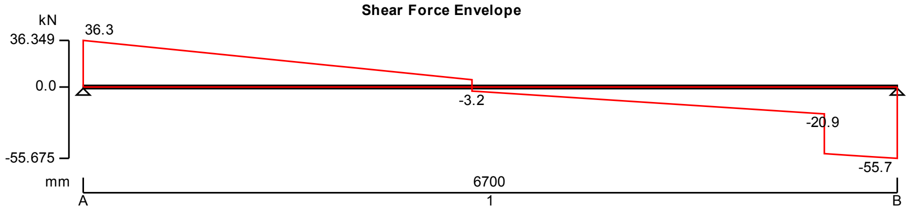
STEEL BEAM ANALYSIS & DESIGN (BS5950)

In accordance with BS5950-1:2000 incorporating Corrigendum No.1

TEDDS calculation version 3.0.05







Support conditions

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

Applied loading

Beam loads	Imposed self weight of beam × 1
	Imposed partial UDL 5.8 kN/m from 0 mm to 3200 mm
	Imposed partial UDL 3.5 kN/m from 3200 mm to 6700 mm
	Imposed point load 5.9 kN at 3200 mm
	Imposed point load 20.7 kN at 6100 mm
	Imposed point load 22.7 kN at 6700 mm

Load combinations

Load combination 1	Support A	Dead × 1.50
		Imposed × 1.50
	Span 1	Dead × 1.50
		Imposed × 1.50
	Support B	Dead × 1.50

Imposed × 1.50

Analysis results

Maximum moment	M_{max} = **67.3** kNm	M_{min} = **0** kNm
Maximum moment span 1 segment 1	$M_{s1\_seg1\_max}$ = **67** kNm	$M_{s1\_seg1\_min}$ = **0** kNm
Maximum moment span 1 segment 2	$M_{s1\_seg2\_max}$ = **67.3** kNm	$M_{s1\_seg2\_min}$ = **0** kNm
Maximum shear	V_{max} = **36.3** kN	V_{min} = **-55.7** kN
Maximum shear span 1 segment 1	$V_{s1\_seg1\_max}$ = **36.3** kN	$V_{s1\_seg1\_min}$ = **0** kN
Maximum shear span 1 segment 2	$V_{s1\_seg2\_max}$ = **6.2** kN	$V_{s1\_seg2\_min}$ = **-55.7** kN
Deflection segment 3	δ_{max} = **16.5** mm	δ_{min} = **0** mm
Maximum reaction at support A	R_{A_max} = **36.3** kN	R_{A_min} = **36.3** kN
Unfactored imposed load reaction at support A	$R_{A\_Imposed}$ = **24.2** kN	
Maximum reaction at support B	R_{B_max} = **89.7** kN	R_{B_min} = **89.7** kN
Unfactored imposed load reaction at support B	$R_{B\_Imposed}$ = **59.8** kN	

Section details

Section type **UKC 203x203x60 (Tata Steel Advance)** Steel grade **S355**

Classification of cross sections - Section 3.5

Tensile strain coefficient ε = **0.88** Section classification **Plastic**

Shear capacity - Section 4.2.3

Design shear force F_v = **55.7** kN Design shear resistance P_v = **419.7** kN

PASS - Design shear resistance exceeds design shear force

Moment capacity at span 1 segment 2 - Section 4.2.5

Design bending moment M = **67.3** kNm Moment capacity low shear M_c = **232.9** kNm

Buckling resistance moment - Section 4.3.6.4

Buckling resistance moment M_b = **182** kNm M_b / m_{LT} = **182** kNm

PASS - Buckling resistance moment exceeds design bending moment

Check vertical deflection - Section 2.5.2

Consider deflection due to dead and imposed loads

Limiting deflection δ_{lim} = **18.611** mm Maximum deflection δ = **16.457** mm

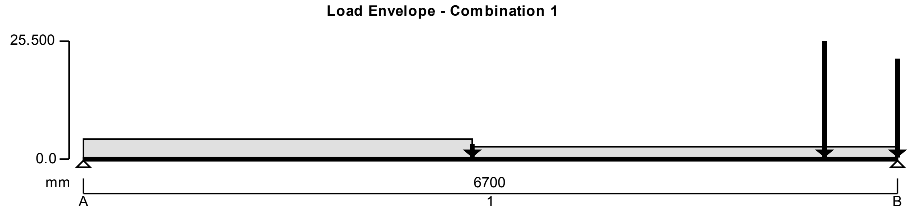
PASS - Maximum deflection does not exceed deflection limit

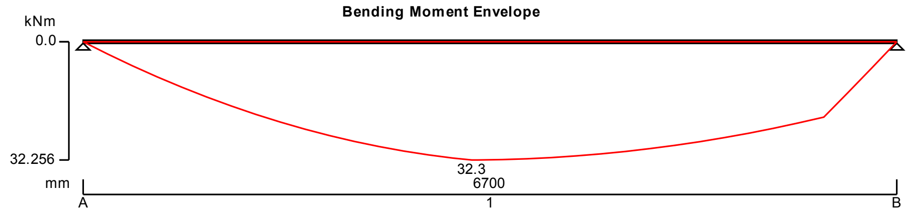
	Project				Job no.	
AXIOM STRUCTURES	85 Camden Mews				15005	
	Calcs for				Start page no./Revision	
	1B2 - DL + 0.1IL				1	
	Calcs by	Calcs date	Checked by	Checked date	Approved by	Approved date
	KK	04/12/2018	AP	04/12/2018		

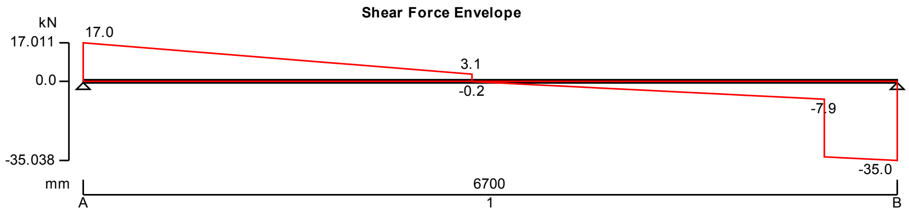
STEEL BEAM ANALYSIS & DESIGN (BS5950)

In accordance with BS5950-1:2000 incorporating Corrigendum No.1

TEDDS calculation version 3.0.05







Support conditions

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

Applied loading

Beam loads	Imposed self weight of beam × 1
	Imposed partial UDL 2.3 kN/m from 0 mm to 3200 mm
	Imposed partial UDL 1.2 kN/m from 3200 mm to 6700 mm
	Imposed point load 2.2 kN at 3200 mm
	Imposed point load 17 kN at 6100 mm
	Imposed point load 14.5 kN at 6700 mm

Load combinations

Load combination 1	Support A	Dead × 1.50
		Imposed × 1.50
	Span 1	Dead × 1.50
		Imposed × 1.50
	Support B	Dead × 1.50

Imposed × 1.50

Analysis results

Maximum moment	M_{max} = **32.3** kNm	M_{min} = **0** kNm
Maximum moment span 1 segment 1	$M_{s1\_seg1\_max}$ = **32.1** kNm	$M_{s1\_seg1\_min}$ = **0** kNm
Maximum moment span 1 segment 2	$M_{s1\_seg2\_max}$ = **32.3** kNm	$M_{s1\_seg2\_min}$ = **0** kNm
Maximum shear	V_{max} = **17** kN	V_{min} = **-35** kN
Maximum shear span 1 segment 1	$V_{s1\_seg1\_max}$ = **17** kN	$V_{s1\_seg1\_min}$ = **0** kN
Maximum shear span 1 segment 2	$V_{s1\_seg2\_max}$ = **3.4** kN	$V_{s1\_seg2\_min}$ = **-35** kN
Deflection segment 3	δ_{max} = **8.2** mm	δ_{min} = **0** mm
Maximum reaction at support A	R_{A_max} = **17** kN	R_{A_min} = **17** kN
Unfactored imposed load reaction at support A	$R_{A\_Imposed}$ = **11.3** kN	
Maximum reaction at support B	R_{B_max} = **56.8** kN	R_{B_min} = **56.8** kN
Unfactored imposed load reaction at support B	$R_{B\_Imposed}$ = **37.9** kN	

Section details

Section type **UKC 203x203x60 (Tata Steel Advance)** Steel grade **S355**

Classification of cross sections - Section 3.5

Tensile strain coefficient ε = **0.88** Section classification **Plastic**

Shear capacity - Section 4.2.3

Design shear force F_v = **35** kN Design shear resistance P_v = **419.7** kN

PASS - Design shear resistance exceeds design shear force

Moment capacity at span 1 segment 2 - Section 4.2.5

Design bending moment M = **32.3** kNm Moment capacity low shear M_c = **232.9** kNm

Buckling resistance moment - Section 4.3.6.4

Buckling resistance moment M_b = **182** kNm M_b / m_{LT} = **182** kNm

PASS - Buckling resistance moment exceeds design bending moment

Check vertical deflection - Section 2.5.2

Consider deflection due to dead and imposed loads

Limiting deflection δ_{lim} = **18.611** mm Maximum deflection δ = **8.159** mm

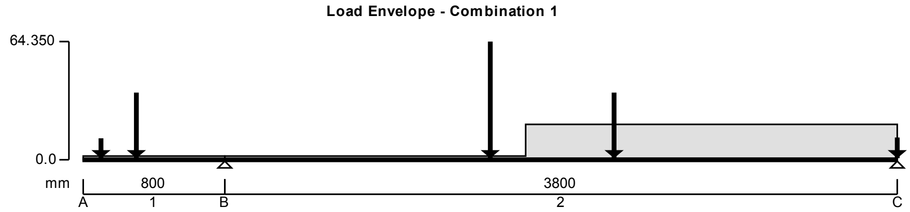
PASS - Maximum deflection does not exceed deflection limit

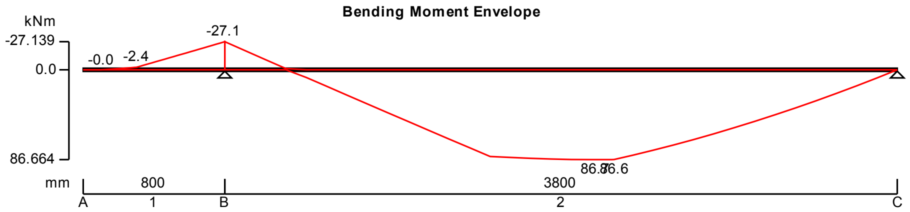
AXIOM STRUCTURES	Project				Job no.	
	85 Camden Mews				15005	
	Calcs for				Start page no./Revision	
	1B3				1	
	Calcs by	Calcs date	Checked by	Checked date	Approved by	Approved date
	KL	04/12/2018	AP	12/04/2017		

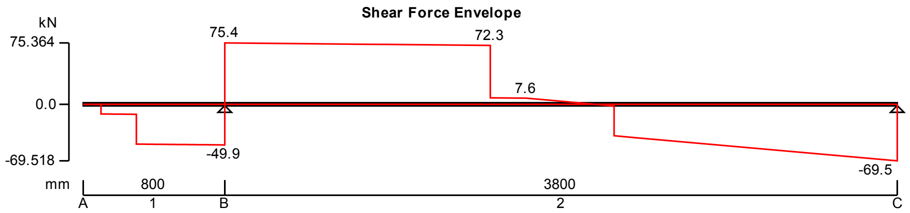
STEEL BEAM ANALYSIS & DESIGN (BS5950)

In accordance with BS5950-1:2000 incorporating Corrigendum No.1

TEDDS calculation version 3.0.05







Support conditions

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

Applied loading

Beam loads	Imposed self weight of beam × 1
	Imposed full UDL 0.9 kN/m
	Imposed point load 24.4 kN at 300 mm
	Imposed point load 24.4 kN at 3000 mm
	Imposed point load 35.1 kN at 2300 mm
	Imposed point load 7.8 kN at 100 mm
	Imposed point load 7.8 kN at 2300 mm
	Imposed point load 8.1 kN at 4600 mm
	Imposed partial UDL 11.5 kN/m from 2500 mm to 4600 mm

Load combinations

Load combination 1	Support A	Dead × 1.50
		Imposed × 1.50
	Span 1	Dead × 1.50
		Imposed × 1.50
	Support B	Dead × 1.50
		Imposed × 1.50
	Span 2	Dead × 1.50
		Imposed × 1.50
	Support C	Dead × 1.50
		Imposed × 1.50

Analysis results

Maximum moment	M_{max} = **86.7** kNm	M_{min} = **-27.1** kNm
Maximum moment span 1	M_{s1_max} = **0** kNm	M_{s1_min} = **-27.1** kNm
Maximum moment span 2	M_{s2_max} = **86.7** kNm	M_{s2_min} = **-27.1** kNm
Maximum shear	V_{max} = **75.4** kN	V_{min} = **-69.5** kN
Maximum shear span 1	V_{s1_max} = **0** kN	V_{s1_min} = **-49.9** kN
Maximum shear span 2	V_{s2_max} = **75.4** kN	V_{s2_min} = **-69.5** kN
Deflection	δ_{max} = **8.4** mm	δ_{min} = **4.3** mm
Deflection span 1	δ_{s1_max} = **0** mm	δ_{s1_min} = **4.3** mm
Deflection span 2	δ_{s2_max} = **8.4** mm	δ_{s2_min} = **0** mm
Maximum reaction at support A	R_{A_max} = **0** kN	R_{A_min} = **0** kN
Maximum reaction at support B	R_{B_max} = **125.3** kN	R_{B_min} = **125.3** kN
Unfactored imposed load reaction at support B	$R_{B\_Imposed}$ = **83.5** kN	
Maximum reaction at support C	R_{C_max} = **81.7** kN	R_{C_min} = **81.7** kN
Unfactored imposed load reaction at support C	$R_{C\_Imposed}$ = **54.4** kN	

Section details

Section type **UC 203x203x46 (BS4-1)** Steel grade **S355**

Classification of cross sections - Section 3.5

Tensile strain coefficient ε = **0.88** Section classification **Semi-compact**

Shear capacity - Section 4.2.3

Design shear force F_v = **75.4** kN Design shear resistance P_v = **311.6** kN

PASS - Design shear resistance exceeds design shear force

Moment capacity at span 2 - Section 4.2.5

Design bending moment M = **86.7** kNm Moment capacity low shear M_c = **174.1** kNm

PASS - Moment capacity exceeds design bending moment

Check vertical deflection - Section 2.5.2

Consider deflection due to dead and imposed loads

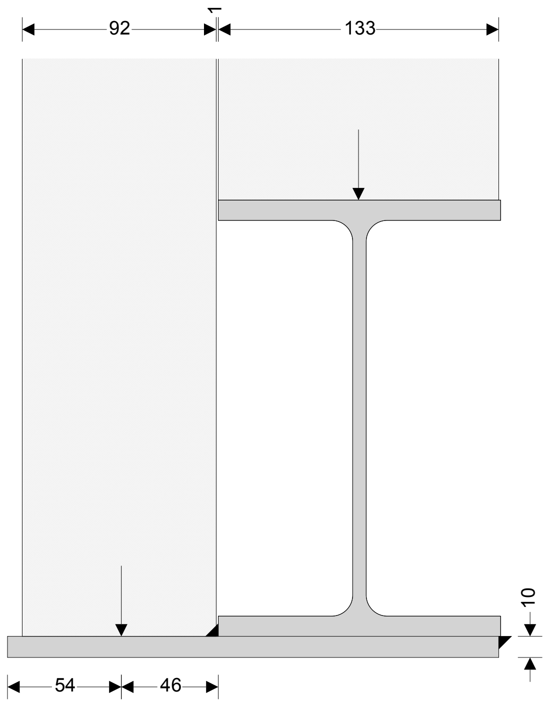
Limiting deflection δ_{lim} = **10.556** mm Maximum deflection δ = **8.387** mm

PASS - Maximum deflection does not exceed deflection limit

STEEL MASONRY SUPPORT

In accordance with BS5950-1:2000 incorporating Corrigendum No.1

Tedds calculation version 1.0.04



Steel member details

Torsion beam	UKB 203x133x30	Masonry support angle	plate
Steel grade of support angle	User	Design strength support angle	p_{ysb} = **355** N/mm^2
Modulus of elasticity	E = 205000 N/mm^2	Constant	ε = **0.880**
Length of plate beyond beam	l_h = **100** mm	Total length of plate	l_{plate} = **233** mm
Thickness of plate	t_{sb} = **10** mm	Width of main beam	B_{mb} = **134** mm
Area of plate	A_{sbu} = **2330.0** mm^2	Dist weld position to CoG	c_{yysb} = **-17** mm

Supported materials detail

Density mas. main beam	$\rho_{m,mb}$ = **20.0** kN/m^3	Width masonry main beam	b_{mmb} = **133** mm
Height masonry main beam	h_{mmb} = **2300** mm		
Ecc. of main beam material	e_{mb} = **0** mm		
Add dead force main beam	P_{Gaddmb} = **5.8** kN/m	Add live force main beam	P_{Qaddmb} = **0.0** kN/m
Density mas. support beam	$\rho_{m,sb}$ = **20.0** kN/m^3	Width masonry support beam	b_{msb} = **92** mm
Height masonry support beam	h_{msb} = **2300** mm		
Add dead force support beam	P_{Gaddsb} = **0.0** kN/m	Add live force support beam	P_{Qaddsb} = **0.0** kN/m

Geometry

Cavity width	c = **1** mm	Supported width of masonry	d_m = **99** mm

Biaxial stress effects in the plate (SCI-P-110)

Max overall bending moment	M_x = **21.2** kNm	Dist to NA combined section	$y_{e,all}$ = **41** mm
Second moment of area	$I_{xx,all}$ = **4596** cm^4	Elastic section modulus	$Z_{xx,all}$ = **635.39** cm^3
Section modulus of plate	$Z_{xx,plate}$ = **16.67** cm^3/m	Eccentricity on support beam	e_1 = **46** mm
Force on support plate	P_1 = **5.9** kN/m	Bending at heel	$M_{x,plate}$ = **0.3** kNm/m
Moment capacity of plate	M_c = **7.1** kNm/m		

PASS - Design strength exceeds stress at heel

Long stress overall bending σ_1 = **33.4** N/mm^2 Von Mises curve constant c_{fp} = **707.6** N/mm^2

Trans bending stress ratio limit α_{ts} = **0.994** Trans bending stress ratio α_{ls} = **0.038**

PASS - Transverse bending stress ratio less than allowable limit

Deflection at toe

Unfact force on plate P_{1SLS} = **4.2** kN/m Distance from weld to load a_m = **46** mm

Load resultant to edge of plate b_m = **54** mm Weld to load pos as ratio a_l = **0.460**

Effect second mnt of inertia I_{eff_def} = **83333** mm^4/m Deflection at toe δ = **0.02** mm

Deflection limit δ_{lim} = **2.00** mm

PASS - Deflection is within specified criteria

Weld details - assume a full length weld and that the plate acts as a propped cantilever with the prop at the weld position and the fixed end at the centre of the torsion beam

Leg length of weld s_{weld} = **6** mm Throat size of weld a_{weld} = **4.2** mm

Shear force at weld position R_A = **12.0** kN/m Max possible force in plate R_p = **830.3** kN

Long shear beam/plate R_l = **615.1** kN/m Horizontal shear beam/plate R_h = **34.1** kN/m

Resultant weld force R_{weld} = **0.616** kN/mm Strength of weld (Table 37) p_{weld} = **220.0** N/mm^2

Capacity of full length weld $p_{c,weld}$ = **0.933** kN/mm

$1/\sqrt{(2)} \times s_{weld}$

Torsional loading ULS

Loading support beam w_{1ULS} = **5.92** kN/m Loading of main beam w_{2ULS} = **16.69** kN/m

Self weight of support beam w_{3ULS} = **0.26** kN/m

Torsional loading SLS

Loading support beam w_{1SLS} = **4.23** kN/m Loading of main beam w_{2SLS} = **11.92** kN/m

Self weight of support beam w_{3SLS} = **0.18** kN/m

Eccentricities

Distance of shear centre e_{0mb} = **0** mm Ecc of support beam masonry e_{1mb} = **114** mm

Ecc of main beam masonry e_{2mb} = **0** mm Ecc of support beam e_{3mb} = **50** mm

Torsional effects

Applied torque T_{qULS} = **0.70** kNm/m Torsional moment (ULS) T_q = **1.88** kNm

Applied torque (SLS) T_{qSLS} = **0.50** kNm/m Torsional moment (SLS) T_{qu} = **1.34** kNm

STEEL BEAM TORSION DESIGN

In accordance with BS5950-1:2000 incorporating Corrigendum No.1

Tedds calculation version 2.0.02

Section details

Section type UKB 203x133x30 Steel grade S355

Design stength $p_{yw} = p_y$ = **355** N/mm^2 Constant ε = **0.880**

Geometry - Beam unrestrained against lateral-torsional buckling between supports.

Effective span L = **2700** mm

Length of segment LTB L_{LT} = **2700** mm Effective length for LTB L_{E_LT} = **3114** mm

Loading - Torsional loading comprises only full-length uniformly distributed load(s)

Internal forces & moments on member under factored loading for uls design

Applied shear force F_{vy} = **31.4** kN Maximum bending moment $M_{LT} = M_x$ = **21.22** kNm

Applied torque T_q = **1.88** kNm Minor axis bending moment M_y = 0 kNm

Compression force F_c = 0 kN

Equivalent uniform moment factors

EUM factor (Cl.4.3.6.6 & T18) m_{LT} = **1.000**

Torsional deflection parameters

Beam is torsion fixed and warping free at each end. (as defined in SCI-P-057 section 2.1.6) - Appendix B case 4

Dist for first deriv of twist	z_1 = 0 mm	Dist for second deriv of twist	z_2 = L / 2 = **1350** mm
First deriv of angle of twist	ϕ'_1 = **4.21×10^{-2}** rads/m	Third deriv of angle of twist	ϕ'''_1 = **-7.78×10^{-2}** rads/m^3
Angle of twist	ϕ_2 = **0.035** rads	Second deriv of angle of twist	ϕ''_2 = **-4.55×10^{-2}** rads/m^2

Design parameters

Total angle of twist	ϕ = **0.035** rads	First derivative of ϕ	ϕ' = **4.21×10^{-2}** rads/m
Second derivative of ϕ	ϕ'' = **4.55×10^{-2}** rads/m^2	Third derivative of ϕ	ϕ''' = **7.78×10^{-2}** rads/m^3

Section classification

	b / T = **7.0**		d / t = **26.9**
	r_{1s} = **0.000**		r_{2s} = **0.000**

Section classification is plastic

Shear capacity (parallel to y-axis)

Design shear force	F_{vy} = **31.4** kN	Design shear resist (cl. 4.2.3)	P_{vy} = **281.9** kN

Pass - Shear

Moment capacity (x-axis)

Design bending moment	M_x = **21.2** kNm	Mnt cap low shear (cl. 4.2.5.1)	M_{cx} = **111.6** kNm

Pass - Moment capacity exceeds design bending moment

Lateral torsional buckling

Effective length for LTB	L_{E_LT} = **3114** mm		
Slenderness ratio	λ = **98**	Buckling parameter	u = **0.881**
Flange ratio	η = 0.5	Torsional index	x = **21.5**
Slenderness factor	v = **0.84**	Ratio - cl 4.3.6.9	β_w = **1.000**
Equiv slenderness – cl 4.3.6.7	λ_{LT} = **72**	Limit slendernes – Ann B2.2	λ_{L0} = **30**
Euler stress	p_E = **386** N/mm^2	Perry factor	η_{LT} = **0.295**
	ϕ_{LT} = **427731813.312**	Bending strength	p_b = **214** N/mm^2
Buckling resistance moment	M_b = **67.2** kNm	Max mnt gov buckling resist	M_{LT} = **21.2** kNm
Equiv uniform mnt factor LTB	m_{LT} = **1.00**		M_b / m_{LT} = **67.2** kNm

Pass - lat. tors. buckling

Buckling under combined bending & torsion -SCI-P-057 section 2.3

For simplicity, a conservative check is applied using the maximum stresses due to each of the separate load effects, even though these do not necessarily all occur at the same section along the member.

Span factor	L / a = **2.78**	Angle of twist	ϕ = **0.035** rads
Second derivative of ϕ	ϕ'' = **45.5×10^{-3}** rads/m^2	Induced minor axis moment	M_{yt} = **0.75** kNm
Normal stress flange M_{yt}	σ_{byt} = **13** N/mm^2	Normal stress flange warping	σ_w = **62** N/mm^2
Interaction index	i_b = **0.56**		

Pass - Combined bending and torsion check satisfied

Local capacity under combined bending & torsion

For simplicity, a conservative check is applied using the maximum stresses due to each of the separate load effects, even though these do not necessarily all occur at the same section along the member.

Max. direct stress due to M_x	σ_{bx} = M_x / Z_x = **76** N/mm^2		
Combined stress - eqn 2.22	$\sigma_{bx} + \sigma_{byt} + \sigma_w$ = **150** N/mm^2	Design strength	p_y = **355** N/mm^2

Pass - Local capacity

Combined shear stresses - SCI-P-057 section 2.3

For simplicity, a conservative check is applied using the maximum shear stresses due to each of the separate load effects, even though these do not necessarily all occur at the same section along the member.

Shear stress bending web τ_{bw} = **27** N/mm^2

Shear stress bending flange τ_{bf} = **7** N/mm^2

Shear stresses torsion web τ_{tw} = **21** N/mm^2

Shear stresses torsion flange τ_{tf} = **32** N/mm^2

Shear stresses warping flange τ_{wf} = **4** N/mm^2

Shear stress tors & warp web τ_{vtw} = **25** N/mm^2

Shear str tors & warp flange τ_{vtf} = **41** N/mm^2

Combined shear stresses due to bending, torsion & warping:

Comb shear stresses in web τ_w = **51** N/mm^2

Comb shear stresses in flange τ_f = **48** N/mm^2

Shear strength p_v = **213** N/mm^2

Pass - Combined shear stresses

Twist check

Total applied torque T_{qu} = **1.34** kNm

Max twist under sls loading ϕ_{sls} = **1.44** deg

Twist limit ϕ_{lim} = **2.00** deg

Pass - Twist

Deflection

Maximum y-axis deflection δ_{y_max} = **1.9** mm

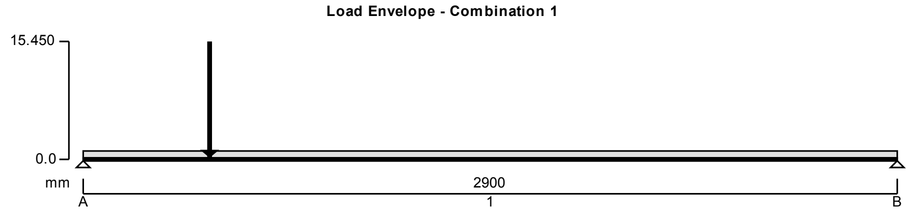
Deflection limit - cl. 2.5.2 δ_{lim} = **7.5** mm

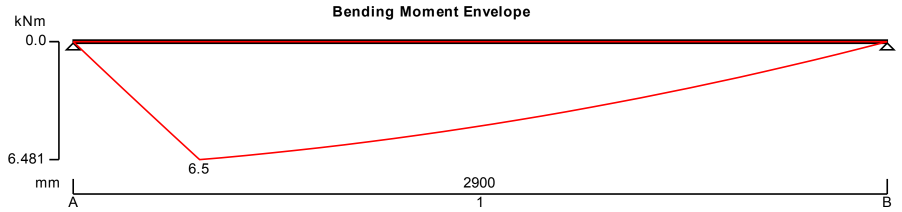
Pass - Deflection within specified limit

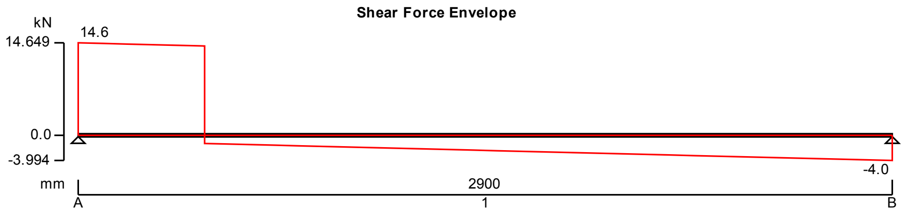
STEEL BEAM ANALYSIS & DESIGN (BS5950)

In accordance with BS5950-1:2000 incorporating Corrigendum No.1

TEDDS calculation version 3.0.05







Support conditions

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

Applied loading

Beam loads	Imposed self weight of beam × 1
	Imposed full UDL 0.5 kN/m
	Imposed point load 10.3 kN at 450 mm

Load combinations

Load combination 1	Support A	Dead × 1.50
		Imposed × 1.50
	Span 1	Dead × 1.50
		Imposed × 1.50
	Support B	Dead × 1.50
		Imposed × 1.50

Analysis results

Maximum moment	M_{max} = **6.5** kNm	M_{min} = **0** kNm
Maximum shear	V_{max} = **14.6** kN	V_{min} = **-4** kN
Deflection	δ_{max} = **1.3** mm	δ_{min} = **0** mm
Maximum reaction at support A	R_{A_max} = **14.6** kN	R_{A_min} = **14.6** kN
Unfactored imposed load reaction at support A	$R_{A\_Imposed}$ = **9.8** kN	
Maximum reaction at support B	R_{B_max} = **4** kN	R_{B_min} = **4** kN
Unfactored imposed load reaction at support B	$R_{B\_Imposed}$ = **2.7** kN	

Section details

Section type **PFC 150x90x24 (BS4-1)** Steel grade **S355**

Classification of cross sections - Section 3.5

Tensile strain coefficient ε = **0.88** Section classification **Plastic**

Shear capacity - Section 4.2.3

Design shear force F_v = **14.6** kN Design shear resistance P_v = **207.7** kN

PASS - Design shear resistance exceeds design shear force

Moment capacity - Section 4.2.5

Design bending moment M = **6.5** kNm Moment capacity low shear M_c = **63.4** kNm

Buckling resistance moment - Section 4.3.6.4

Buckling resistance moment M_b = **40.3** kNm M_b / m_{LT} = **40.3** kNm

PASS - Buckling resistance moment exceeds design bending moment

Check vertical deflection - Section 2.5.2

Consider deflection due to dead and imposed loads

Limiting deflection δ_{lim} = **8.056** mm Maximum deflection δ = **1.291** mm

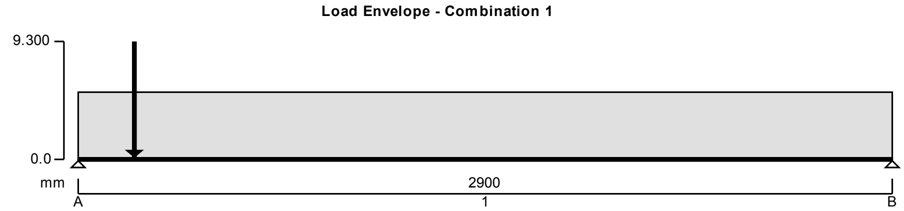
PASS - Maximum deflection does not exceed deflection limit

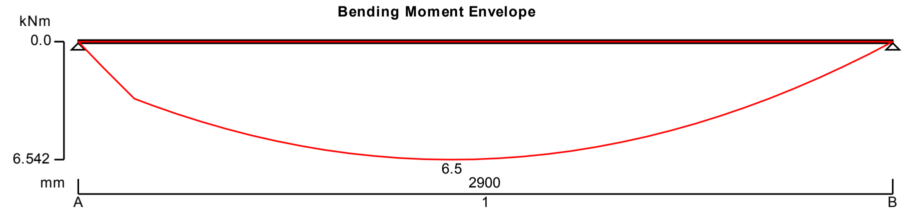
AXIOM STRUCTURES	Project 85 Camden Mews				Job no. 15005	
	Calcs for 1B6-2				Start page no./Revision 1	
	Calcs by KK	Calcs date 04/12/2018	Checked by AP	Checked date 04/12/2018	Approved by	Approved date

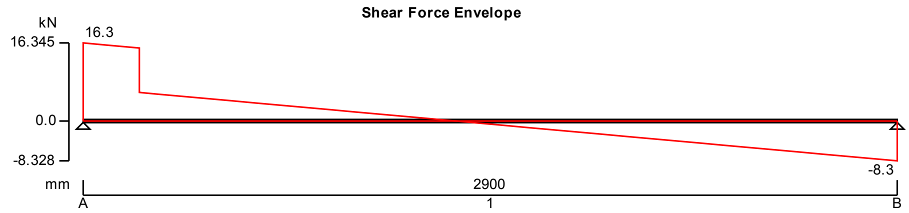
STEEL BEAM ANALYSIS & DESIGN (BS5950)

In accordance with BS5950-1:2000 incorporating Corrigendum No.1

TEDDS calculation version 3.0.05







Support conditions

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

Applied loading

Beam loads	Imposed self weight of beam × 1 Imposed full UDL 3.3 kN/m Imposed point load 6.2 kN at 200 mm

Load combinations

Load combination 1	Support A	Dead × 1.50 Imposed × 1.50
	Span 1	Dead × 1.50 Imposed × 1.50
	Support B	Dead × 1.50 Imposed × 1.50

Analysis results

Maximum moment	M_{max} = **6.5** kNm	M_{min} = **0** kNm
Maximum shear	V_{max} = **16.3** kN	V_{min} = **-8.3** kN
Deflection	δ_{max} = **1.6** mm	δ_{min} = **0** mm
Maximum reaction at support A	R_{A_max} = **16.3** kN	R_{A_min} = **16.3** kN
Unfactored imposed load reaction at support A	$R_{A\_Imposed}$ = **10.9** kN	
Maximum reaction at support B	R_{B_max} = **8.3** kN	R_{B_min} = **8.3** kN
Unfactored imposed load reaction at support B	$R_{B\_Imposed}$ = **5.6** kN	

Section details

Section type **PFC 150x90x24 (BS4-1)** Steel grade **S355**

Classification of cross sections - Section 3.5

Tensile strain coefficient ε = **0.88** Section classification **Plastic**

Shear capacity - Section 4.2.3

Design shear force F_v = **16.3** kN Design shear resistance P_v = **207.7** kN

PASS - Design shear resistance exceeds design shear force

Moment capacity - Section 4.2.5

Design bending moment M = **6.5** kNm Moment capacity low shear M_c = **63.4** kNm

Buckling resistance moment - Section 4.3.6.4

Buckling resistance moment M_b = **40.3** kNm M_b / m_{LT} = **40.3** kNm

PASS - Buckling resistance moment exceeds design bending moment

Check vertical deflection - Section 2.5.2

Consider deflection due to dead and imposed loads

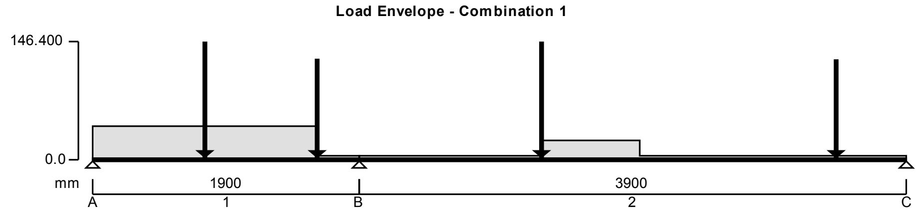
Limiting deflection δ_{lim} = **8.056** mm Maximum deflection δ = **1.639** mm

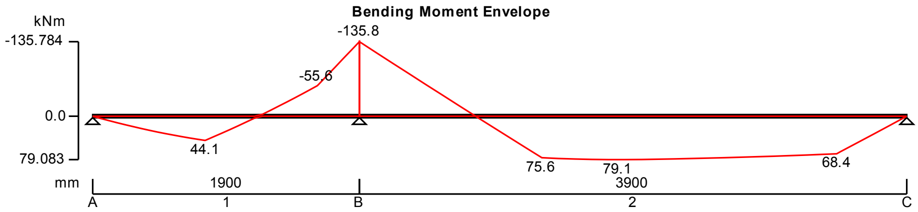
PASS - Maximum deflection does not exceed deflection limit

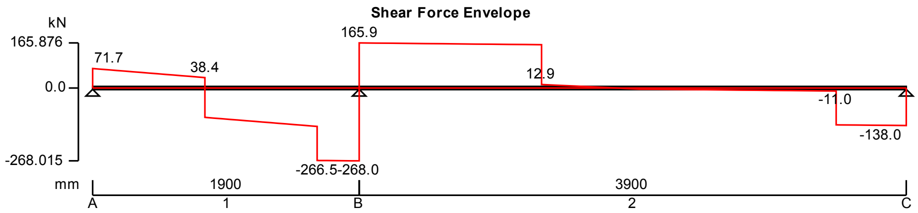
STEEL BEAM ANALYSIS & DESIGN (BS5950)

In accordance with BS5950-1:2000 incorporating Corrigendum No.1

TEDDS calculation version 3.0.05







Support conditions

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

Applied loading

Beam loads	Imposed self weight of beam × 1
	Imposed full UDL 2.9 kN/m
	Imposed partial UDL 24.4 kN/m from 0 mm to 1600 mm
	Imposed point load 97.6 kN at 800 mm
	Imposed point load 97.6 kN at 3200 mm
	Imposed point load 83 kN at 5300 mm
	Imposed point load 83.5 kN at 1600 mm
	Imposed partial UDL 12.7 kN/m from 3200 mm to 3900 mm

Load combinations

Load combination 1	Support A	Dead × 1.50

		Imposed × 1.50
	Span 1	Dead × 1.50
		Imposed × 1.50
	Support B	Dead × 1.50
		Imposed × 1.50
	Span 2	Dead × 1.50
		Imposed × 1.50
	Support C	Dead × 1.50
		Imposed × 1.50

Analysis results

Maximum moment	M_{max} = **79.1** kNm	M_{min} = **-135.8** kNm
Maximum moment span 1	M_{s1_max} = **44.1** kNm	M_{s1_min} = **-135.8** kNm
Maximum moment span 2	M_{s2_max} = **79.1** kNm	M_{s2_min} = **-135.8** kNm
Maximum shear	V_{max} = **165.9** kN	V_{min} = **-268** kN
Maximum shear span 1	V_{s1_max} = **71.7** kN	V_{s1_min} = **-268** kN
Maximum shear span 2	V_{s2_max} = **165.9** kN	V_{s2_min} = **-138** kN
Deflection	δ_{max} = **8.2** mm	δ_{min} = **0.2** mm
Deflection span 1	δ_{s1_max} = **0.4** mm	δ_{s1_min} = **0.2** mm
Deflection span 2	δ_{s2_max} = **8.2** mm	δ_{s2_min} = **0** mm
Maximum reaction at support A	R_{A_max} = **71.7** kN	R_{A_min} = **71.7** kN
Unfactored imposed load reaction at support A	$R_{A\_Imposed}$ = **47.8** kN	
Maximum reaction at support B	R_{B_max} = **433.9** kN	R_{B_min} = **433.9** kN
Unfactored imposed load reaction at support B	$R_{B\_Imposed}$ = **289.3** kN	
Maximum reaction at support C	R_{C_max} = **138** kN	R_{C_min} = **138** kN
Unfactored imposed load reaction at support C	$R_{C\_Imposed}$ = **92** kN	

Section details

Section type **UC 203x203x46 (BS4-1)** Steel grade **S355**

Classification of cross sections - Section 3.5

Tensile strain coefficient ε = **0.88** Section classification **Semi-compact**

Shear capacity - Section 4.2.3

Design shear force F_v = **268** kN Design shear resistance P_v = **311.6** kN

PASS - Design shear resistance exceeds design shear force

Moment capacity at span 1 - Section 4.2.5

Design bending moment M = **135.8** kNm Moment capacity high shear M_c = **160.4** kNm

Buckling resistance moment - Section 4.3.6.4

Buckling resistance moment M_b = **167.2** kNm M_b / m_{LT} = **167.2** kNm

PASS - Moment capacity exceeds design bending moment

Check vertical deflection - Section 2.5.2

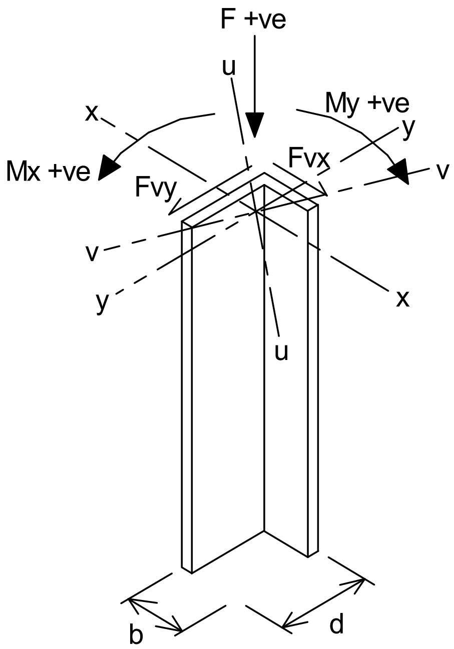
Consider deflection due to dead and imposed loads

Limiting deflection δ_{lim} = **10.833** mm Maximum deflection δ = **8.181** mm

PASS - Maximum deflection does not exceed deflection limit

STEEL ANGLE DESIGN (BS5950-1:2000)

TEDDS calculation version 1.0.04



Element definition

Element being designed	**1**	Section	**RSA 120x120x10**
Steel grade	**S275**	Design strength (Table 9)	p_y = **275** N/mm^2

Design forces and moments

Shear force parallel to y axis	F_{vy} = **0.00** kN	Shear force parallel to x axis	F_{vx} = **1.30** kN
Axial force	F = **0.0 kN**		
Max moment about x axis	M_x = **1.00** kNm	Max moment about y axis	M_y = **0.00** kNm

Section classification (Table 11)

The section is Class 3 (semi-compact) for bending

Design for shear

For shear force parallel to x axis (cl. 4.2.3)

Shear capacity	P_{vx} = **178.20** kN	Shear cap. for 'low shear'	P_{vx_low} = **106.92** kN

PASS - The angle is in low shear parallel to x axis

Design for bending

The angle is not restrained against lateral torsional buckling

Moment capacities

Min mt cap about x-x axis	M_{cx_min} = **10.02** kNm	Max mt cap about x-x axis	M_{cx_max} = **26.15** kNm

PASS - The moment capacity about the x-x axis exceeds the applied moment

Design moments about principal axes

Resultant u axismoment	M_u = **0.71** kNm	Resultant v axis moment	M_v = **0.71** kNm
Max u axis mt in segm't length	M_{LT} = **0.71** kNm		

Equivalent uniform moment factor

The angle is subjected to destabilising loads therefore m_{LT} = 1.0 (Cl. 4.3.6.6)

EUM factor m_{LT} — m_{LT} = **1.000**

Min u axis section modulus — Z_u = **59.2** cm^3 — Min v axis section modulus — Z_v = **27.7** cm^3

Buckling resistance moment

Buckling resistance moment — $M_b = p_b \times Z_u$ = **14.16** kNm — Eff buckling resistance mt — $M_{beff} = M_b/m_{LT}$ = **14.16** kNm

PASS - Effective buckling resistance moment exceeds applied major axis moment

Minor axis bending resistance

Bending resistance — M_{cv} = **7.63** kNm

PASS - Minor axis bending resistance moment exceeds applied minor axis moment

Equivalent uniform moment factors

EUM factor m_u — m_u = **1.000** — EUM factor m_v — m_v = **1.000**

Member buckling resistance (cl 4.8.3.3)

Equation 1 — UF_1 = **0.136** — Equation 2 — UF_2 = **0.143**

PASS - Member buckling resistance is adequate

Project				Job no.	
85 Camden Mews				15005	
Calcs for				Start page no./Revision	
215thk Blockwork Wall				1	
Calcs by	Calcs date	Checked by	Checked date	Approved by	Approved date
KK	05/12/2018				

MASONRY WALL PANEL DESIGN

In accordance with BS5628-1:2005

Tedds calculation version 1.2.10

Masonry panel details

Cavity wall - Unreinforced masonry wall without openings

Panel length L = **2500** mm

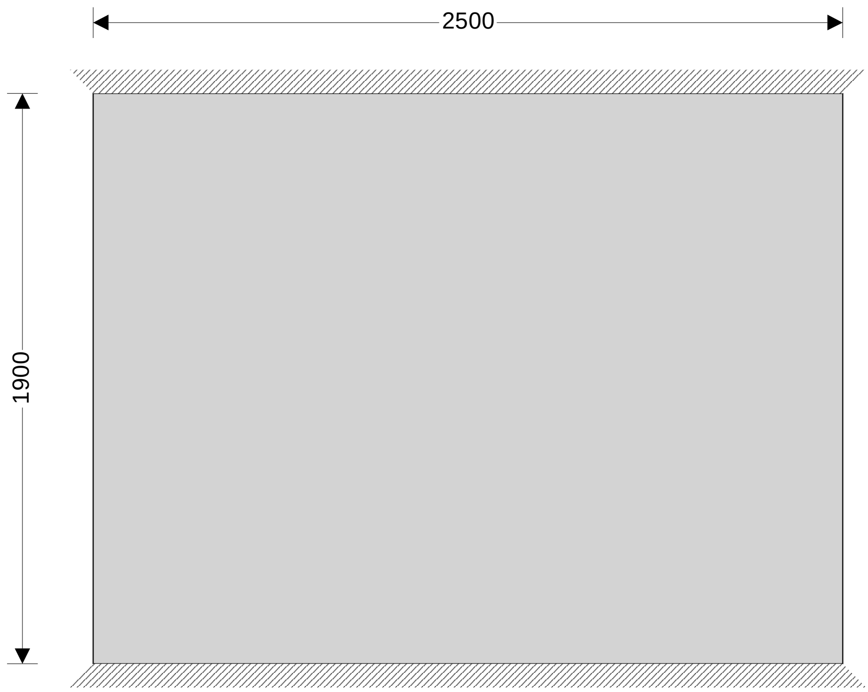
Panel height h = **1900** mm

Panel support conditions

Top and bottom supported

Effective panel length L_{ef} = 2.5 × L = **6250** mm

Effective panel height h_{ef} = 1.0 × h = **1900** mm



Single-leaf wall construction details

Wall thickness t = **215** mm

Effective wall thickness t_{ef} = t = **215** mm

Masonry details

Masonry type	**Aggregate concrete blocks with no voids**
Compressive strength of unit	p_{unit} = **7.3** N/mm^2
Mortar strength Class/Designation	**M4 / (iii)**
Height of masonry units	h_b = **440** mm
Density of masonry	γ = **18.0** kN/m^3
Least horizontal dimension of masonry units	t_{min} = **215** mm
Ratio of height to least horizontal dimension	h_b / t_{min} = **2.05**

From BS5628-1 Table 2d - Characteristic compressive strength of masonry

Characteristic compressive strength	f_k = **6.40** N/mm^2

From BS5628-1 Table 3 - Characteristic flexural strength of masonry

Plane of failure parallel to bed joints	f_{kx_para} = **0.17** N/mm^2
Plane of failure perpendicular to bed joints	f_{kx_perp} = **0.41** N/mm^2

Lateral loading details

Characteristic wind load on panel	W_k = **0.700** kN/m^2

Partial safety factors for material strength

Category of manufacturing control	**Category II**
Category of construction control	**Normal**
Partial safety factor for masonry in compression	γ_{mc} = **3.50**
Partial safety factor for masonry in flexure	γ_{mf} = **3.00**
Partial safety factor for masonry in shear	γ_{mv} = **2.50**

Horizontal loading (cl 32)

Limiting dimensions (cl 32.3)

Limiting wall height	$h_{max} = 40 \times t_{ef}$ = **8600** mm

PASS - Limiting wall height is not exceeded

Partial safety factors for design loads

Partial safety factor for design wind load	γ_{fW} = **1.40**
Partial safety factor for design dead load	γ_{fG} = **0.90**

Design moments of resistance in panels (cl 32.4.2)

Self weight of wall	$S_{wt} = 0.375 \times h \times t \times \gamma$ = **2.76** kN/m
Design vertical compressive stress	$g_d = \gamma_{fG} \times (G_k + S_{wt}) / t$ = **0.01** N/mm^2
Enhanced flexural strength of masonry	$f_{ka\_para} = f_{kx\_para} + \gamma_{mf} \times g_d$ = **0.21** N/mm^2
Section modulus of wall	$Z = t^2 / 6$ = **7704167** mm^3/m
Elastic design moment of resistance	$M_d = f_{ka\_para} \times Z / \gamma_{mf}$ = **0.534** kNm/m

Design moment in panels (cl 32.4.2)

Using elastic analysis to determine bending moment coefficients for a vertically spanning panel

Bending moment coefficient	α = **0.125**
Design moment in wall	$M = \alpha \times W_k \times \gamma_{fW} \times h^2$ = **0.442** kNm/m

PASS - Resistance moment exceeds design moment

MASONRY WALL PANEL DESIGN

In accordance with BS5628-1:2005

Tedds calculation version 1.2.10

Masonry panel details

Cavity wall - Unreinforced masonry wall without openings

Panel length L = **1200** mm

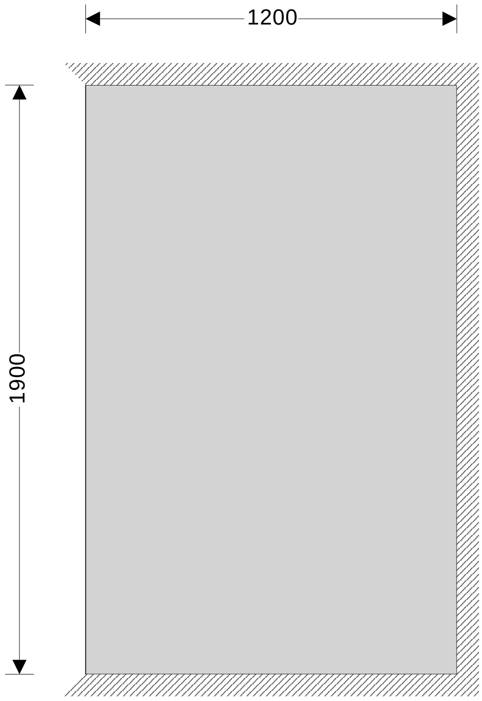
Panel height h = **1900** mm

Panel support conditions

Top, bottom and right supported

Effective panel length $L_{ef} = 2.5 \times L =$ **3000** mm

Effective panel height $h_{ef} = 1.0 \times h =$ **1900** mm



Single-leaf wall construction details

Wall thickness t = **215** mm

Effective wall thickness $t_{ef} = t =$ **215** mm

Masonry details

Masonry type	**Aggregate concrete blocks with no voids**
Compressive strength of unit	p_{unit} = **7.3** N/mm^2
Mortar strength Class/Designation	**M4 / (iii)**
Height of masonry units	h_b = **440** mm
Density of masonry	γ = **18.0** kN/m^3
Least horizontal dimension of masonry units	t_{min} = **215** mm
Ratio of height to least horizontal dimension	h_b / t_{min} = **2.05**

From BS5628-1 Table 2d - Characteristic compressive strength of masonry

Characteristic compressive strength — f_k = **6.40** N/mm^2

From BS5628-1 Table 3 - Characteristic flexural strength of masonry

Plane of failure parallel to bed joints — f_{kx_para} = **0.17** N/mm^2

Plane of failure perpendicular to bed joints — f_{kx_perp} = **0.41** N/mm^2

Lateral loading details

Characteristic wind load on panel — W_k = **0.700** kN/m^2

Partial safety factors for material strength

Category of manufacturing control	**Category II**
Category of construction control	**Normal**
Partial safety factor for masonry in compression	γ_{mc} = **3.50**
Partial safety factor for masonry in flexure	γ_{mf} = **3.00**
Partial safety factor for masonry in shear	γ_{mv} = **2.50**

Horizontal loading (cl 32)

Limiting dimensions (cl 32.3)

Area of panel — $A_p = h \times L$ = **2.3** m^2

Limiting area of panel — $A_{max} = 1350 \times t_{ef}^2$ = **62.4** m^2

PASS - Area of panel does not exceed limiting area of panel

Limiting panel dimension — $L_{max} = 50 \times t_{ef}$ = **10750** mm

PASS - Limiting panel dimension is not exceeded

Partial safety factors for design loads

Partial safety factor for design wind load — γ_{fW} = **1.40**

Partial safety factor for design dead load — γ_{fG} = **0.90**

Design moments of resistance in panels (cl 32.4.2)

Design vertical compressive stress — $g_d = \gamma_{fG} \times G_k / t$ = **0.00** N/mm^2

Enhanced flexural strength of masonry — $f_{ka\_para} = f_{kx\_para} + \gamma_{mf} \times g_d$ = **0.17** N/mm^2

Section modulus of wall — $Z = t^2 / 6$ = **7704167** mm^3/m

Elastic design moment of resistance — $M_d = f_{ka\_para} \times Z / \gamma_{mf}$ = **0.445** kNm/m

Design moment in panels (cl 32.4.2)

Orthogonal strength ratio — $\mu = f_{ka\_para} / f_{kx\_perp}$ = **0.42**

Using yield line analysis to calculate bending moment coefficient

Bending moment coefficient — α = **0.232**

Design moment in wall — $M = \mu \times \alpha \times W_k \times \gamma_{fW} \times L^2$ = **0.139** kNm/m

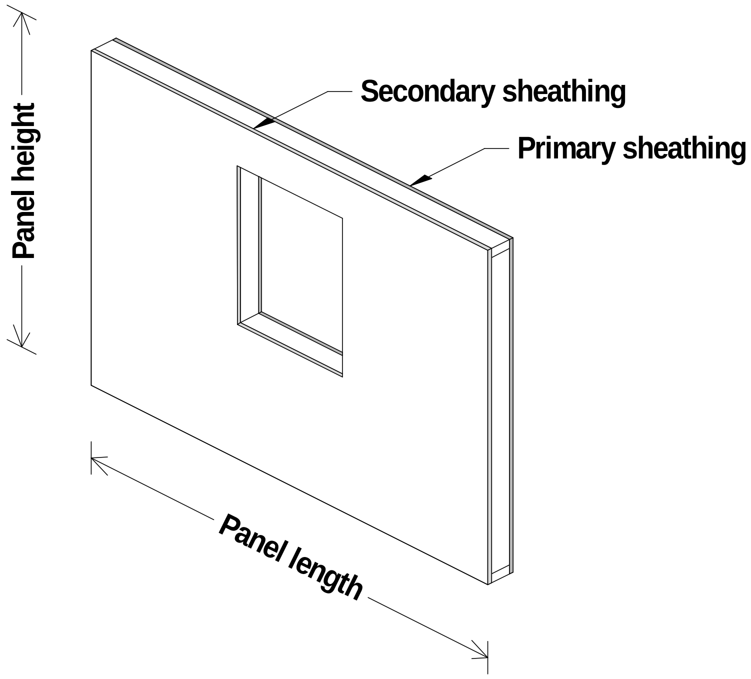
PASS - Resistance moment exceeds design moment

TIMBER PANEL RACKING RESISTANCE – BS5268:SECTION 6.1:1996

TEDDS calculation version 1.0.05

Dwellings not exceeding seven storeys

Timber panel



Wall panel details

Length of panel — L = **2.600** m

Height of panel — H_{wp} = **2.700** m

Total area of wall panel — $A_t = L \times H_{wp}$ = **7.020** m^2

Aggregate area of framed panel openings — A_a = **1.200** m^2

Timber members — **38 mm x 72 mm or larger**

Uniformly distributed load on timber frame wall — F_{udl} = **0.000** kN/m

For calculation equivalent uniformly distributed load — $F = \min(F_{udl}, 10.5\ \text{kN/m})$ = **0.000** kN/m

Primary sheathing details

Primary board type — **OSB**

Standard board thickness — t_p = **9.00** mm

Proposed board thickness — T_p = **9.00** mm

Ratio of proposed to standard board thickness — $B_p = \min(\max(T_p / t_p, 0.75), 1.25)$ = **1.00**

Nail diameter — D_p = **3.50** mm

Standard perimeter nail spacing — s_p = **150** mm

Proposed perimeter nail spacing — S_p = **75** mm

From Table 2 – Basic racking resistance for a range of materials and combinations of materials

Basic racking resistance — R_{bp} = **1.680** kN/m

Modification factors for variation in fixing and thickness of primary sheathing

Variation in nail diameter — $K_{101p} = D_p / 3\ \text{mm}$ = **1.167**

Variation in nail spacing — $K_{102p} = 1 / (0.6 \times (S_p / s_p) + 0.4)$ = **1.429**

Variation in board thickness $K_{103p} = 2.8 \times B_p - B_p^2 - 0.8 = \mathbf{1.000}$

Material modification factors $K_{mp} = K_{101p} \times K_{102p} \times K_{103p} = \mathbf{1.667}$

Secondary sheathing details

Secondary board type **Plywood**

Standard board thickness $t_s = \mathbf{9.50}$ mm

Proposed board thickness $T_s = \mathbf{9.00}$ mm

Ratio of proposed to standard board thickness $B_s = \min(\max(T_s / t_s, 0.75), 1.25) = \mathbf{0.95}$

Nail diameter $D_s = \mathbf{3.50}$ mm

Standard perimeter nail spacing $s_s = \mathbf{150}$ mm

Proposed perimeter nail spacing $S_s = \mathbf{150}$ mm

From Table 2 – Basic racking resistance for a range of materials and combinations of materials

Basic racking resistance $R_{bs} = \mathbf{0.840}$ kN/m

Modification factors for variation in fixing and thickness of secondary sheathing

Variation in nail diameter $K_{101s} = D_s / 3 \text{ mm} = \mathbf{1.167}$

Variation in nail spacing $K_{102s} = 1 / (0.6 \times (S_s / s_s) + 0.4) = \mathbf{1.000}$

Variation in board thickness $K_{103s} = 2.8 \times B_s - B_s^2 - 0.8 = \mathbf{0.955}$

Material modification factors $K_{ms} = K_{101s} \times K_{102s} \times K_{103s} = \mathbf{1.114}$

Modification factors for wall height, length, openings, vertical load and interaction

Height of wall panels $K_{104} = 2.4 \text{ m} / H_{wp} = \mathbf{0.889}$

Length of walls $K_{105} = (L / 2.4 \text{ m})^{0.4} = \mathbf{1.033}$

Fully framed openings in walls $K_{106} = (1 - 1.3 \times A_a / A_t)^2 = \mathbf{0.605}$

Vertical load on timber frame wall $K_{107} = 1 + [(0.09 \times (F / 1 \text{ kN/m}) - 0.0015 \times (F / 1 \text{ kN/m})^2) \times (2.4 \text{ m} / L)^{0.4}]$

$K_{107} = \mathbf{1.000}$

Interaction $K_{108} = \mathbf{1.100}$

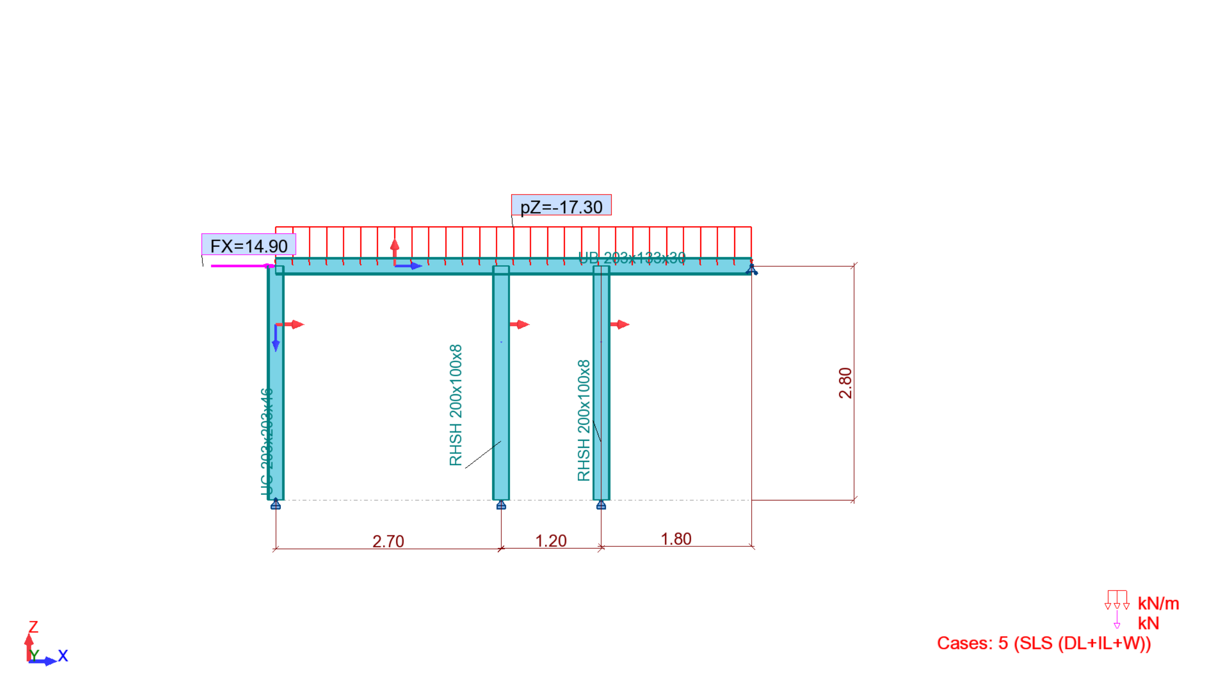
Wall modification factors $K_w = K_{104} \times K_{105} \times K_{106} \times K_{107} \times K_{108} = \mathbf{0.611}$

Racking resistance of wall panel

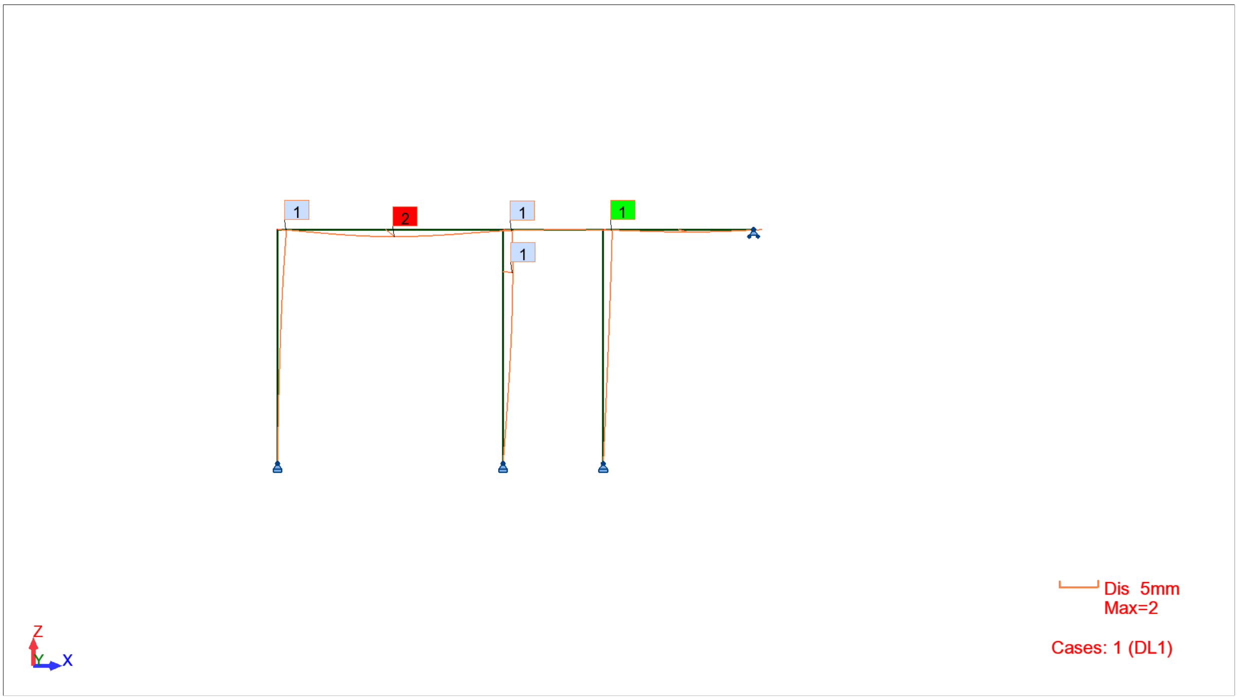
Racking resistance of wall panel $R_R = L \times K_w \times (R_{bp} \times K_{mp} + R_{bs} \times K_{ms}) = \mathbf{5.933}$ kN

Wind Load=
2.7m x 6.2m/2 x 0.7kN/m2= 5.86kN
< Rr hence OK

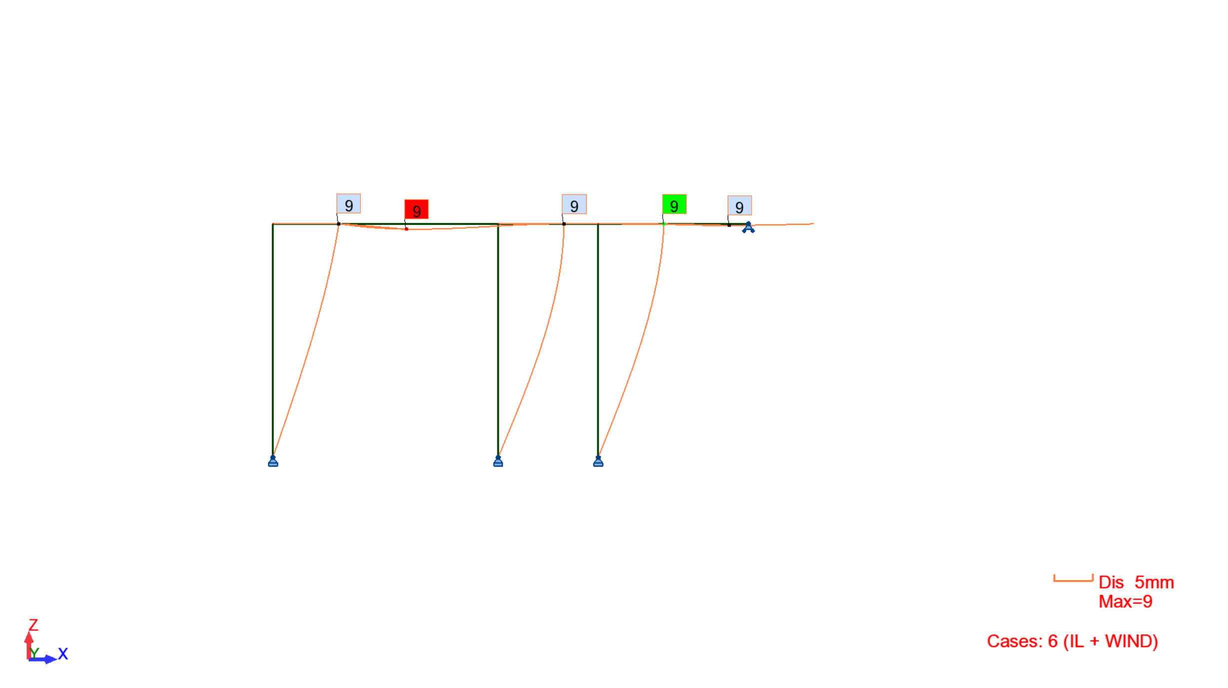
View - Cases: 5 (SLS (DL+IL+W))



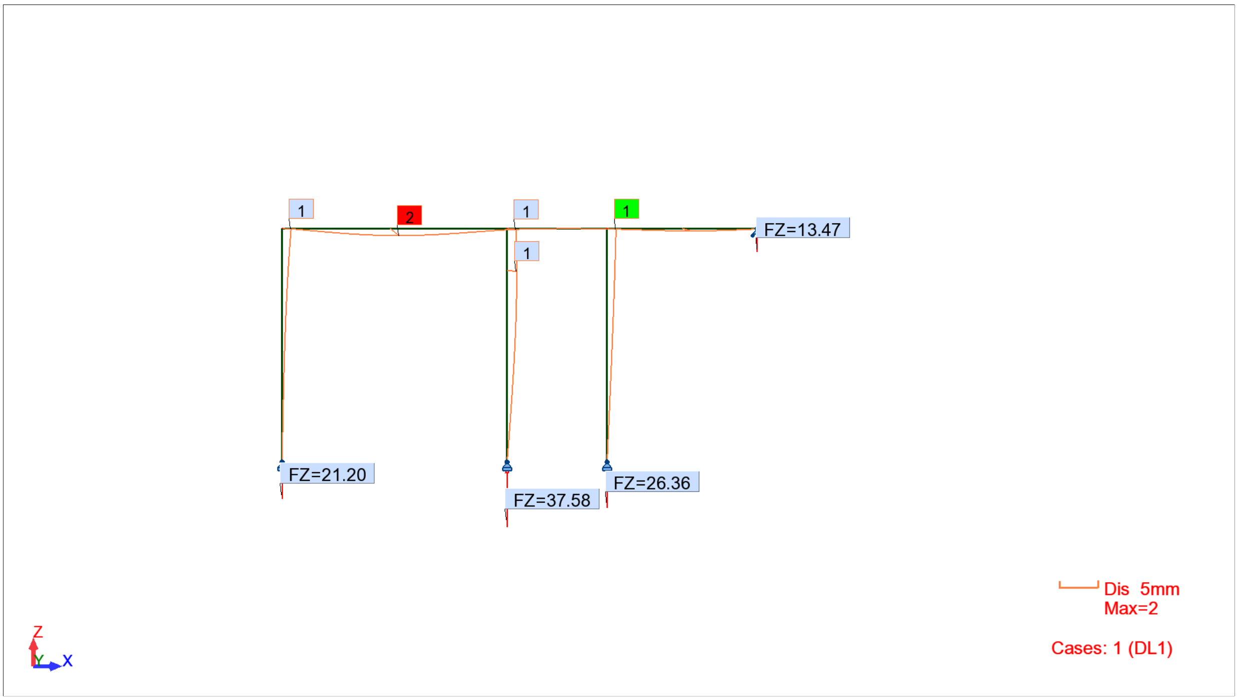
View - Exact deformation(s), Cases: 1 (DL1) 1



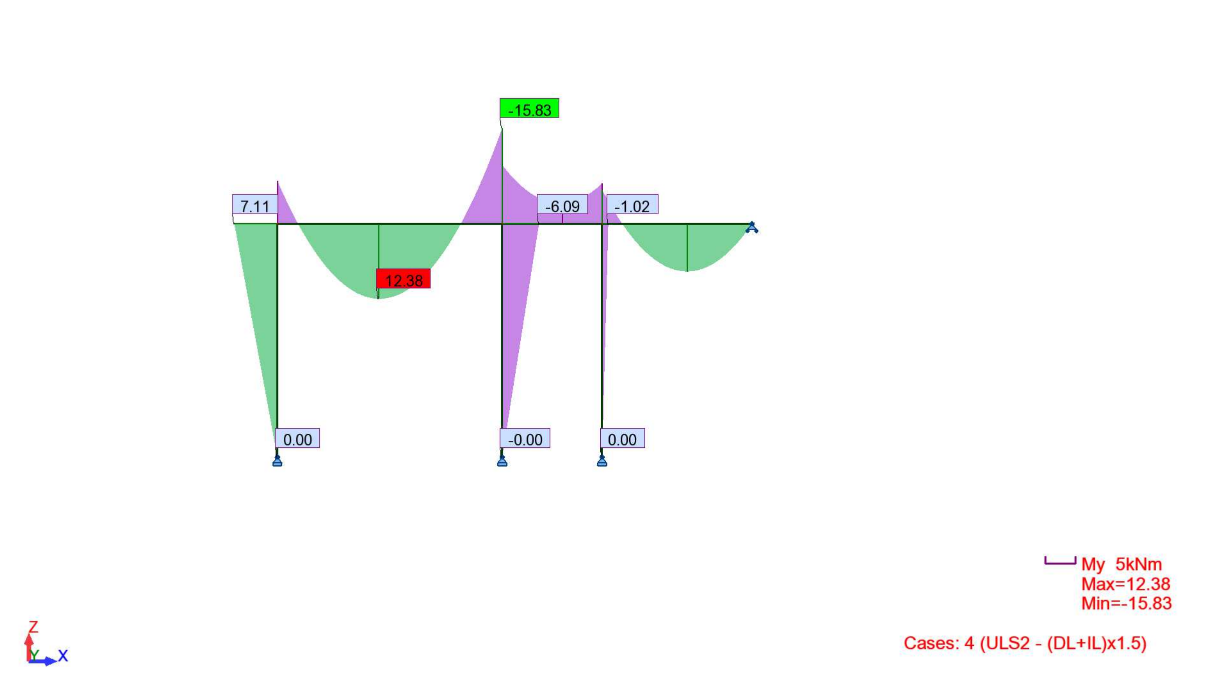
View - Exact deformation(s), Cases: 6 (IL + WIND)



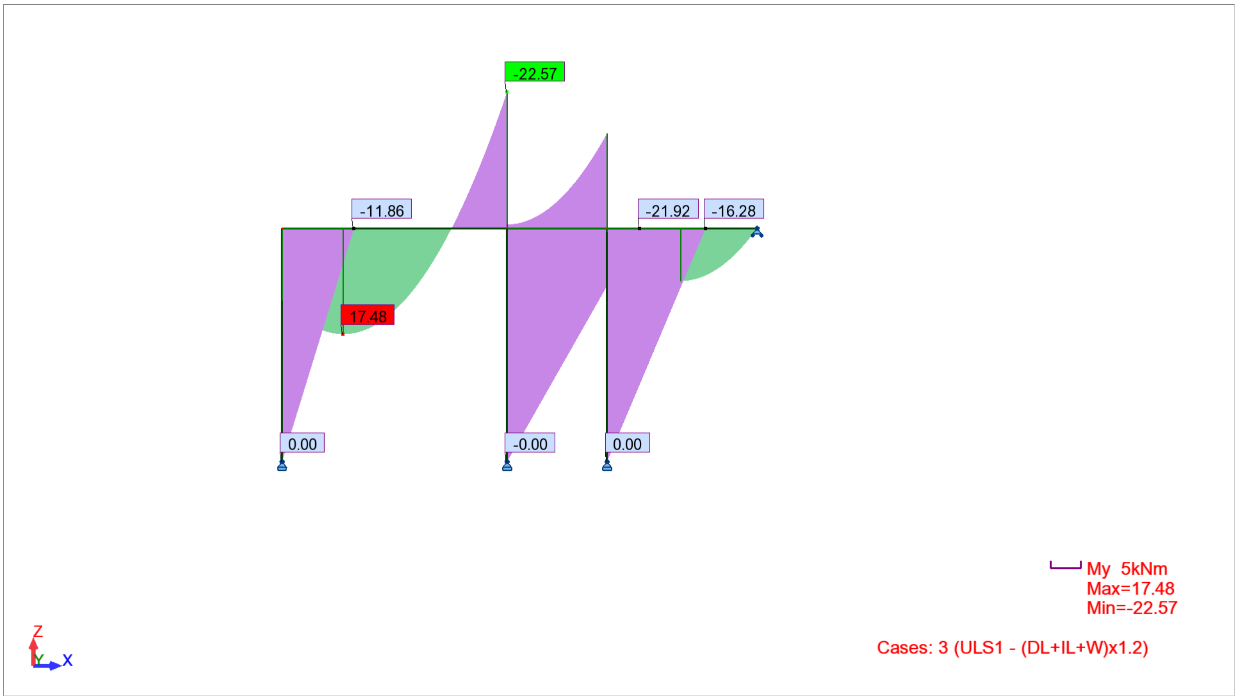
View - Exact deformation(s),Reaction forces(kN), Cases: 1 (DL1) 6



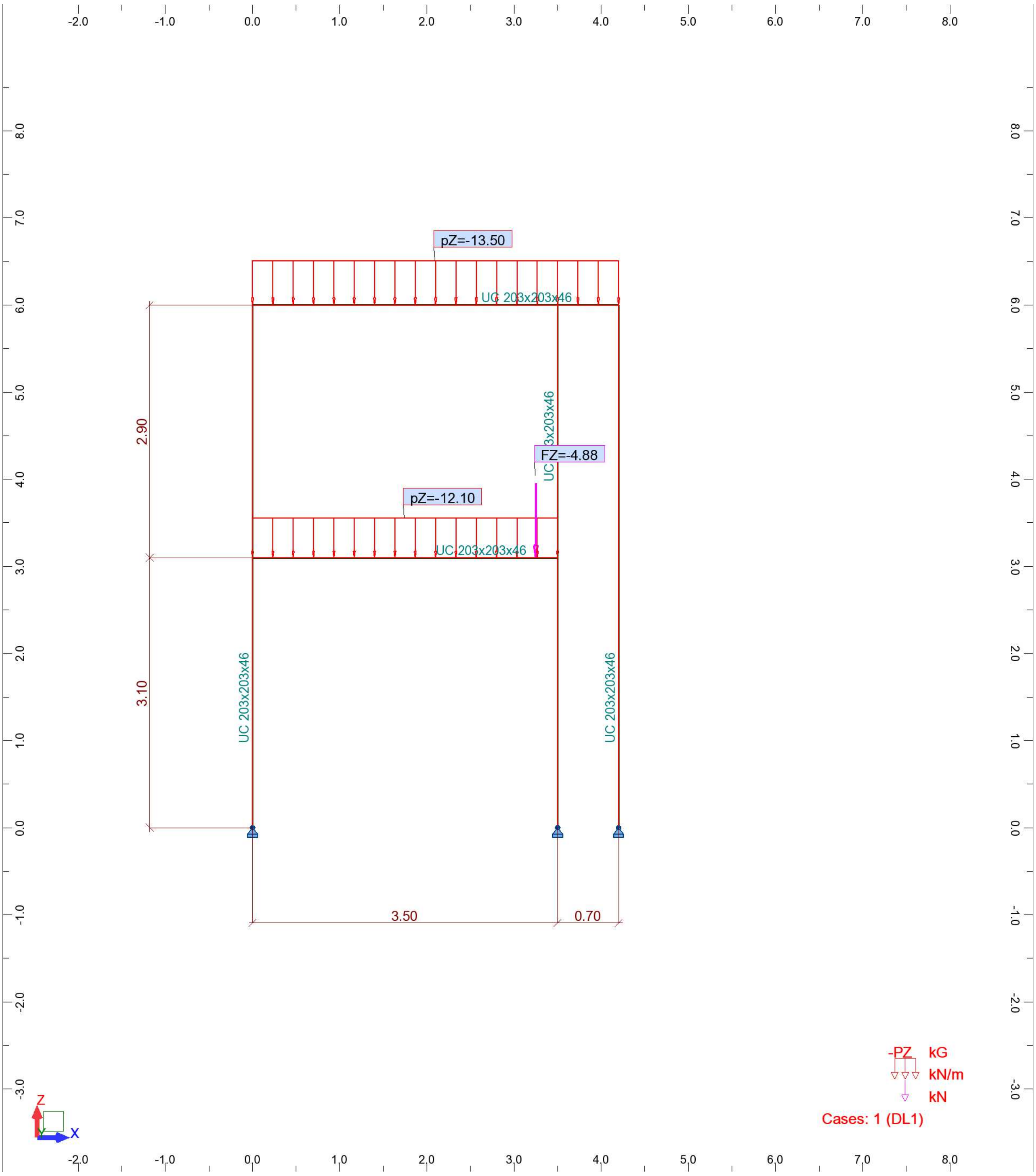
View - MY, Cases: 4 (ULS2 - (DL+IL)x1.5) 1



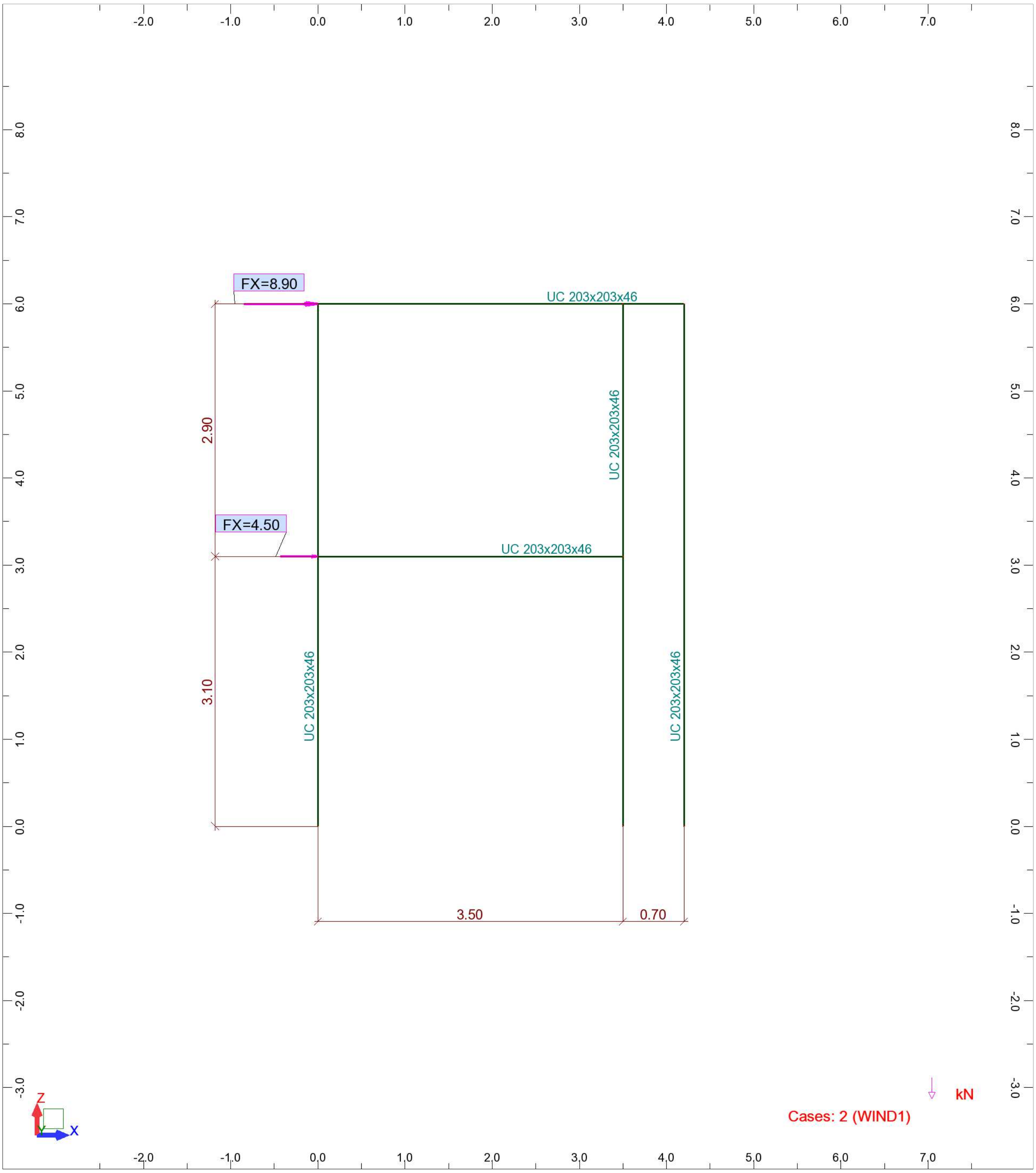
View - MY, Cases: 3 (ULS1 - (DL+IL+W)x1.2) 1



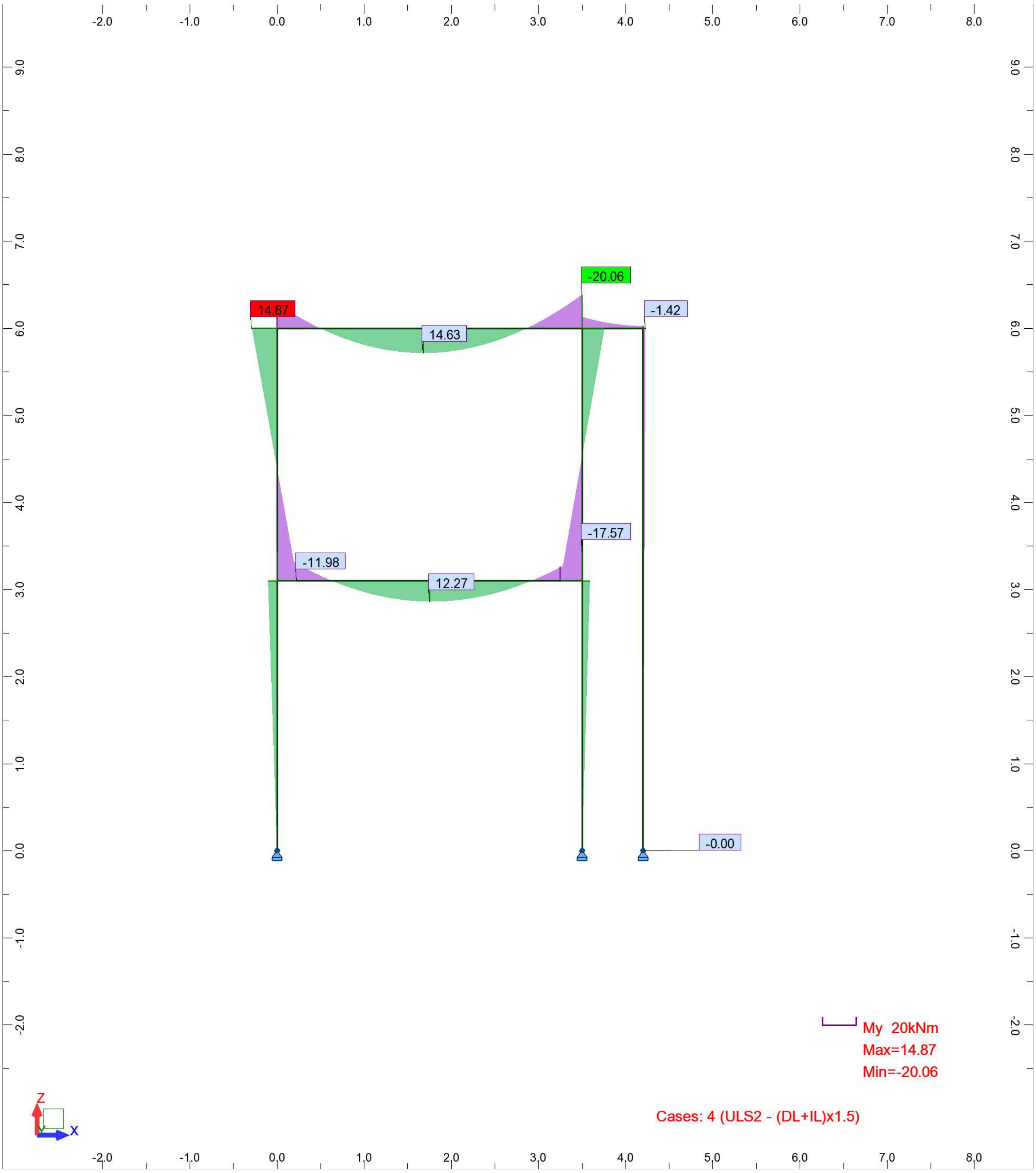
View - Reaction moments(kN*m), Cases: 1 (DL1 + IL1)



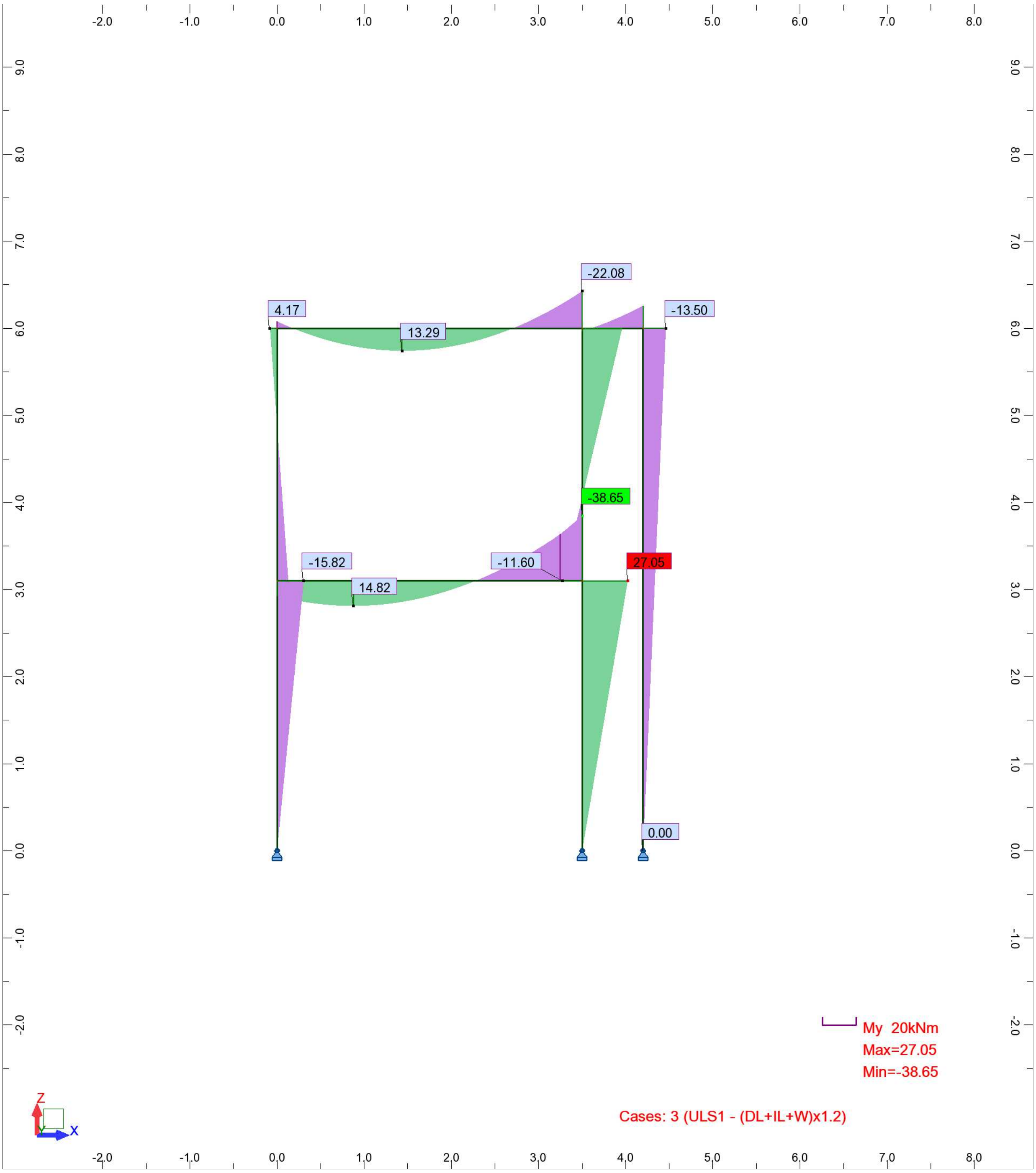
View - Reaction moments(kN*m), Cases: 2 (WIND1) 1



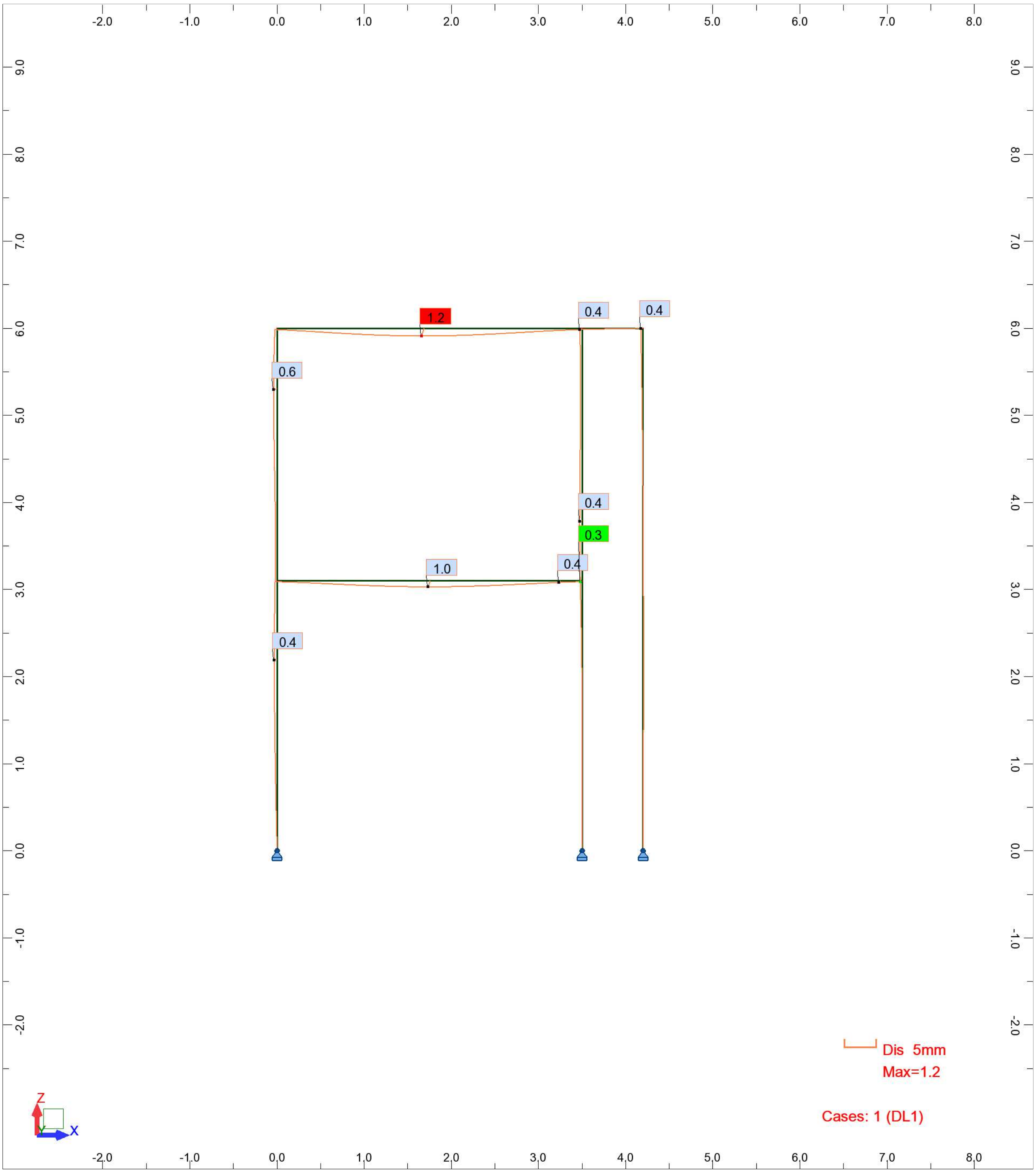
View - MY,Reaction moments(kN*m), Cases: 4 (ULS2 - (DL+IL)x1.5)



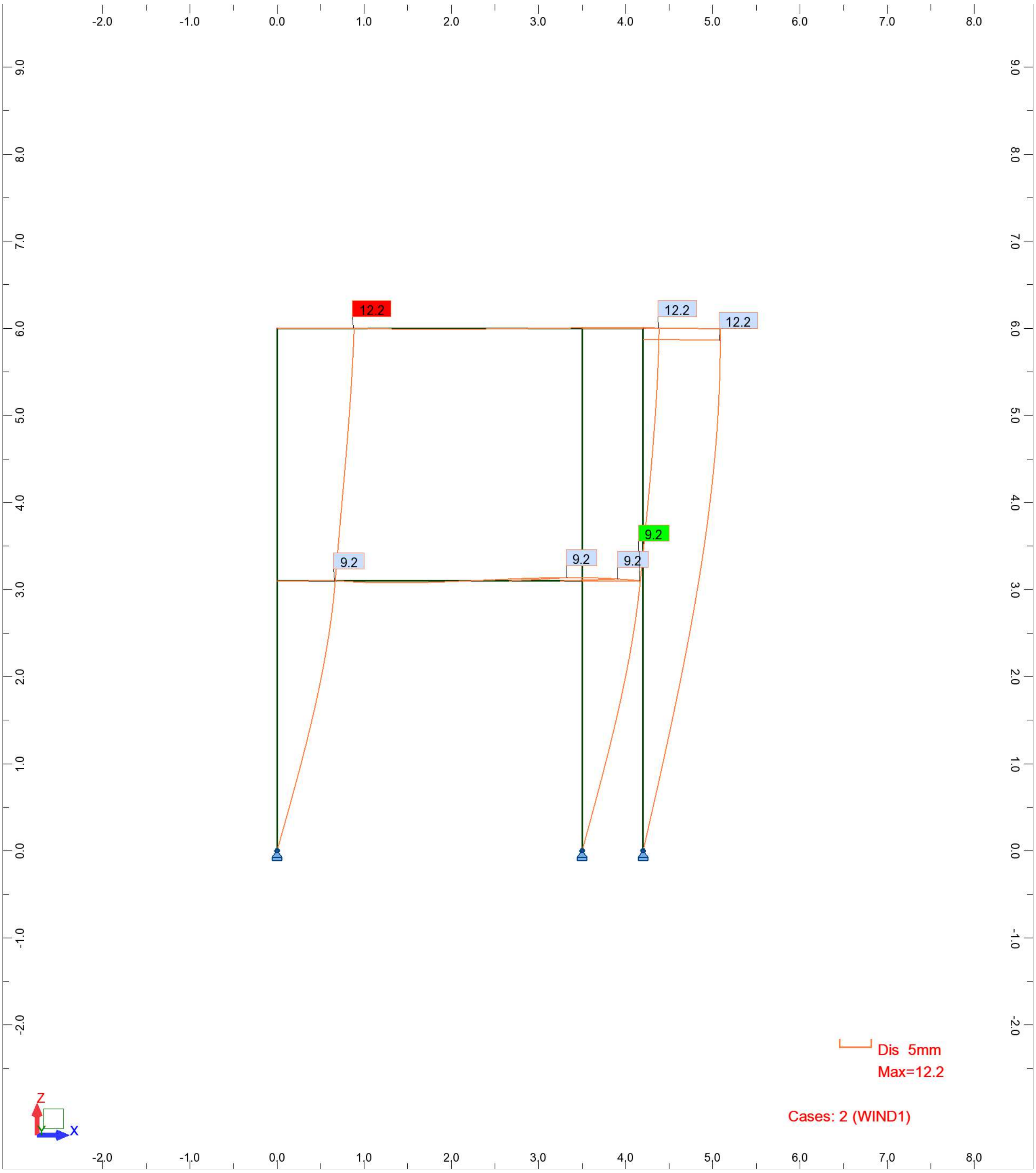
View - MY,Reaction moments(kN*m), Cases: 3 (ULS1 - (DL+IL+W)x1.2)



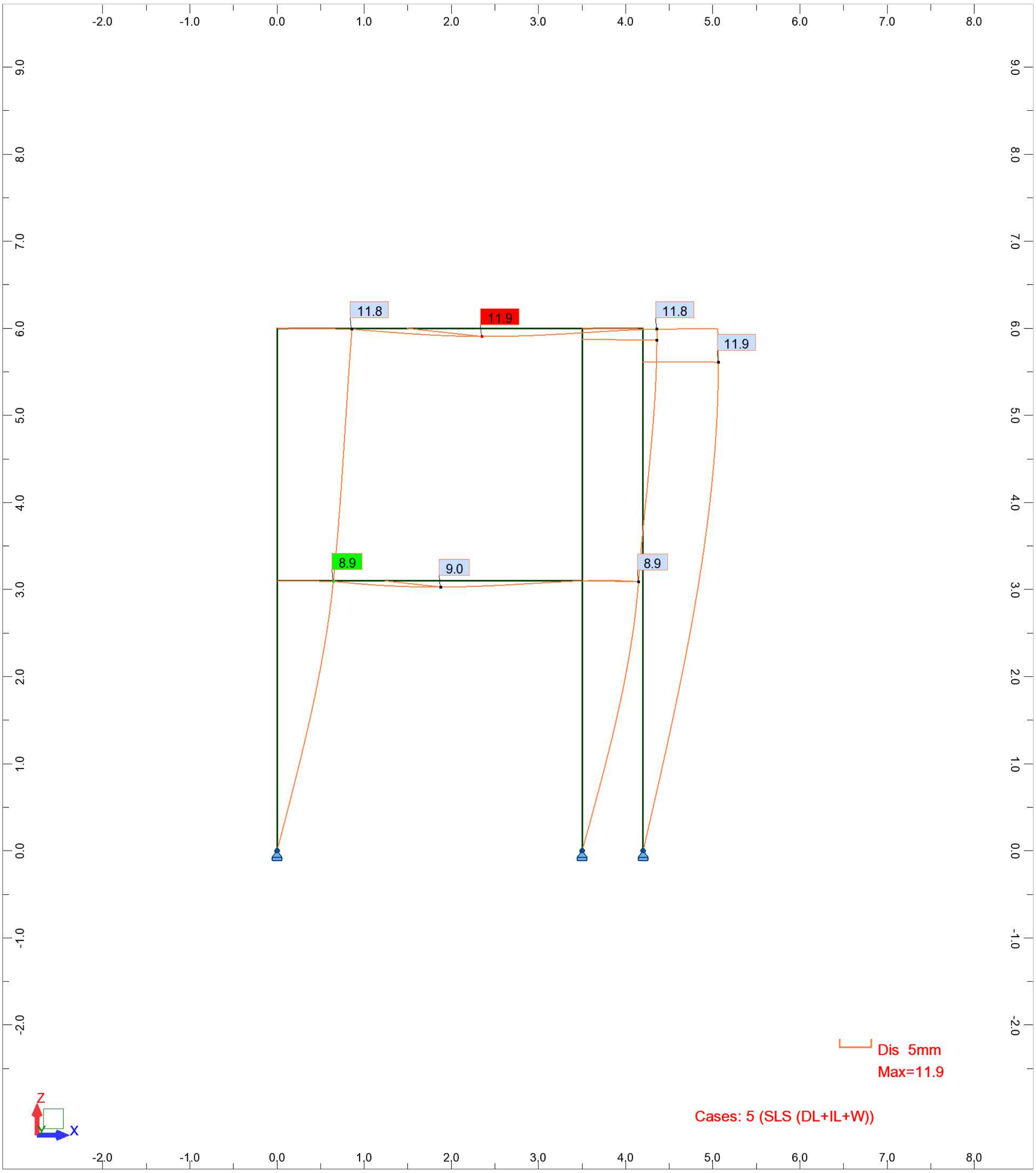
View - Exact deformation(s),Reaction moments(kN*m), Cases: 1 (DL1 + IL1)



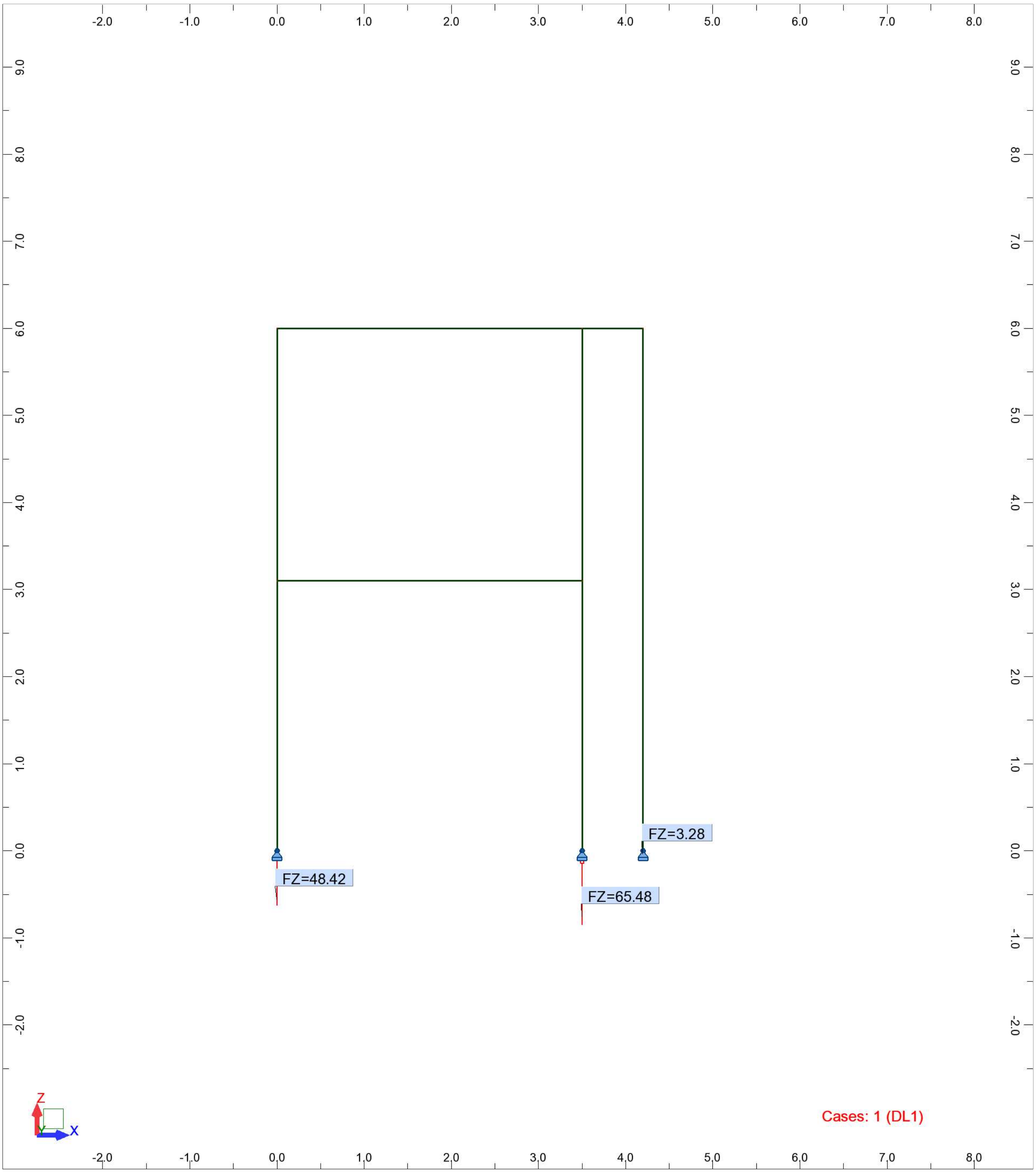
View - Exact deformation(s),Reaction moments(kN*m), Cases: 2 (WIND1)



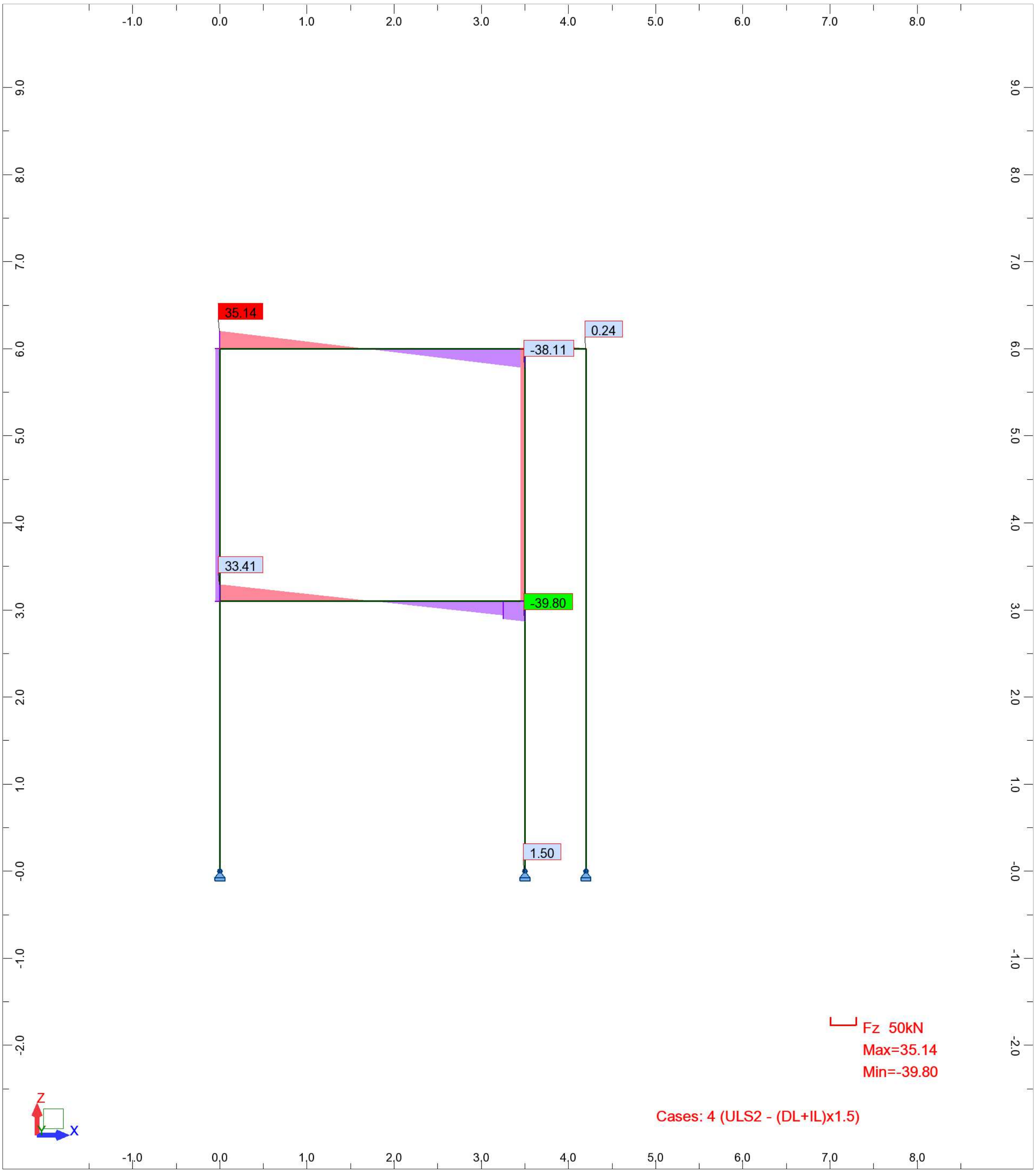
View - Exact deformation(s),Reaction moments(kN*m), Cases: 5 (SLS (DL+IL+W))



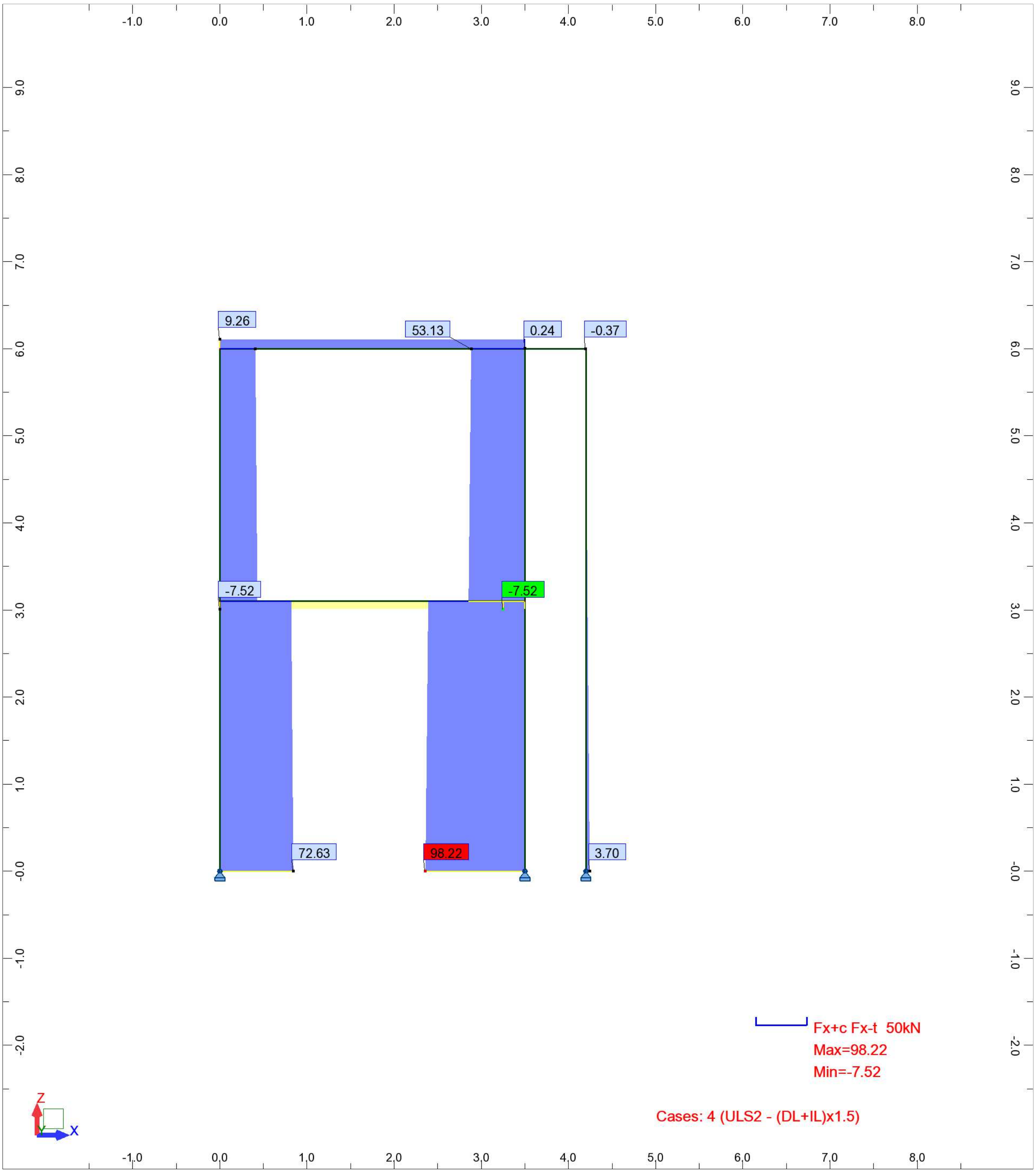
View - Reaction forces(kN),Reaction moments(kN*m), Cases: 1 (DL1 + IL1)



View - FZ,Reaction moments(kN*m), Cases: 4 (ULS2 - (DL+IL)x1.5)



View - FX,Reaction moments(kN*m), Cases: 4 (ULS2 - (DL+IL)x1.5)



SCI	ComFlor		v9.0.33.0
JOB REFERENCE:	85 Camden Mews	Date:	12/17/2018
DECK REFERENCE:	ComFlor 60/1.2/G500	Time:	16:50:34
COMPANY NAME:	Axiom Structures Limited	Job No.:	15005
CLIENT NAME:			
FILENAME: 150MD.pmd **Calcs by:**	KK	**Checked by:**	

SUMMARY OUTPUT

NOTE: SECTION DESIGNED TO BRITISH STANDARDS

Construction Stage:	**PASS**	**Max Unity Factor = 0.49**
Normal Stage:	**PASS**	**Max Unity Factor = 0.27**
Fire Condition:	**PASS**	**Max Unity Factor = 0.00**
Serviceability:	**SATISFACTORY**	**Max Unity Factor = 0.70**

*** SECTION ADEQUATE ***

FLOOR PLAN DATA : (unpropped composite construction with ComFlor 60/1.2/G500 decking)

Beam centres	3.10 m	Span type	SINGLE
Beam or wall width	150 mm	Propping	NONE

PROFILE DATA : (ComFlor 60/1.2/G500 decking)

Depth	60 mm	Pitch of deck ribs	300 mm
Trough width	120 mm	Crest width	130.7 mm
Nominal sheet thickness	1.20 mm	Design sheet thickness	1.16 mm
Deck weight	0.14 kN/m²	Yield strength	500 N/mm²

CONCRETE SLAB : [Normal Weight Concrete ; Mesh : 662/265]

Overall slab depth	150 mm		
Concrete characteristic strength	30 N/mm²	Concrete wet density	2400 kg/m³
Modular ratio	10	Concrete dry density	2350 kg/m³
Bar reinforcement :			
Diameter	8 mm	Yield strength	300 N/mm²
Distance from slab soffit	30 mm		
Mesh reinforcement :			
Mesh	662/265	Yield strength	460 N/mm²
Cover to Mesh	30 mm	Mesh Layers	Single
Account for End Anchorage	NO	Shear connectors per rib	N/A
Diameter of Shear Connectors	N/A		
Screed depth	75 mm	Screed density	2000 kg/m³

SECTION PROPERTIES :

***NOTE - 1: All values of inertia are expressed in steel units
***NOTE - 2: Average inertia is used for deflection calculations for the composite stage
***NOTE - 3: Cracked dynamic inertia is used for natural frequency calculations

DECK PROFILE:

Sagging Inertia, Ixx	132.910 cm4/m	Area of profile (Net), Ap	1721 mm²/m
Hogging Inertia, Iyy	121.600 cm4/m		

COMPOSITE:

Inertia, Ixx - Uncracked	2472 cm4/m	Cracked	1385 cm4/m
Average inertia	1928 cm4/m	Cracked inertia (dynamic)	1592 cm4/m
Shear bond coefficients - Mr	208.86	Kr	0.014700
Concrete volume	0.118 $m^3/m/m$		

LOADS ACTING ON SLAB :

*** NOTE: Slab subjected to uniformly distributed loads (UDL) ONLY

Imposed (occupancy)	1.50 kN/m^2	Partitions	0.50 kN/m^2
Ceilings and services	0.50 kN/m^2	Finishes	0.50 kN/m^2
Self weight of concrete slab (wet)	2.79 kN/m^2	Self weight of decking	0.14 kN/m^2
Self weight of concrete slab (dry)	2.73 kN/m^2	Self weight of screeds	1.47 kN/m^2
Construction load	1.50 kN/m^2	(applied over middle 3m length, 0.75kN/m^2 elsewhere)	

LINE LOADS PERPENDICULAR TO DECK SPAN :

None

LINE LOADS PARALLEL TO DECK SPAN :

None

FIRE DATA :

Design method	FIRE ENGINEERING	Fire resistance period	30 mins
Non-permanent imposed loads	0.0 kN/m^2		

PARTIAL SAFETY FACTORS :

Dead (self weight)	1.40	Imposed	1.60
Super imposed dead	1.40	Fire (occupancy loads)	0.80

Project	85 Camden Mews		**The Concrete Centre**	
Client	Private	Made by	Date	Page
Location	150thk Slab	KK	17-Dec-2018	1
	1 to 5: A to B	Checked	Revision	Job No
	2-WAY SPANNING INSITU CONCRETE SLABS to BS 8110:2005 (Table 3.14)	AP	-	15005
	Originated from RCC94.xls v4.0 © 2006 TCC			

DIMENSIONS

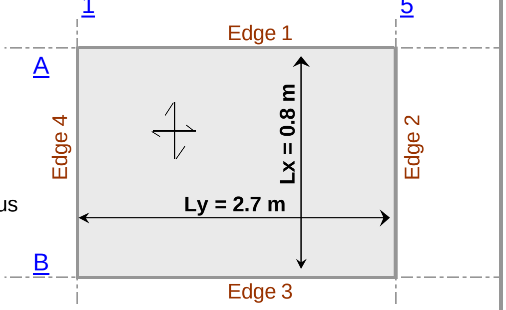
short span, lx	m	**0.80**	
long span, ly	m	**2.70**	
h	mm	150	
Top cover	mm	50	
Btm cover	mm	50	

MATERIALS

fcu	N/mm²	40	γ_c = 1.50
fy	N/mm²	500	γ_s = 1.15
steel class		A	
Density	kN/m³	0	
(Lightweight concrete)			

STATUS VALID DESIGN

Plan



LOADING *characteristic*

Self weight	kN/m²	0.00		
Extra dead	kN/m²	2.00		
Total Dead, gk	kN/m²	2.00	γ_f=	1.40
Imposed, qk	kN/m²	1.50	γ_f=	1.60
Design load, n	kN/m²	5.20		

EDGE CONDITIONS

Edge 1	d	C = Continuous
Edge 2	d	D = Discontinuous
Edge 3	d	
Edge 4	d	

See Figure 3.8 and clauses 3.5.3.5-6

MAIN STEEL		SHORT SPAN x	LONG SPAN y	EDGE 1 Free	EDGE 2 Free	EDGE 3 Free	EDGE 4 Free	BS8110 Reference
ßs		0.134	0.056	0.000	0.000	0.000	0.000	Table 3.14
M	kNm/m	0.4	0.2	0.0	0.0	0.0	0.0	
d	mm	96.0	88.0	96.0	88.0	96.0	88.0	
k'		0.156	0.156	0.156	0.156	0.156	0.156	
k		0.001	0.001	0.000	0.000	0.000	0.000	
Z	mm	91.2	83.6	91.2	83.6	91.2	83.6	3.4.4.4
As req	mm²/m	11	5	0	0	0	0	
As min	mm²/m	195	195	195	195	195	195	Table 3.25
As deflection	mm²/m	11	5	~	~	~	~	
Ø	mm	8	8	8	8	8	8	
Layer		B 1	B 2	T 1	T 2	T 1	T 2	
@	mm	250	250	250	250	250	250	
As prov	mm²/m	201	201	201	201	201	201	
=	%	0.209	0.228	0.209	0.228	0.209	0.228	%
S max	mm	296	272	296	272	296	272	Clause
Subclause		(a)	(a)	(a)	(a)	(a)	(a)	3.12.11.2.7
DEFLECTION								
fs		19	9	0	0	0	0	Eqn 8
Mod factor		2.000						Eqn 7
Perm L/d		34.00	Actual L/d	8.33	*Asx enhanced 0.0% for deflection control*			Table 3.10

TORSION STEEL

			BOTH EDGES DISCONTINUOUS X	BOTH EDGES DISCONTINUOUS Y	ONE EDGE DISCONTINUOUS X	ONE EDGE DISCONTINUOUS Y	
Ø	mm	10					
As req	mm²/m		195		195		3.5.3.5
As prov T	mm²/m		201	201	5000	5000	
Additional As T req	mm²		0	0	0	0	
As prov B	mm²/m		201	201	201	201	

Bottom steel not curtailed in edge strips at free edges

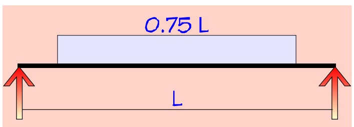
SUPPORT REACTIONS (kN/m char uno) *(See Figure 3.10)*

Sum ßvx = 1.136
Sum ßvy = 0.667

Table 3.15

		EDGE 1 A, 1-5	EDGE 2 5, B-A	EDGE 3 B, 1-5	EDGE 4 1, B-A
ßv		0.568	0.333	0.568	0.333
Dead	kN/m	0.91	0.53	0.91	0.53
Imposed	kN/m	0.68	0.40	0.68	0.40
Vs	kN/m	2.4	1.4	2.4	1.4

equations 19 & 20



OUTPUT/SUMMARY

	SHORT SPAN	LONG SPAN	EDGE 1 A, 1-5	EDGE 2 5, B-A	EDGE 3 B, 1-5	EDGE 4 1, B-A
PROVIDE MAIN STEEL	H8 @ 250 B1	H8 @ 250 B2	H8 @ 250 T1	H8 @ 250 T2	H8 @ 250 T1	H8 @ 250 T2

ADDITIONAL TORSION STEEL	CORNER 1 1A	CORNER 2 5A	CORNER 3 5B	CORNER 4 1B
X direction				
Y direction				

placed in edge strips

CHECKS Lx > Ly	BAR Ø < COVER	SINGLY REINFORCED	MIN SPACING	MAX SPACING	DEFLECTION	GLOBAL STATUS
OK	OK	OK	OK	OK	OK	VALID DESIGN

Project	Spreadsheets to BS 8110	The Concrete Centre	
Client	85 Camden Mews	Made by: KK	Date: 17-Dec-18	Page: 1
Location	Basemenet	Checked	Revision: -	Job No: 15005

PAD FOUNDATION DESIGN to BS 8110:2005 — Single column base

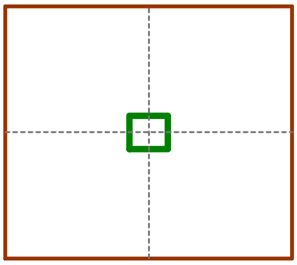
Originated from **RCC81.xls** v4.1 on CD

MATERIALS

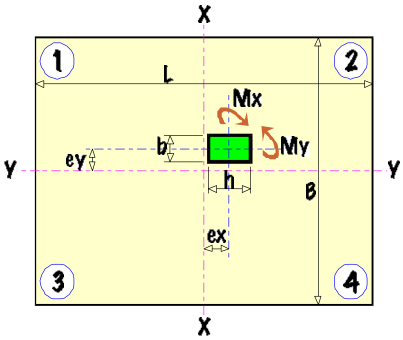
fcu	35	N/mm²	h agg	20	mm	γ_c	1.5	concrete
fy	500	N/mm²	cover	50	mm	γ_s	1.15	steel
Densities - Concrete	24	kN/m³	Soil	21	kN/m³	steel class	A	
Bearing pressure	150	kN/m² (net allowable increase)						

DIMENSIONS mm

BASE		COLUMN	
L =	1500	h =	200
B =	1500	b =	200
depth H =	500		
ex =	0	ey =	0



Plot (to scale)



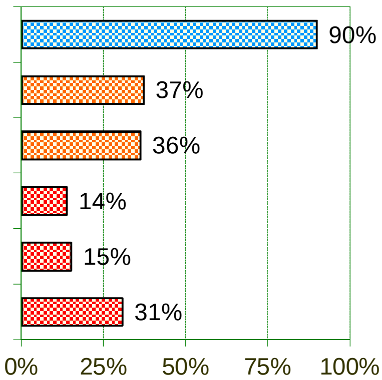
Key

COLUMN REACTIONS kN, kNm *characteristic*

	DEAD	IMPOSED	WIND
Axial (kN)		300.0	
Mx (kNm)			
My (kNm)			
Hx (kN)			
Hy (kN)			

Overturning FOS = Large
Uplift FOS = infinite

STATUS VALID DESIGN



BEARING PRESSURES kN/m² *characteristic*

CORNER	1	2	3	4
no wind	134.8	134.8	134.8	134.8
with wind	134.8	134.8	134.8	134.8

REINFORCEMENT

Mxx =	67.6	kNm	Myy =	67.6	kNm
b =	1500	mm	b =	1500	mm
d =	444	mm	d =	432	mm
As =	369	mm²	As =	379	mm²
PROVIDE	9 H12 @ 175 B1		PROVIDE	9 H12 @ 175 B2	
As prov =	1018	mm²	As prov =	1018	mm²

BEAM SHEAR

Vxx =	67.2	kN at d from col face	Vyy =	73.6	kN at d from col face
v =	0.101	N/mm²	v =	0.114	N/mm²
or Vxx =	0.0	kN at 2d from col face	or Vyy =	0.0	kN at 2d from col face
v =	0.000	N/mm²	v =	0.000	N/mm²
vc =	0.368	N/mm²	vc =	0.374	N/mm²

PUNCHING SHEAR

d ave =	438	mm	u crit =	0	mm
As prov =	0.155	%	v max =	1.453	N/mm² at col face
v =	0.000	N/mm²	vc =	0.371	N/mm²

200PFC to 254UC connection
Connection Design Beam to Beam

Vuls = 50 kN

Check 1: Essential detailing requirements

Use 10thk end plate, 2M16 8.8 bolts

D =	18	mm
min. pitch = 2.5 x D =	45	mm
gauge =	90	mm
min. edge distance =	26	mm

Check 2: Shear capacity of bolt group connecting endplate to supporting beam

Bolts: M16 8.8 =

Ps = 58.9 kN
no bolts = 2

Vb = V/bolts = 25 kN < Ps = 58.9 kN OK

Bearing strength:
pb = 430 N/mm2
Bearing capacity per bolt:
Pbs = dbolt x pb x tmin = 16mm x 430N/mm x 10mm = 68.8 kN

Vb = V/bolts = 25 kN < Pbs = 68.8 kN OK

Check 3: Capacity of endplate

Shear capacity of 10thk endplate:
Pv = 0.6 x 10mm x (45 mm + 26 mm) x 275 N/mm2
Pv = 117.2 kN > 0.5 x Vuls = 25 kN OK

Check 4: Shear capacity of the supported beam web at the endplate

py = 355 N/mm2
notch deep = 30 mm
d notched = 170 mm
Pv = 0.6 x t x d notched x 0.9 x 355 N/mm2
Pv = 195.5 kN > Vuls = 50 kN OK

Check 5: Capacity of fillet welds between supported beam and end plate

E web weld = 2 x d notched = 340 mm

Shear = V / Eweb 0.15 kN/mm < pL = 0.924 kN/mm OK

Check 6: Bending capacity of reduced supported beam section at the notch

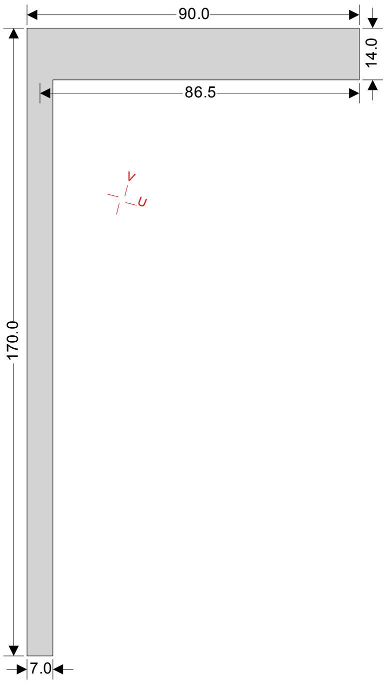
py = 355 N/mm2
L notch = 128 mm
Znotched = modulus of notched section * = 29 cm3
* Refer to next page for Tedds calculations
Mc = moment capacity of notched section = py x Znotched 10.3 kNm
Mapplied = Vuls x (Lnotch + tendplate) 6.9 kNm

Mc = 10.3 kNm > Mapplied = 6.9 kNm OK

AXIOM STRUCTURES Axiom Structures Ltd	Project 85 Camden Mews			Job no. 15005	
	Calcs for Notched PFC - section properties			Start page no./Revision 1	
	Calcs by KK	Calcs date 02/01/2019	Checked by	Checked date	Approved by	Approved date

CALCULATION OF SECTION PROPERTIES

Tedds calculation version 2.0.05



Area

A = **23.52** cm^2

2nd moment of area

I_{uu} = **725.** cm^4 I_{vv} = **107.** cm^4 I_{xx} = **646.** cm^4 I_{yy} = **186.** cm^4

Radius of gyration

r_{uu} = **55.5** mm r_{vv} = **21.4** mm r_{xx} = **5.2** cm r_{yy} = **2.8** cm

Plastic section modulus (only shapes with all rectangles at 90 degs)

S_{xx} = **93.9** cm^3 S_{yy} = **52.4** cm^3

Distance to combined centroid

X_e = **0.0** mm Y_e = **0.0** mm

Distance to equal axis area (only shapes with all rectangles at 90 degs)

X_p = **-18.8** mm Y_p = **33.4** mm

Elastic section modulus

Z_{xx} = **52.3** cm^3 Z_{yy} = **29.0** cm^3