



Project: BRUCE CASTLE

Job Ref: 22129

Drawn: RF

Scale: **NTS**

Sheet No: 1 - 0

Rev:

Element: STRUCTURAL CALCULATIONS

Date: JUNE '22

Project:	BRUCE CASTLE		Job Ref:	22129	
Drawn:	RF	Scale:	NTS	Sheet No:	1 - KEY
Element:	KEY.			Date:	JUNE '23

Key.

FLOOR LOADS.

G.A.'s

MEMBER LOADING SHEETS

WALLS AND FOUNDATIONS

PADSTONES

SPLICES TYPE "B"

LATERAL STABILITY OF EAST WALL

FREE

MEMBER DESIGN.

MEMBER SUMMARY

2nd J.L. BM's BETWEEN JOISTS OF WEST BAY.

SPLICE DETAILS OF ABOVE TYPE "A"

SPLICE TYPE "C"

KEY

01

02 - 03

04 - 06

07 - 10

11 - 12

13 - 14

15

16

17 - 38

39.

42 - 47

48 - 51

52 - 55

Project: **BRUCE CASTLE**

Job Ref: **22129**

Drawn: **RF**

Scale: **NTS**

Sheet No: **1-01**

Rev:

Element: **LOAD ASSESSMENT**

Date: **JUNE '23**

FLOOR LOADS, FOR 1ST FLR

BOARDS	.15
FIRTINGS	.05
JOISTS	.15
BEECHES	.10
C/D	.10
CERAMIC	.25
PARTITIONS	.0
	<u>.80</u>
	1 KN/m² DL

2.5 1 KN/m² LL

INITIALLY BOTH FLOORS WERE SET AT A LL = 2.5 KN/m² AND CALC RUN ACCORDINGLY. MIDWAY THROUGH IT BECAME CLEAR THAT WALLS AND, PARTICULARLY FOUNDATIONS WERE NOT UP TO THIS SO LL WAS RESET TO 2.5 KN/m² ON 2ND FL AND 2.0 " " 1ST "

(REMOVED)

2ND FL PARTITION E/O

$$\text{PARTITION @ } .15 + 0.15 + .15 = 0.45 \text{ KN/m}^2$$

a) 2.10m H

∴ say **1.0 KN/m** LINE LOAD + ROOF

ROOF LOADS.

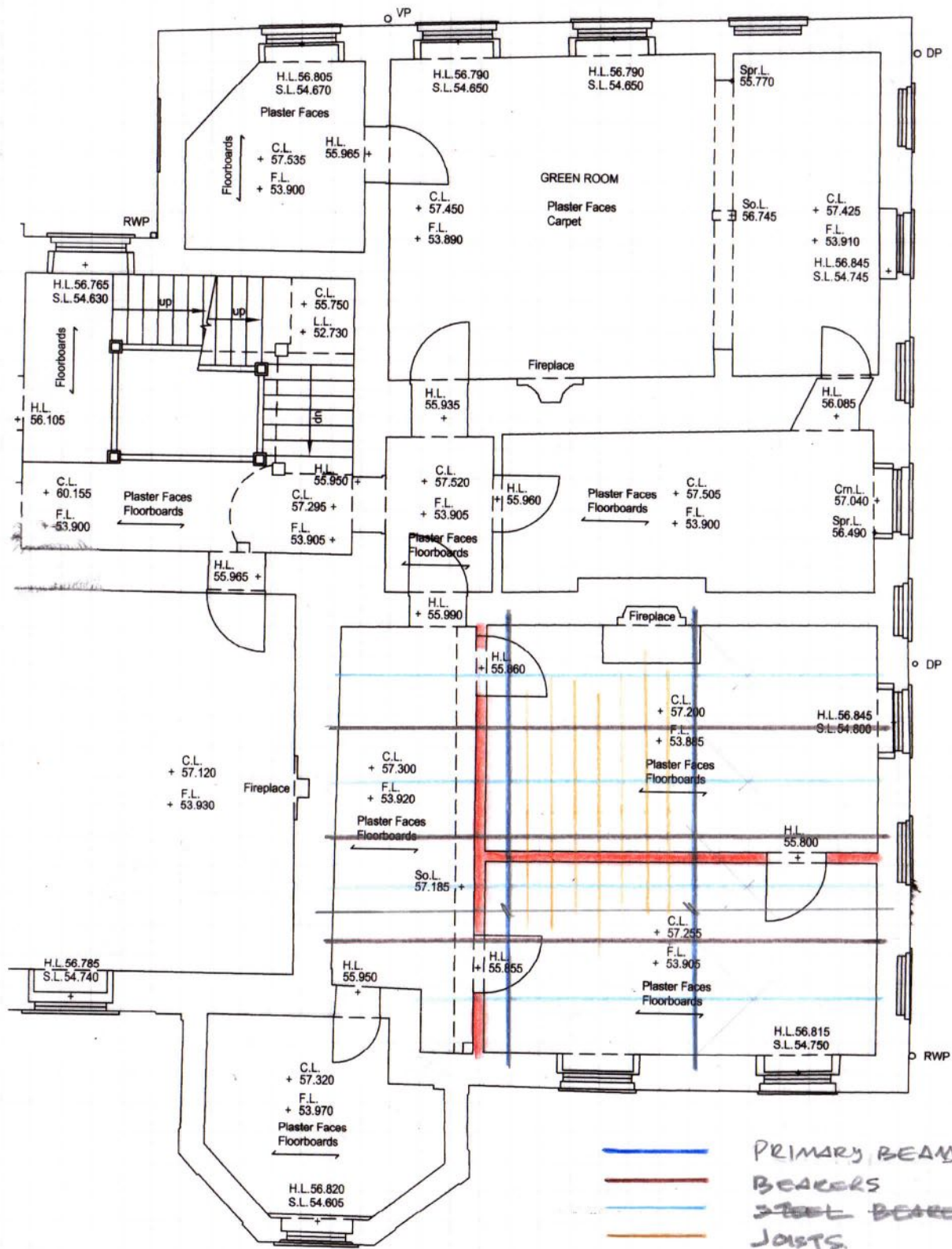
SLATES	.6
FELT	.1
RATERS	.25
C/Ds	.25
C.	<u>.25</u>
	1.35 × 1.4 = 1.9

LL. **0.75**

ROOF LOAD DOWN PARTITION.

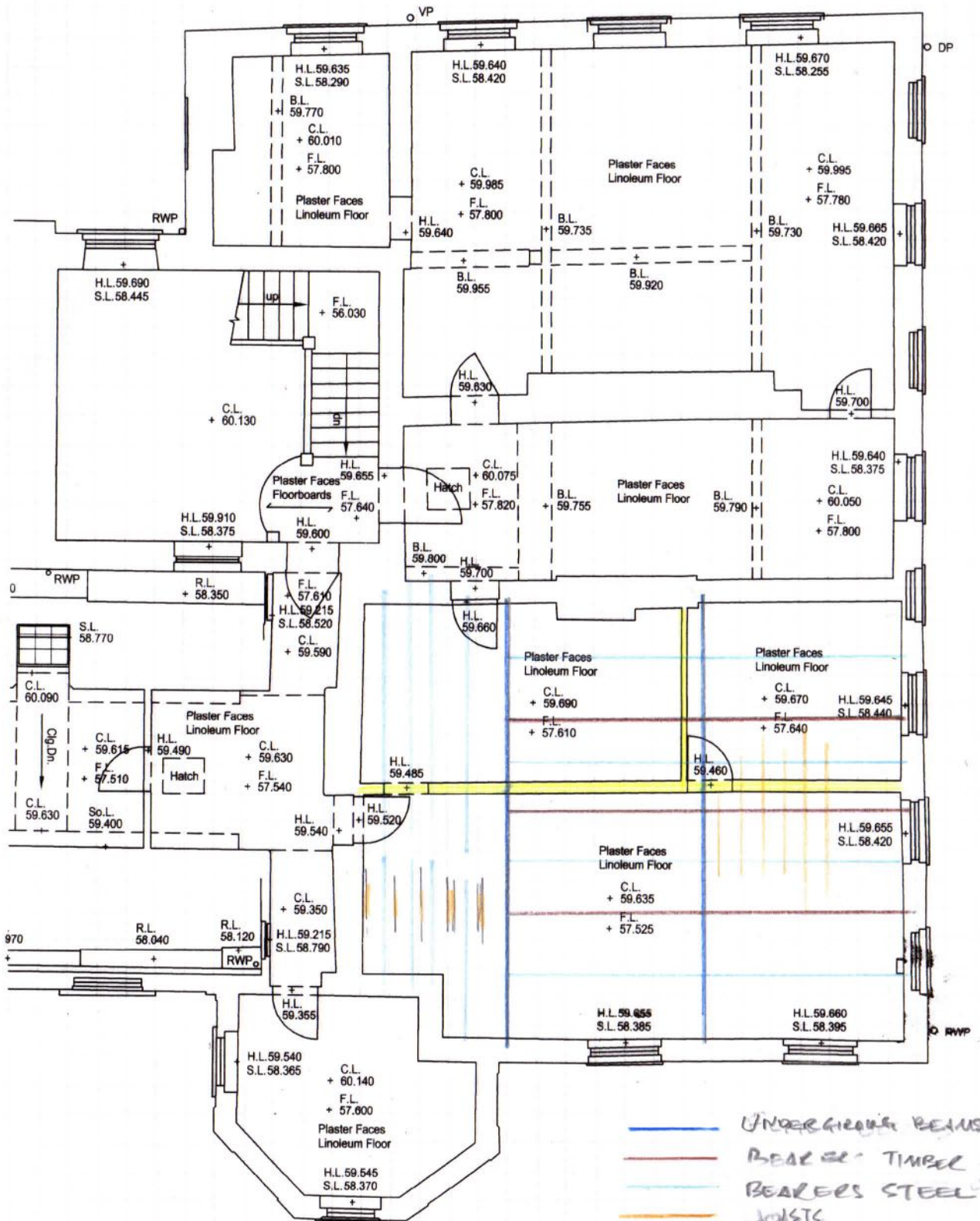
$$\frac{1}{2} \times 7.5 \times 1.9 = 7.1 \text{ KN/m} \pm 1.0 = 8.1 \text{ KN/m}$$

$$\frac{1}{2} \times 7.5 \times 0.75 = 2.8 \text{ KN/m} = 2.8 \text{ KN/m}$$



PARTITIONS TO BE REMOVED

omitted + SIDE SPLINED TO BEAMERS

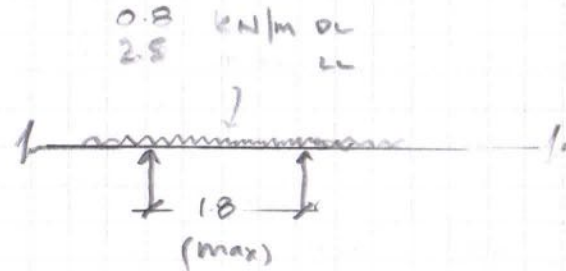


- UNDERGROUND BEAMS
- BEAMS - TIMBER
- BEAMS - STEEL
- JOISTS
- PARTITION (RETAINED)
- STEEL JOISTS

(But surveyed pattern varies)

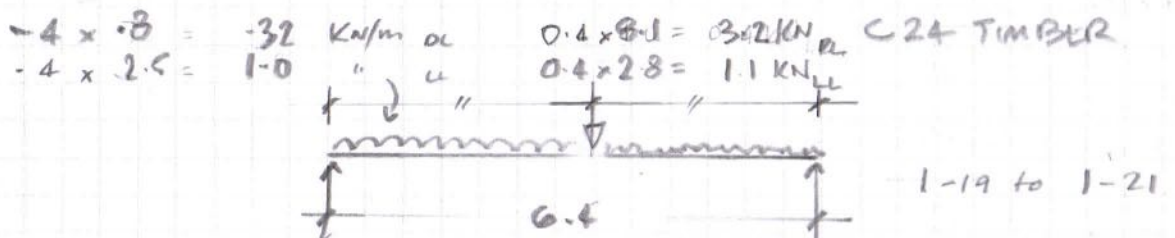
1. EXISTING JOISTS (OVER BEARERS) 1st + 2nd fl.

140 x 65 @ 400 mm C24



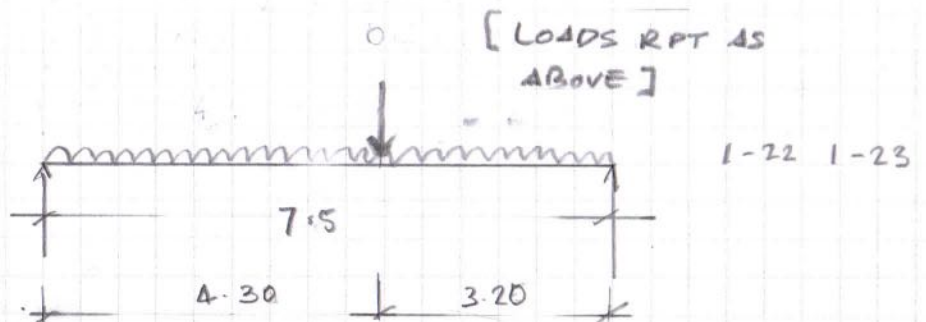
1-16 to 1-18.

2. EXISTING JOISTS IN WESTERN BAY OF 2ND FL (6.40 m SP)



270 x 80 INDIVIDUAL JOISTS DESIGNED AS INDIVIDUAL BEAMS
"BEAMS" @ 400 mm

3. EXISTING JOISTS IN WESTERN BAY OF 2ND FL (7.5 m SP)

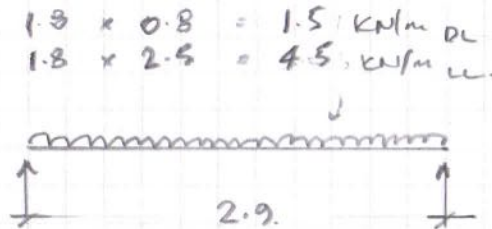


270 x 160 INDIVIDUAL JOISTS DESIGNED AS INDIVIDUAL BEAMS

2/3

4. EXISTING BEARERS.

200 x 75 C24

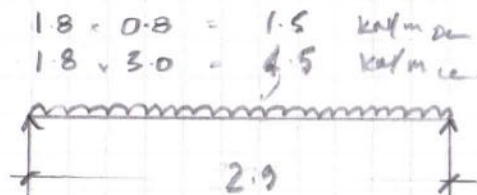


$$1.8 \times 0.8 = 1.5 \text{ KN/m DL}$$

$$1.8 \times 2.9 = 4.5 \text{ KN/m LL}$$

5. NEW STEEL BEARERS.

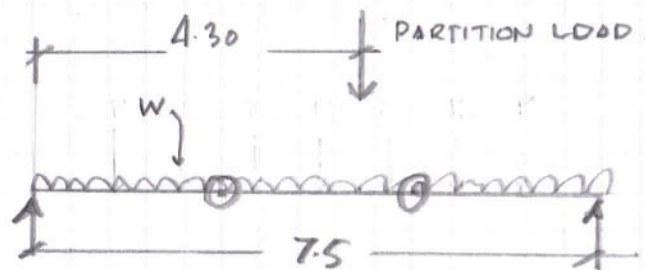
2x 100 x 50 PFC



$$1.8 \times 0.8 = 1.5 \text{ KN/m DL}$$

$$1.8 \times 3.0 = 4.5 \text{ KN/m LL}$$

6. 2ND FL MAIN BEAM

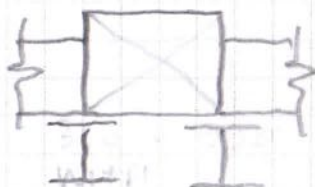


$$W = 0.8 \times 3.25 = 2.60 \text{ KN/m} + 1.5 (\text{SW}) = 4.1 \text{ KN/m DL}$$

$$2.9 \times 3.25 = 8.13 \text{ KN/m LL}$$

1-31 - 1-34

PARTITION LOAD



$$3.25 \times 8.1 = 26.3 \text{ KN DL}$$

$$3.25 \times 2.9 = 8.1 \text{ KN LL}$$

305 x 165 x 40

Project: BRUCE CASTLE
S.E. WING.

Job Ref: 22129

Drawn: RF.

Scale: NTS

Sheet No: 1-06

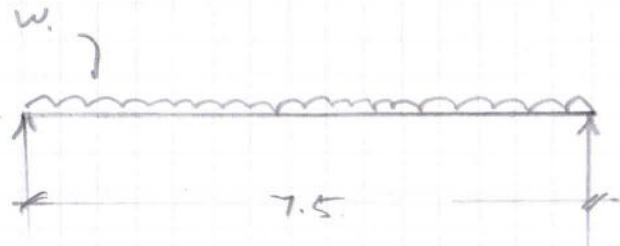
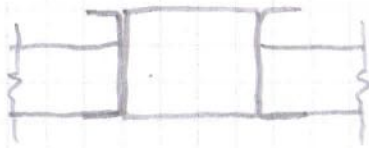
Rev:

Element: MEMBER LOADING SHEET.

Date: SEPT. '23

⑦ 1st FL MAIN BEAM WITH REC SIDE SPLICES ^{2/3}

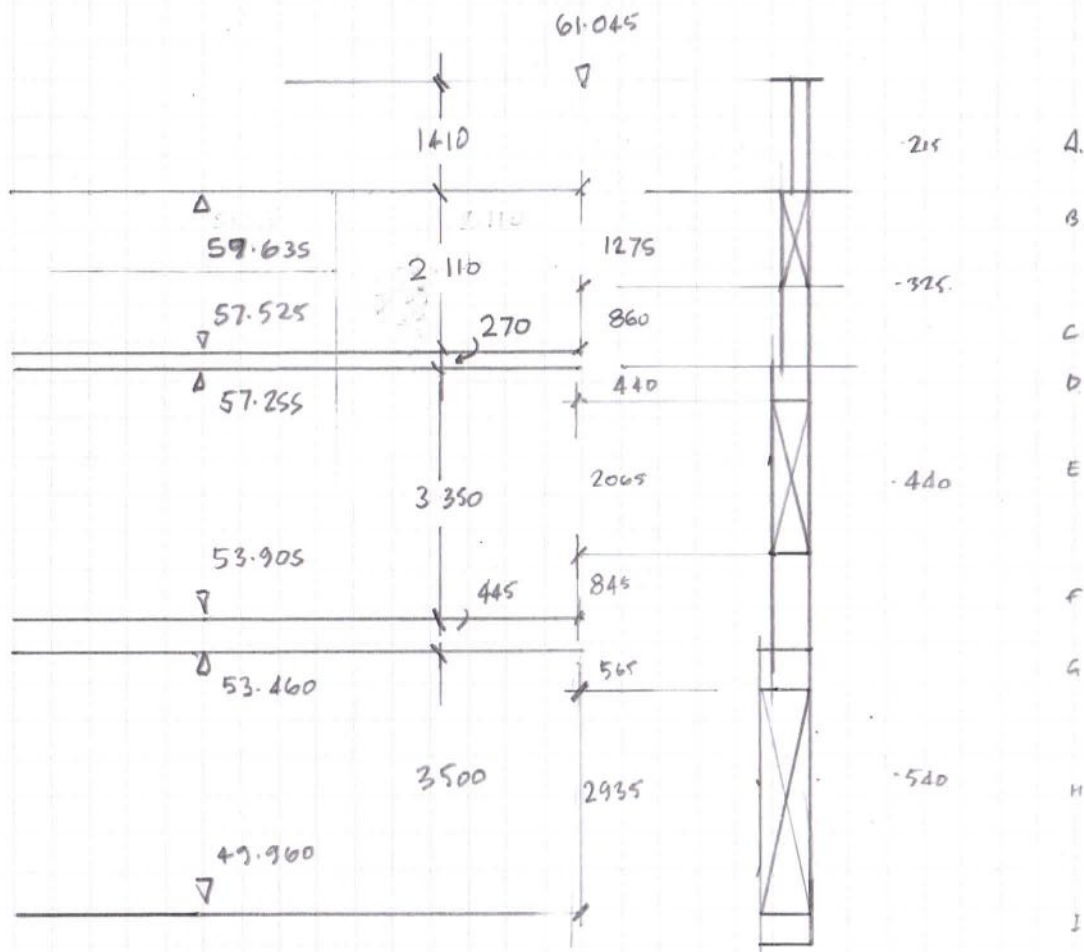
$$W = \begin{matrix} 0.8 \times 3.25 & (+1.5) \\ 2.5 \times 3.25 \end{matrix} = \begin{matrix} 4.1 \text{ KN/m DL} \\ 8.13 \text{ KN/m LL} \end{matrix}$$



1-35 to 1-38.



Project: BRUCE CASTLE S.E. WING.		Job Ref: 22129	
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Element: ASSESSMENT OF WALLS AND FOUNDATIONS		Date: SEPT. '23	



	W.	H	Windows factor	L.	x.		KN
A	215	x 14	x 1.0	x 8.6	x 18	=	46
B	325	x 1.275	x 65%	x 8.6	x 18	=	42
C	325	x (.86 + .27)	x 1.0	x 8.6	x 18	=	57
D	440	x .44	x 1.0	x 8.6	x 18	=	30
E	440	x 2.065	x 65%	x 8.6	x 18	=	92
F	440	x (.445 + 8.45)	x 1.0	x 8.6	x 18	=	86
G	540	x 5.65	x 1.0	x 8.6	x 18	=	47
H	540	x 2.935	x 75%	x 8.6	x 18	=	184
I	840	x 13	x 1.0	x 8.6	x 18	=	20

604 KN.

Roof DL $(DL/m^2) (R) (L = 0.75 \times 8.6)$
 $1.9 \times \frac{7.5}{2} \times 6.5 = 46$
 (21.4)

2ND FL DL $0.8 \times \frac{7.5}{2} \times 6.5 = 20$

1st FL DL $= 20$
 86 KN.

Roof LL $0.75 \times \frac{7.5}{2} \times 6.5 = 18$ 18

2ND FL LL $1.0 \times \frac{7.5}{2} \times 6.5$ assume 2.5 KN/m² = 24 61

1st FL LL $1.0 \times \frac{7.5}{2} \times 6.5$ assume 2.0 KN/m² = 24 49

ASSESSMENT OF
EXISTING CONDITION.

66 128

Project: BRUCE CASTLE S.E. WING.		Job Ref: 22129	
Drawn: RF	Scale: NTS	Sheet No: 1-09	Rev:
Element: ASSESSMENT OF WALLS AND FOUNDATIONS		Date: SEPT '23	

tot DL ON WALLS EXISTING $604 + 86 = 690 \text{ KN.}$
 " LL " " " $= 66 \text{ KN.}$
 " DL " " NEW. $604 + 86 + 6 = 696 \text{ KN (SA4)}$
 " LL " " " $= 128 \text{ KN.}$

$$\therefore \text{FND INCREMENT \%} = \frac{(696 + 128) - (690 + 66)}{(690 + 66)} \cdot \frac{69}{756} = 9.1\%$$

< 10%

OK

TR9 ASHLEY ST. WALLS 21' 4" 2.5' 11' 4" 11' 4" 11' 4"

2nd fl. 3' 5" 2nd fl. 2' 5"

1st fl. 3' 5" 1st fl. 2' 5"

7' 5"

2nd fl. 3' 5" 2nd fl. 2' 5"

1st fl. 3' 5" 1st fl. 2' 5"

CONSIDER INCREMENT $\frac{1}{2}$ WAY DOWN E.

DL WALL	=	46	DL ROOF	46	LL ROOF	18
		42				
		57	DL FLOOR	20	LL FLOOR	24
		30	2nd FL (ONLY)		(ASSUMING 1.0 KN/m ²)	
		<u>46</u> ($\frac{1}{2} \times 92$)				
		221		66		42

DL ON WALLS EXISTING	221 + 66	=	287
LL " " "		=	42
DL ON " NEW	287 + 3	=	290
LL " " "	2.5 x 24	=	60

$$\frac{(290 + 60) - (287 + 42)}{(287 + 42)} = \frac{21}{329} \quad 6.4\% < 9.1\%$$

... see previous for limits.

THEREFORE.

LOAD INCREMENT LIMITED BY GRD FLR WALLS / FND INCREMENT

THE MOST DESIGN LOAD POSSIBLE IS —

1st FL 2.0 KN/m²

2nd FL 2.5 KN/m²

RECOMMEND THESE VALUES



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UNIT 26 BASEPOINT ENTERPRISE
CENTRE
ANDERSONS ROAD

Project

Bruce Castle

Job Ref.

22129

Section

1st f.l. padstones.

Sheet no./rev.

1 - 11

Calc. by

RF

Date

04/10/2023

Chk'd by

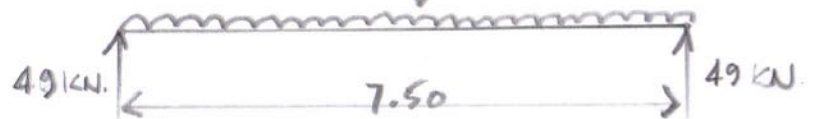
Date

App'd by

Date

1st f.l. padstones to main beams.

$$\text{from 1-36} \begin{cases} 4.10 \\ 8.13 \\ \hline 12.23 \text{ kN/m} \end{cases}$$



(note loads are unfactored for assessment to CURIA III.)

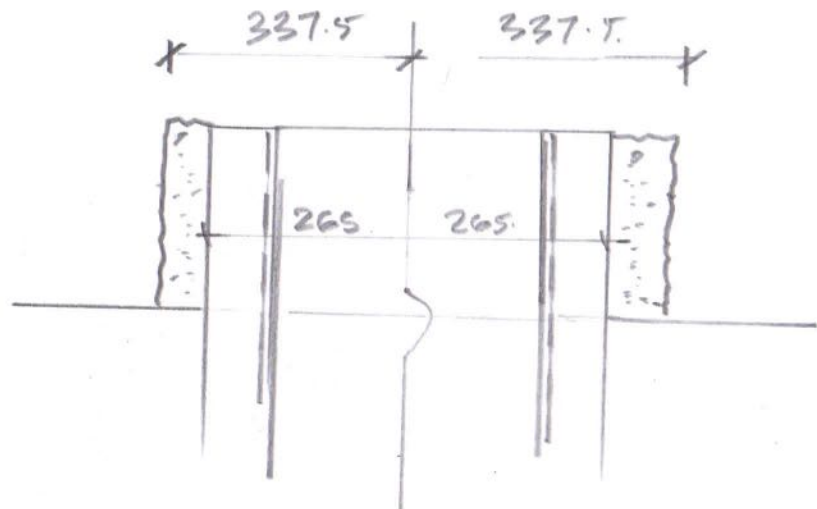
max penetration into the wall 225mm.

∴

$$\frac{49 \times 3}{225 \times B} \leq 0.42 \text{ N/mm}^2.$$

$B \geq 486 \text{ mm}$. say 3 BRICKS @ 225 ∴ 675

$$\frac{49 \times 3}{225 \times 675} = 0.322 < 0.42 \text{ N/mm}^2.$$





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UNIT 26 BASEPOINT ENTERPRISE
CENTRE
ANDERSONS ROAD

Project

Bruce Castle

Job Ref.

22129

Section

2nd f.l. PADSTONES

Sheet no./rev.

1 - 12

Calc. by

RF

Date

04/10/2023

Chk'd by

Date

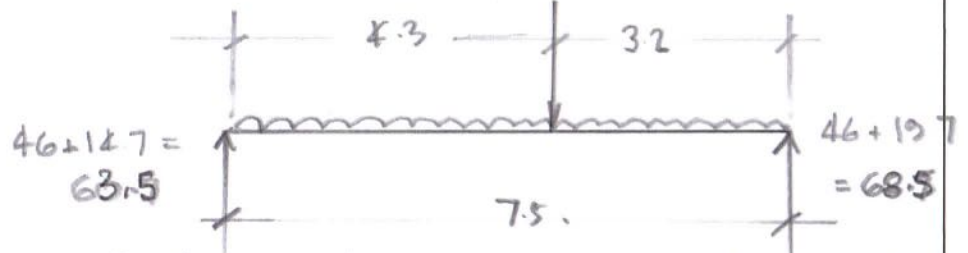
App'd by

Date

2nd f.l. padstones to main bms.

reactions from

from 1.05 { $\frac{263}{8.1} = 32.4 \text{ kN}$



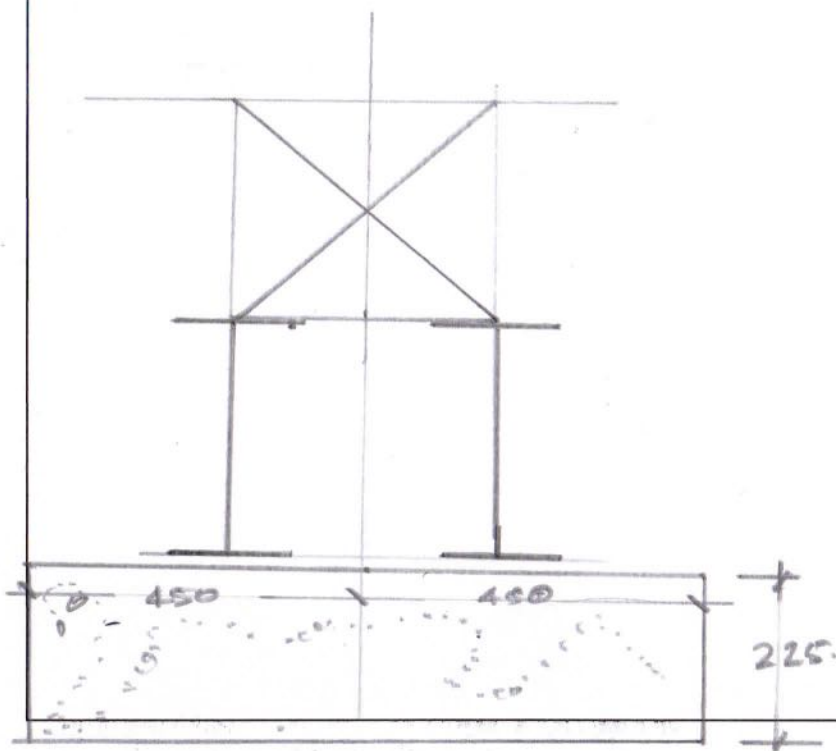
(note loads are unfactored for assessment to Clrea III)

max penetration into the wall 225mm

$$\frac{68.5 \text{ kN}}{225 \times B} \leq 0.42 \text{ N/mm}^2$$

$$B = 679 \text{ mm say } 4 \text{ Bricks @ } 225 = 900 \text{ mm.}$$

$$\frac{68.5 \text{ kN}}{225 \times 900} = 0.34 < 0.42 \text{ N/mm}^2$$





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Project Bruce Castle				Job Ref. 22129	
Section 1st FL MAIN BEAM - C/PWB.				Sheet no./rev. 1 - 13	
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260 x 90 x 35 kg/m

$M_f = 70 \text{ kNm}_f$ ✓

$V_f = 74 \text{ kN}_f$ ✓

force in plates $\frac{70}{0.26} = 269 \text{ kN}_f$

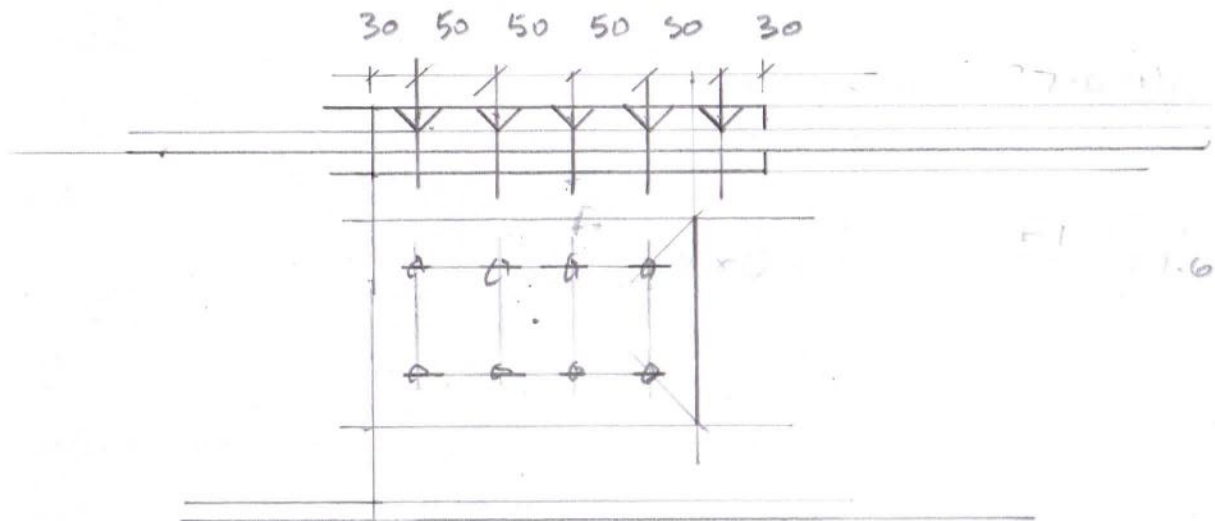
plate $(90 - 18) \times 15 \times 275 = 297 \text{ kN}_f$

add $(65 - 18) \times 15 \times 275 = 193 \text{ kN}_f$

$490 \text{ kN}_f > 269 \text{ kN}_f$

Bolt group. $\frac{269}{58.9} = 4.6 \text{ NO BOLTS. SAY } 5 \text{ NO}$

CHECK RESIDUAL FLANGE AREA $(90 - 18) \times 14 \times 260 \times 275 = 72 \text{ kNm}$



$M_f \cdot 120 \times 74 = 16.3 \text{ kNm}_f$

$(4 \times 0.8) + (4 \times 0.4) = \frac{16.3}{4.8} = 3.4 \text{ kN} \times \frac{74}{2} = 43 \text{ kN}$



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UNIT 26 BASEPOINT ENTERPRISE
CENTRE
ANDERSONS ROAD

Project		Bruce Castle		Job Ref.	
Section		2nd FL MAIN BM SPlice		22129	
Calc. by		Date		Sheet no./rev.	
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305 x 165 x 10 UB.

$M_f = 171 \text{ kN.m}_f$ from tables absolute max.

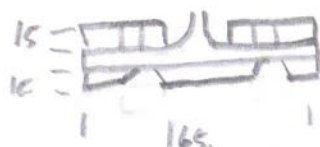
$V_f = 65 \times 1.5 = 97.5 \text{ kN}_f$

force in plates $171/0.3 = 570 \text{ kN}$.
Cap. of

plates = $275 \times (165 - 22 - 22) \times 15 = 499 \text{ kN}_f$.

add $\frac{275}{165} \times (180 - 22 - 22) \times 15 = 354 \text{ kN}_f$.

$853 \text{ kN}_f > 570 \text{ kN}_f$

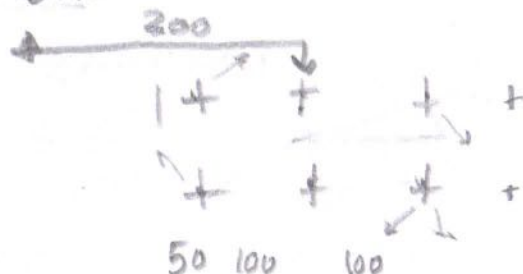


no Bolts req'd.

$$\frac{570}{92} = 6.2 \text{ use 8 Bolts}$$

$$8 \times 92 = 736 \text{ kN}_f > 570 \text{ kN}_f$$

SHEAR



$$97.5 \times 0.3 = 30 \text{ kN}_f$$

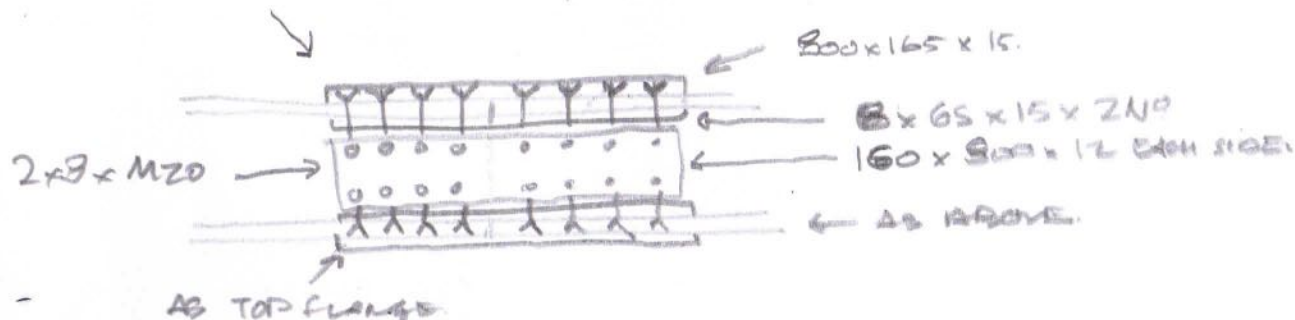
$$4 \times 0.1 \times x = 30$$

$$x = 75$$

$$85 \text{ kN}_f \text{ approx} < 92 \text{ per hole}$$

2 x 8 x M20. CSK.

make bolt group of 8 bolts
to match flange bolts



 SFK CONSULTING UNIT 26 BASEPOINT ENTERPRISE CENTRE ANDERSONS ROAD	Project Bruce Castle				Job Ref. 22129	
	Section LATERAL STABILITY OF E. WALL				Sheet no./rev. 1 - 15	
	Calc. by RF	Date 04/10/2023	Chk'd by	Date	App'd by	Date

(from 1-07)

$$\begin{array}{rcl}
 \text{HEIGHT OF GRD } \frac{1}{2} \text{ STOREY} & = & 2935 \\
 & & 565 \\
 & & \underline{445} \\
 & & 3945 \div 2 = 1.97\text{m} \\
 & & \text{say } 2.0\text{m} \\
 \\
 \text{" " 1st } \frac{1}{2} \text{ " } & = & 445 \\
 & & \underline{3350} \\
 & & 3795 \div = 1.9\text{m}
 \end{array}$$

$$H_{\text{tot}} = 2.0 + 1.9 = 3.9\text{m}.$$

$$\text{a) wind suction} \leq 1.5 \text{ kN/m}^2$$

$$\therefore 5.9 \text{ kN/m wind.}$$

$$\text{Load of wall } 604 - 47 - 184 - 20 = 353 \text{ kN.}$$

$$353 \text{ kN} \times 0.03 = 10.6 \text{ kN/m DLOAD. OF WALL.}$$

But say $\frac{1}{2}$ is taken by 2nd f.l which remains in place.

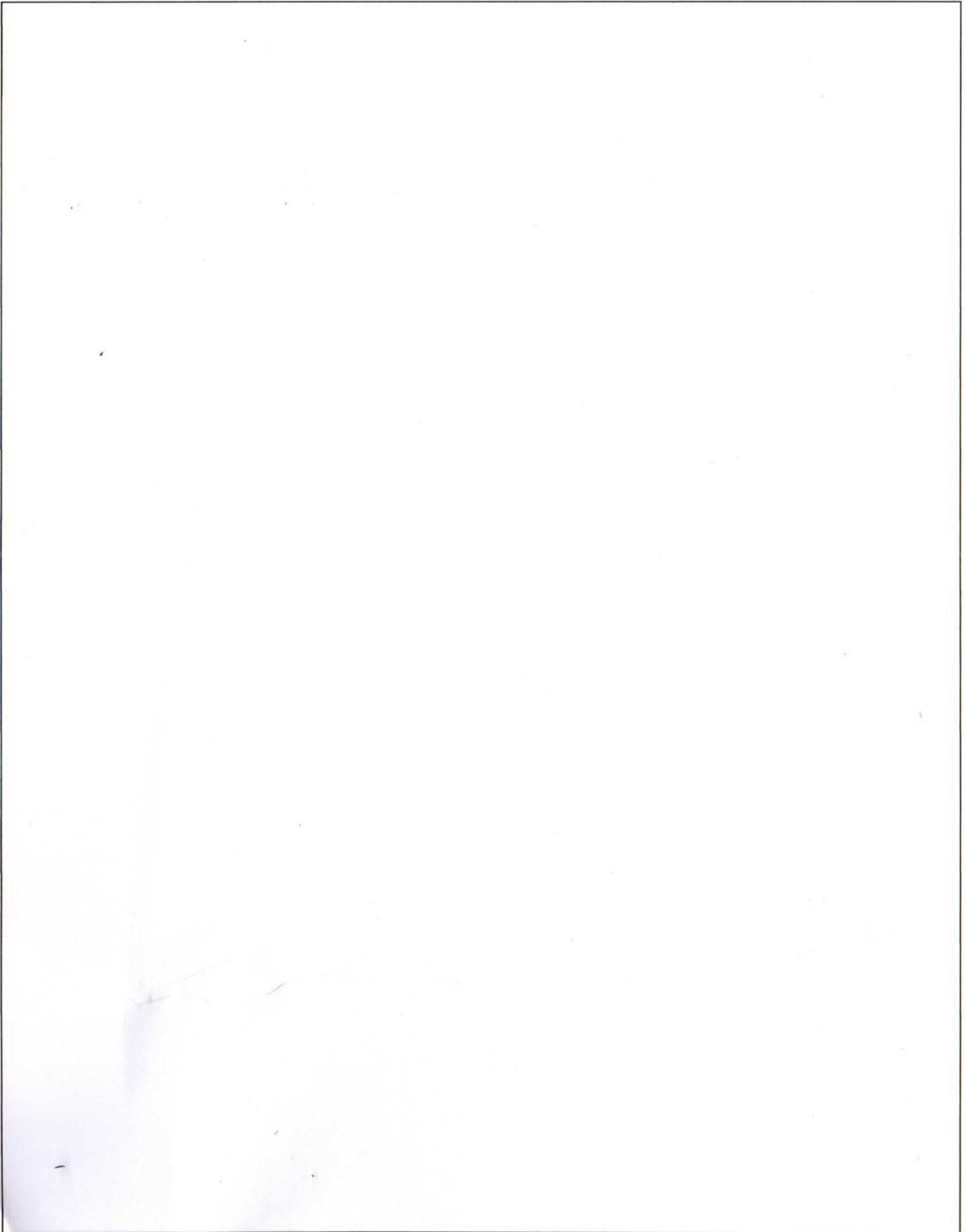
$\therefore$ If f.o.s. is sat at x2.0

$$\text{design load for temp works} = 10 \text{ kN/m unf.}$$



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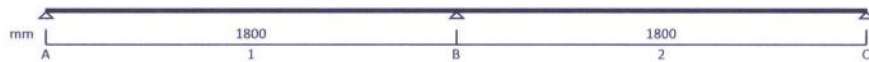
TIMBER JOIST DESIGN (BS5268)

TIMBER JOIST DESIGN (BS5268-2:2002)

Tedds calculation version 1.1.04

Joist details

Joist breadth	b = 65 mm	Joist depth	h = 140 mm
Joist spacing	s = 400 mm	Service class of timber	1
Timber strength class	C24		

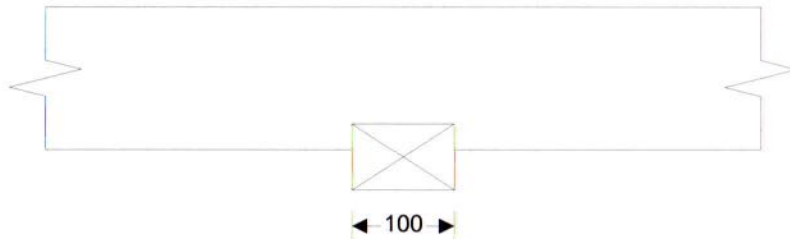
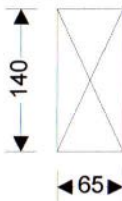


Span details

Number of spans	N_{span} = 2	Length of bearing	L_b = 100 mm
Clear length of span 1	L_{s1} = 1800 mm	Clear length of span 2	L_{s2} = 1800 mm

Notch details - Underside of joists notched at supports

Joist depth at notch **h_e = 115 mm**



Section properties (full depth section)

Second moment of area	I = 14863333 mm⁴
Section modulus	Z = 212333 mm³

Loading details

Joist self weight	F_{swt} = 0.03 kN/m
Imposed UDL(Long term)	F_{i_udi} = 2.50 kN/m²
Imposed point load (Medium)	F_{i_pt} = 2.70 kN

Consider long term loads

Design bending moment	M = 0.547 kNm
Design support reaction	R = 3.040 kN

Check bending stress

Permissible bending stress	σ_{m_adm} = 8.971 N/mm²
----------------------------	---

Check shear stress

Permissible shear stress	τ_{adm} = 0.642 N/mm²
--------------------------	---

Section properties (notched section)

Second moment of area	I_n = 8238073 mm⁴
Section modulus	Z_n = 143271 mm³

Dead load	F_{d_udi} = 0.80 kN/m²
-----------	--

Design shear force	V = 1.520 kN
Design deflection	δ = 0.814 mm

Applied bending stress	σ_{m_max} = 3.820 N/mm²
------------------------	---

PASS - Applied bending stress within permissible limits

Applied shear stress	τ_{max} = 0.305 N/mm²
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PASS - Applied shear stress within permissible limits



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UNIT 26 BASEPOINT ENTERPRISE
CENTRE
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Check bearing stress

Permissible bearing stress $\sigma_{c_adm} = 2.904 \text{ N/mm}^2$

Applied bearing stress $\sigma_{c_max} = 0.468 \text{ N/mm}^2$

PASS - Applied bearing stress within permissible limits

Check deflection

Permissible deflection $\delta_{adm} = 5.400 \text{ mm}$

Actual deflection $\delta = 0.814 \text{ mm}$

PASS - Actual deflection within permissible limits

Consider medium term loads

Design bending moment $M = 1.072 \text{ kNm}$

Design shear force $V = 3.095 \text{ kN}$

Design support reaction $R = 3.490 \text{ kN}$

Design deflection $\delta = 1.777 \text{ mm}$

Check bending stress

Permissible bending stress $\sigma_{m_adm} = 11.214 \text{ N/mm}^2$

Applied bending stress $\sigma_{m_max} = 5.050 \text{ N/mm}^2$

PASS - Applied bending stress within permissible limits

Check shear stress

Permissible shear stress $\tau_{adm} = 0.802 \text{ N/mm}^2$

Applied shear stress $\tau_{max} = 0.621 \text{ N/mm}^2$

PASS - Applied shear stress within permissible limits

Check bearing stress

Permissible bearing stress $\sigma_{c_adm} = 3.630 \text{ N/mm}^2$

Applied bearing stress $\sigma_{c_max} = 0.537 \text{ N/mm}^2$

PASS - Applied bearing stress within permissible limits

Check deflection

Permissible deflection $\delta_{adm} = 5.400 \text{ mm}$

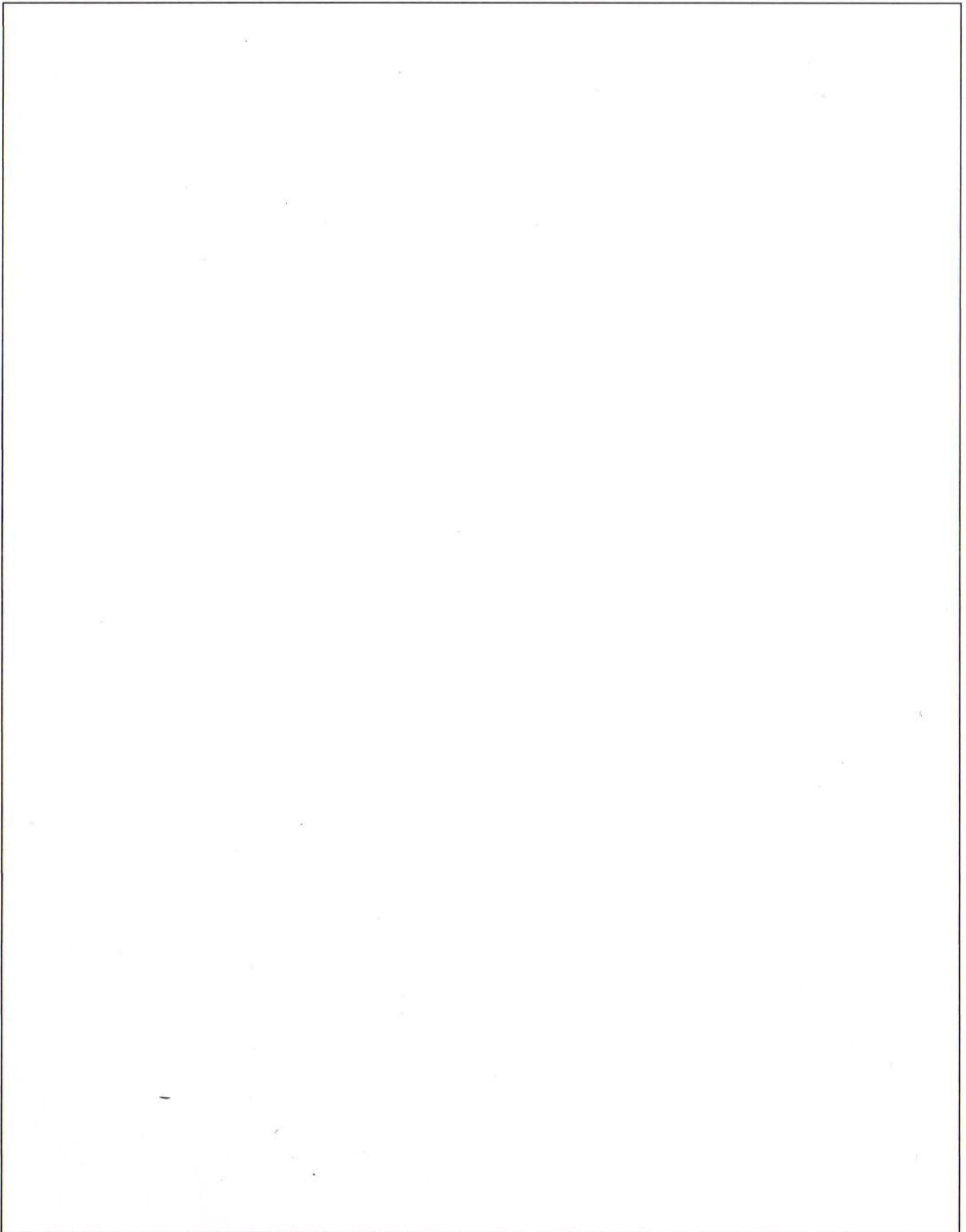
Actual deflection $\delta = 1.777 \text{ mm}$

PASS - Actual deflection within permissible limits



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RF	04/10/2023				

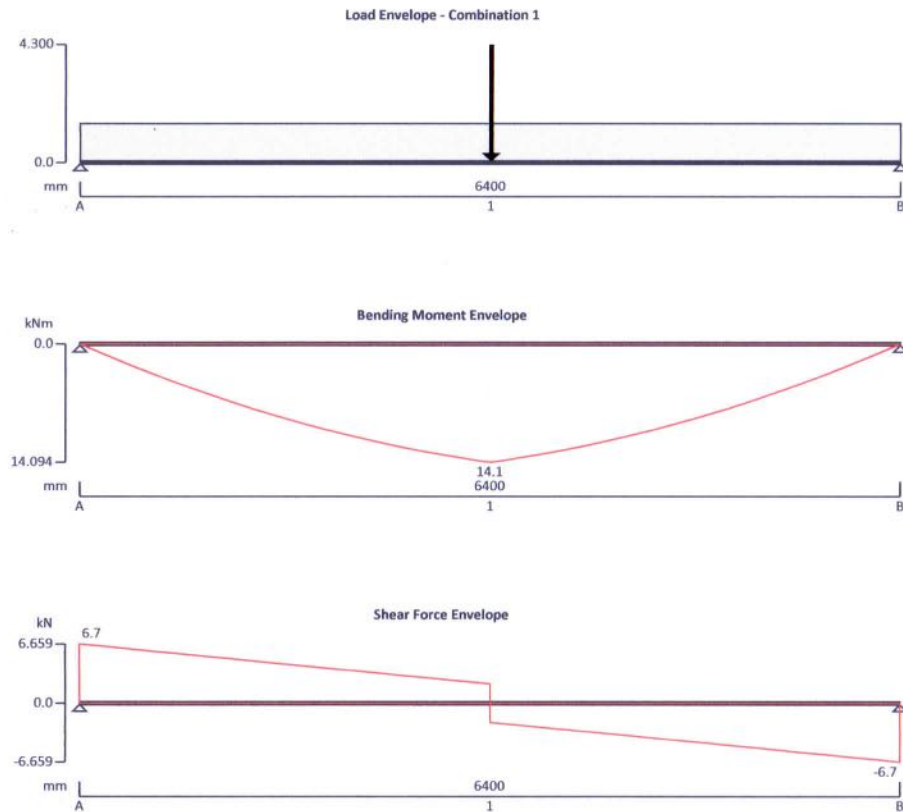


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TIMBER BEAM ANALYSIS & DESIGN (BS5268)

TIMBER BEAM ANALYSIS & DESIGN TO BS5268-2:2002

TEDDS calculation version 1.7.01



Applied loading

Beam loads

Dead self weight of beam $\times 1$
Dead full UDL 0.320 kN/m
Imposed full UDL 1.000 kN/m
Dead point load 3.200 kN at 3200 mm
Imposed point load 1.100 kN at 3200 mm

Load combinations

Load combination 1

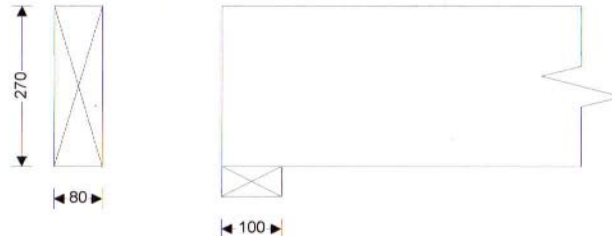
Support A	Dead $\times 1.00$
	Imposed $\times 1.00$
Span 1	Dead $\times 1.00$
	Imposed $\times 1.00$
Support B	Dead $\times 1.00$
	Imposed $\times 1.00$

Analysis results

Design moment	$M = 14.094$ kNm	Design shear	$F = 6.659$ kN
Total load on beam	$W_{tot} = 13.317$ kN		
Reactions at support A	$R_{A_max} = 6.659$ kN	$R_{A_min} = 6.659$ kN	

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Unfactored dead load reaction at support A $R_{A_Dead} = 2.909 \text{ kN}$
 Unfactored imposed load reaction at support A $R_{A_Imposed} = 3.750 \text{ kN}$
 Reactions at support B $R_{B_max} = 6.659 \text{ kN}$ $R_{B_min} = 6.659 \text{ kN}$
 Unfactored dead load reaction at support B $R_{B_Dead} = 2.909 \text{ kN}$
 Unfactored imposed load reaction at support B $R_{B_Imposed} = 3.750 \text{ kN}$



Timber section details

Breadth of section	$b = 80 \text{ mm}$	Depth of section	$h = 270 \text{ mm}$
Number of sections	$N = 1$	Breadth of beam	$b_b = 80 \text{ mm}$
Timber strength class	C24		

Member details

Service class of timber	1	Load duration	Long term
Length of span	$L_{s1} = 6400 \text{ mm}$		
Length of bearing	$L_b = 100 \text{ mm}$		

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	5.00	Actual depth-to-breadth ratio	3.38
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PASS - Lateral support is adequate

Check bearing stress

Permissible bearing stress	$\sigma_{c_adm} = 2.640 \text{ N/mm}^2$	Applied bearing stress	$\sigma_{c_a} = 0.832 \text{ N/mm}^2$
----------------------------	--	------------------------	--

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

Bending parallel to grain

Permissible bending stress	$\sigma_{m_adm} = 8.346 \text{ N/mm}^2$	Applied bending stress	$\sigma_{m_a} = 14.500 \text{ N/mm}^2$
----------------------------	--	------------------------	---

FAIL - Applied bending stress exceeds permissible bending stress

Shear parallel to grain

Permissible shear stress	$\tau_{adm} = 0.781 \text{ N/mm}^2$	Applied shear stress	$\tau_a = 0.462 \text{ N/mm}^2$
--------------------------	-------------------------------------	----------------------	---------------------------------

PASS - Applied shear stress is less than permissible shear stress

Deflection

Permissible deflection	$\delta_{adm} = 13.995 \text{ mm}$	Total deflection	$\delta_a = 59.174 \text{ mm}$
------------------------	------------------------------------	------------------	--------------------------------

FAIL - Total deflection exceeds permissible deflection

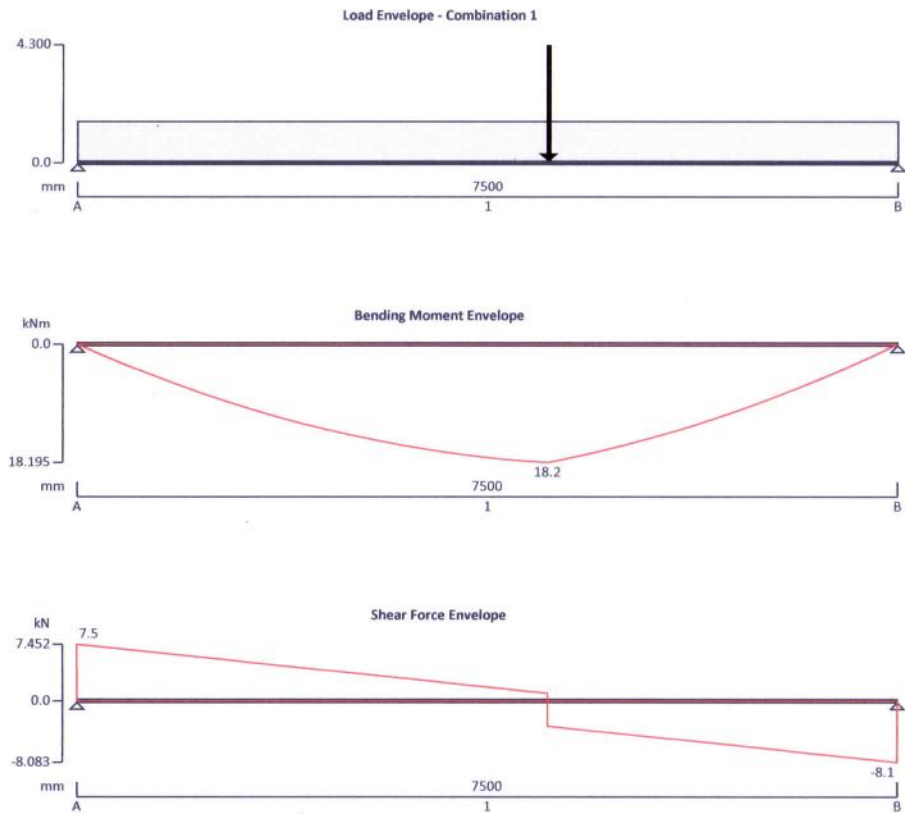
Subs in pg. 1-42 etc.

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TIMBER BEAM ANALYSIS & DESIGN (BS5268)

TIMBER BEAM ANALYSIS & DESIGN TO BS5268-2:2002

TEDDS calculation version 1.7.01



Applied loading

Beam loads

- Dead self weight of beam × 1
- Dead full UDL 0.320 kN/m ✓
- Imposed full UDL 1.000 kN/m ✓
- Dead point load 3.200 kN at 4300 mm ✓
- Imposed point load 1.100 kN at 4300 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 = 18.195 kNm	Design shear	F = 8.083 kN
Total load on beam	W _{tot} = 15.534 kN		
Reactions at support A	R _{A_max} = 7.452 kN	R _{A_min} = 7.452 kN	

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Unfactored dead load reaction at support A

$$R_{A_Dead} = 3.233 \text{ kN}$$

Unfactored imposed load reaction at support A

$$R_{A_Imposed} = 4.219 \text{ kN}$$

Reactions at support B

$$R_{B_max} = 8.083 \text{ kN}$$

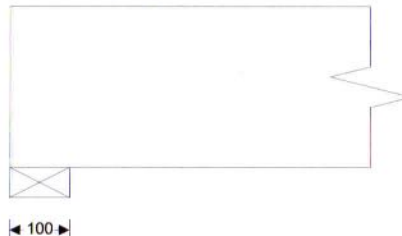
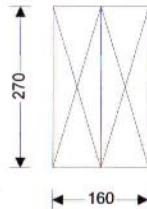
$$R_{B_min} = 8.083 \text{ kN}$$

Unfactored dead load reaction at support B

$$R_{B_Dead} = 3.702 \text{ kN}$$

Unfactored imposed load reaction at support B

$$R_{B_Imposed} = 4.381 \text{ kN}$$



Timber section details

Breadth of section

$$b = 80 \text{ mm}$$

Depth of section

$$h = 270 \text{ mm}$$

Number of sections

$$N = 2$$

Breadth of beam

$$b_b = 160 \text{ mm}$$

Timber strength class

$$C24$$

Member details

Service class of timber

$$1$$

Load duration

Long term

Length of span

$$L_{s1} = 7500 \text{ mm}$$

Length of bearing

$$L_b = 100 \text{ mm}$$

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 **5.00**

Actual depth-to-breadth ratio **1.69**

PASS - Lateral support is adequate

Check bearing stress

Permissible bearing stress

$$\sigma_{c_adm} = 2.640 \text{ N/mm}^2$$

Applied bearing stress

$$\sigma_{c_a} = 0.505 \text{ N/mm}^2$$

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

Bending parallel to grain

Permissible bending stress

$$\sigma_{m_adm} = 8.346 \text{ N/mm}^2$$

Applied bending stress

$$\sigma_{m_a} = 9.359 \text{ N/mm}^2$$

FAIL - Applied bending stress exceeds permissible bending stress

Shear parallel to grain

Permissible shear stress

$$\tau_{adm} = 0.781 \text{ N/mm}^2$$

Applied shear stress

$$\tau_a = 0.281 \text{ N/mm}^2$$

PASS - Applied shear stress is less than permissible shear stress

Deflection

Permissible deflection

$$\delta_{adm} = 13.995 \text{ mm}$$

Total deflection

$$\delta_a = 35.464 \text{ mm}$$

FAIL - Total deflection exceeds permissible deflection

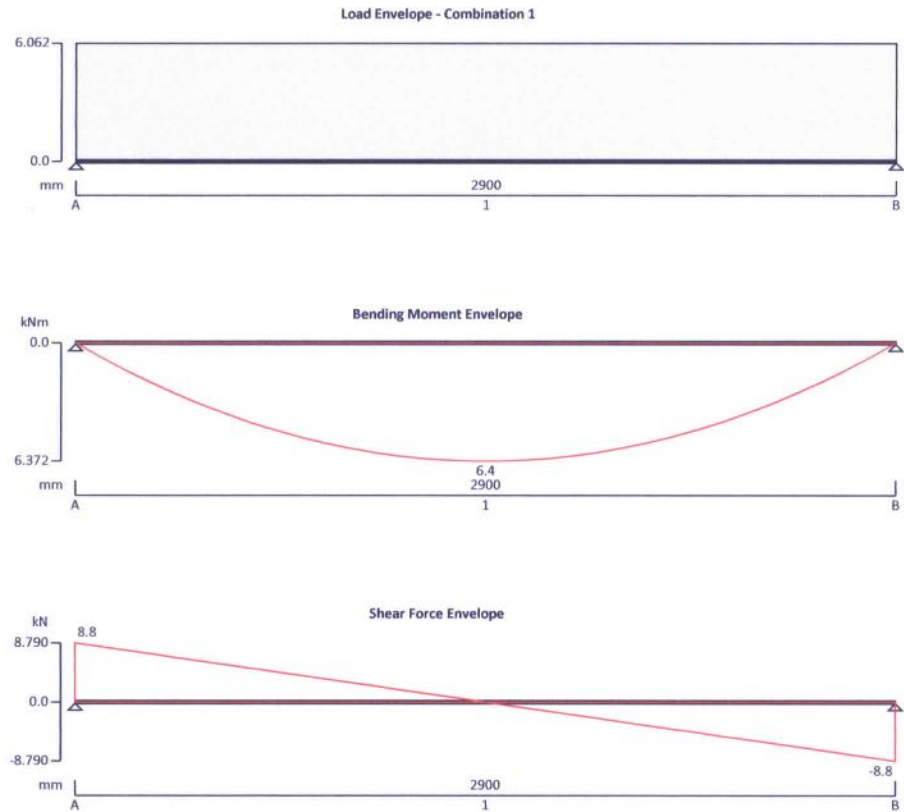
Sum in pg. 1-42 etc.

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TIMBER BEAM ANALYSIS & DESIGN (BS5268)

TIMBER BEAM ANALYSIS & DESIGN TO BS5268-2:2002

TEDDS calculation version 1.7.01



Applied loading

Beam loads

Dead self weight of beam × 1
Dead full UDL 1.500 kN/m
Imposed full UDL 4.500 kN/m

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} = 6.372 kNm	M _{min} = 0.000 kNm
Design moment	M = max(abs(M _{max}),abs(M _{min})) = 6.372 kNm	
Maximum shear	F _{max} = 8.790 kN	F _{min} = -8.790 kN
Design shear	F = max(abs(F _{max}),abs(F _{min})) = 8.790 kN	
Total load on beam	W _{tot} = 17.579 kN	

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Reactions at support A

Unfactored dead load reaction at support A

Unfactored imposed load reaction at support A

Reactions at support B

Unfactored dead load reaction at support B

Unfactored imposed load reaction at support B

$$R_{A_max} = 8.790 \text{ kN}$$

$$R_{A_Dead} = 2.265 \text{ kN}$$

$$R_{A_Imposed} = 6.525 \text{ kN}$$

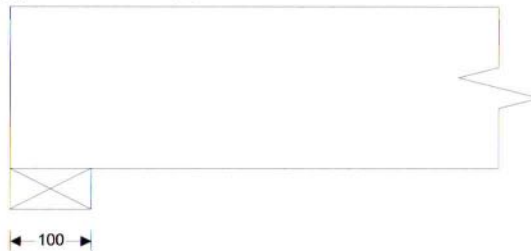
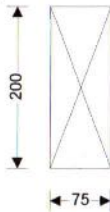
$$R_{B_max} = 8.790 \text{ kN}$$

$$R_{B_Dead} = 2.265 \text{ kN}$$

$$R_{B_Imposed} = 6.525 \text{ kN}$$

$$R_{A_min} = 8.790 \text{ kN}$$

$$R_{B_min} = 8.790 \text{ kN}$$



Timber section details

Breadth of sections

$$b = 75 \text{ mm}$$

Depth of sections

$$h = 200 \text{ mm}$$

Number of sections in member

$$N = 1$$

Overall breadth of member

$$b_b = N \times b = 75 \text{ mm}$$

Timber strength class

C24

Member details

Service class of timber

1

Load duration

Long term

Length of span

$$L_{s1} = 2900 \text{ mm}$$

Length of bearing

$$L_b = 100 \text{ mm}$$

Section properties

Cross sectional area of member

$$A = N \times b \times h = 15000 \text{ mm}^2$$

Section modulus

$$Z_x = N \times b \times h^2 / 6 = 500000 \text{ mm}^3$$

$$Z_y = h \times (N \times b)^2 / 6 = 187500 \text{ mm}^3$$

Second moment of area

$$I_x = N \times b \times h^3 / 12 = 50000000 \text{ mm}^4$$

$$I_y = h \times (N \times b)^3 / 12 = 7031250 \text{ mm}^4$$

Radius of gyration

$$i_x = \sqrt{I_x / A} = 57.7 \text{ mm}$$

$$i_y = \sqrt{I_y / A} = 21.7 \text{ mm}$$

Modification factors

Duration of loading - Table 17

$$K_3 = 1.00$$

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.2.9

$$K_8 = 1.00$$

Lateral support - cl.2.10.8

Ends held in position and members held in line, as by direct connection of sheathing, deck or joists

Permissible depth-to-breadth ratio - Table 19

$$5.00$$

Actual depth-to-breadth ratio

$$h / (N \times b) = 2.67$$

PASS - Lateral support is adequate

Compression perpendicular to grain

Permissible bearing stress (no wane)

$$\sigma_{c_adm} = \sigma_{cp1} \times K_3 \times K_4 \times K_8 = 2.400 \text{ N/mm}^2$$

Applied bearing stress

$$\sigma_{c_a} = R_{A_max} / (N \times b \times L_b) = 1.172 \text{ N/mm}^2$$

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$$\sigma_{c_a} / \sigma_{c_adm} = 0.488$$

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

Bending parallel to grain

Permissible bending stress

$$\sigma_{m_adm} = \sigma_m \times K_3 \times K_7 \times K_8 = 7.842 \text{ N/mm}^2$$

Applied bending stress

$$\sigma_{m_a} = M / Z_x = 12.745 \text{ N/mm}^2 \quad 6.4$$

$$\sigma_{m_a} / \sigma_{m_adm} = 1.625 \quad 0.82$$

FAIL - Applied bending stress exceeds permissible bending stress

Shear parallel to grain

Permissible shear stress

$$\tau_{adm} = \tau \times K_3 \times K_8 = 0.710 \text{ N/mm}^2$$

Applied shear stress

$$\tau_a = 3 \times F / (2 \times A) = 0.879 \text{ N/mm}^2 \quad (440)$$

$$\tau_a / \tau_{adm} = 1.238 \quad 0.62$$

FAIL - Applied shear stress exceeds permissible shear stress

Deflection

Modulus of elasticity for deflection

$$E = E_{min} = 7200 \text{ N/mm}^2$$

Permissible deflection

$$\delta_{adm} = \min(0.551 \text{ in}, 0.003 \times L_{s1}) = 8.700 \text{ mm}$$

Bending deflection

$$\delta_{b_s1} = 15.507 \text{ mm} \quad 7.75$$

Shear deflection

$$\delta_{v_s1} = 1.133 \text{ mm} \quad 1.13$$

Total deflection

$$\delta_a = \delta_{b_s1} + \delta_{v_s1} = 16.640 \text{ mm}$$

$$\delta_a / \delta_{adm} = 1.913$$

FAIL - Total deflection exceeds permissible deflection

2nd fl

EACH OF THESE RESULTS AT 2ND FL MAY BE HALVED AS SECONDARIES ARE INSERTED MIDWAY BETWEEN EXISTING TO TAKE $\frac{1}{2}$ THE LOAD OF THEM.

1st fl

NEW STEEL BEAMS BEING APPLIED.

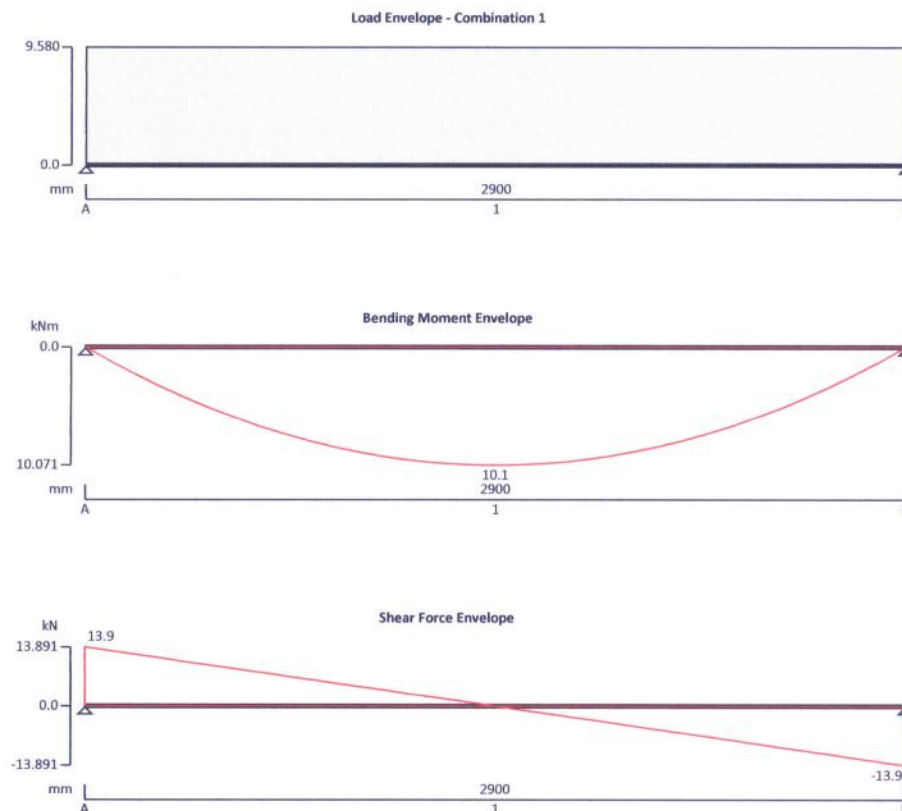
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STEEL BEAM ANALYSIS & DESIGN (BS5950)

STEEL BEAM ANALYSIS & DESIGN (BS5950)

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

TEDDS calculation version 3.0.07



Support conditions

Support A

Vertically restrained

Rotationally free

Support B

Vertically restrained

Rotationally free

Applied loading

Beam loads

Dead self weight of beam $\times 1$

Dead full UDL 1.5 kN/m

Imposed full UDL 4.5 kN/m

Load combinations

Load combination 1

Support A

Dead $\times 1.40$

Imposed $\times 1.60$

Dead $\times 1.40$

Imposed $\times 1.60$

Support B

Dead $\times 1.40$

Imposed $\times 1.60$

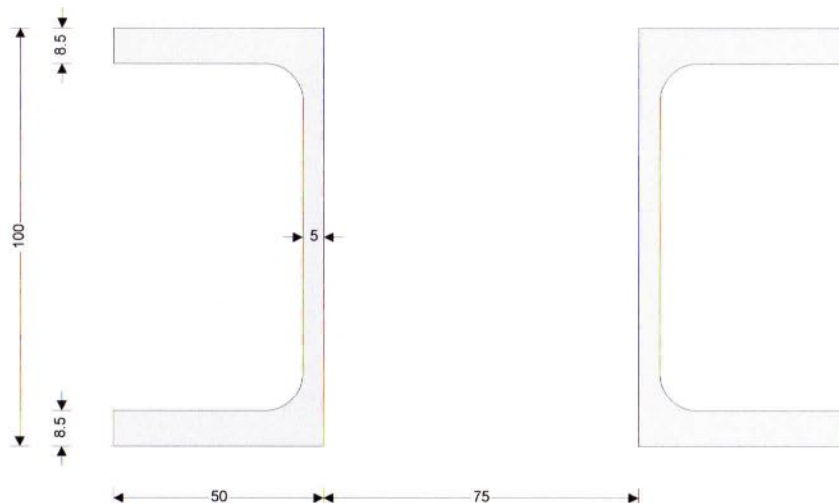
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Analysis results

Maximum moment	$M_{\max} = 10.1 \text{ kNm}$	$M_{\min} = 0 \text{ kNm}$
Maximum shear	$V_{\max} = 13.9 \text{ kN}$	$V_{\min} = -13.9 \text{ kN}$
Deflection	$\delta_{\max} = 4.9 \text{ mm}$	$\delta_{\min} = 0 \text{ mm}$
Maximum reaction at support A	$R_{A_{\max}} = 13.9 \text{ kN}$	$R_{A_{\min}} = 13.9 \text{ kN}$
Unfactored dead load reaction at support A	$R_{A_{\text{Dead}}} = 2.5 \text{ kN}$	
Unfactored imposed load reaction at support A	$R_{A_{\text{Imposed}}} = 6.5 \text{ kN}$	
Maximum reaction at support B	$R_{B_{\max}} = 13.9 \text{ kN}$	$R_{B_{\min}} = 13.9 \text{ kN}$
Unfactored dead load reaction at support B	$R_{B_{\text{Dead}}} = 2.5 \text{ kN}$	
Unfactored imposed load reaction at support B	$R_{B_{\text{Imposed}}} = 6.5 \text{ kN}$	

Section details

Section type	2 x PFC 100x50x10 (BS4-1)
Steel grade	S275
From table 9: Design strength p_y	
Thickness of element	$\max(T, t) = 8.5 \text{ mm}$
Design strength	$p_y = 275 \text{ N/mm}^2$
Modulus of elasticity	$E = 205000 \text{ N/mm}^2$



Lateral restraint

Span 1 has lateral restraint at supports only

Effective length factors

Effective length factor in major axis	$K_x = 1.00$
Effective length factor in minor axis	$K_y = 1.00$
Effective length factor for lateral-torsional buckling	$K_{LT,A} = 1.00$
	$K_{LT,B} = 1.00$

Classification of cross sections - Section 3.5

$$\varepsilon = \sqrt{[275 \text{ N/mm}^2 / p_y]} = 1.00$$

Internal compression parts - Table 11

Depth of section	$d = 65 \text{ mm}$	
	$d / t = 13.0 \times \varepsilon \leq 80 \times \varepsilon$	Class 1 plastic

Outstand flanges - Table 11

Width of section	$b = B = 50 \text{ mm}$
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$$b / T = 5.9 \times \varepsilon \leq 9 \times \varepsilon$$

Class 1 plastic

Section is class 1 plastic

Shear capacity - Section 4.2.3

Design shear force

$$F_v = \max(\text{abs}(V_{\max}), \text{abs}(V_{\min})) = 13.9 \text{ kN}$$

$$d / t < 70 \times \varepsilon$$

Web does not need to be checked for shear buckling

Shear area

$$A_v = t \times D = 500 \text{ mm}^2$$

Design shear resistance

$$P_v = 0.6 \times N \times p_y \times A_v = 165 \text{ kN}$$

PASS - Design shear resistance exceeds design shear force

Moment capacity - Section 4.2.5

Design bending moment

$$M = \max(\text{abs}(M_{s1_max}), \text{abs}(M_{s1_min})) = 10.1 \text{ kNm}$$

Moment capacity low shear - cl.4.2.5.2

$$M_c = N \times \min(p_y \times S_{xx}, 1.2 \times p_y \times Z_{xx}) = 26.9 \text{ kNm}$$

Effective length for lateral-torsional buckling - Section 4.3.5

Effective length for lateral torsional buckling

$$L_E = 1.0 \times L_{s1} = 2900 \text{ mm}$$

Slenderness ratio

$$\lambda = L_E / r_{yy} = 183.984$$

Equivalent slenderness - Section 4.3.6.7

Buckling parameter

$$u = 0.939$$

Torsional index

$$x = 10.126$$

Slenderness factor

$$v = 1 / [1 + 0.05 \times (\lambda / x)^2]^{0.25} = 0.489$$

Ratio - cl.4.3.6.9

$$\beta_w = 1.000$$

Equivalent slenderness - cl.4.3.6.7

$$\lambda_{LT} = u \times v \times \lambda \times \sqrt{\beta_w} = 84.453$$

Limiting slenderness - Annex B.2.2

$$\lambda_{L0} = 0.4 \times (\pi^2 \times E / p_y)^{0.5} = 34.310$$

$\lambda_{LT} > \lambda_{L0}$ - Allowance should be made for lateral-torsional buckling

Bending strength - Section 4.3.6.5

Robertson constant

$$\alpha_{LT} = 7.0$$

Perry factor

$$\eta_{LT} = \max(\alpha_{LT} \times (\lambda_{LT} - \lambda_{L0}) / 1000, 0) = 0.351$$

Euler stress

$$p_E = \pi^2 \times E / \lambda_{LT}^2 = 283.7 \text{ N/mm}^2$$

$$\phi_{LT} = (p_y + (\eta_{LT} + 1) \times p_E) / 2 = 329.1 \text{ N/mm}^2$$

Bending strength - Annex B.2.1

$$p_b = p_E \times p_y / (\phi_{LT} + (\phi_{LT}^2 - p_E \times p_y)^{0.5}) = 155 \text{ N/mm}^2$$

Equivalent uniform moment factor - Section 4.3.6.6

Moment at quarter point of segment

$$M_2 = 7.6 \text{ kNm}$$

Moment at centre-line of segment

$$M_3 = 10.1 \text{ kNm}$$

Moment at three quarter point of segment

$$M_4 = 7.6 \text{ kNm}$$

Maximum moment in segment

$$M_{abs} = 10.1 \text{ kNm}$$

Maximum moment governing buckling resistance

$$M_{LT} = M_{abs} = 10.1 \text{ kNm}$$

Equivalent uniform moment factor for lateral-torsional buckling

$$m_{LT} = \max(0.2 + (0.15 \times M_2 + 0.5 \times M_3 + 0.15 \times M_4) / M_{abs}, 0.44) = 0.925$$

Buckling resistance moment - Section 4.3.6.4

Buckling resistance moment

$$M_b = N \times p_b \times S_{xx} = 15.2 \text{ kNm}$$

$$M_b / m_{LT} = 16.4 \text{ kNm}$$

PASS - Buckling resistance moment exceeds design bending moment

Check vertical deflection - Section 2.5.2

Consider deflection due to imposed loads

Limiting deflection

$$\delta_{lim} = L_{s1} / 360 = 8.056 \text{ mm}$$

Maximum deflection span 1

$$\delta = \max(\text{abs}(\delta_{\max}), \text{abs}(\delta_{\min})) = 4.867 \text{ mm}$$



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PASS - Maximum deflection does not exceed deflection limit

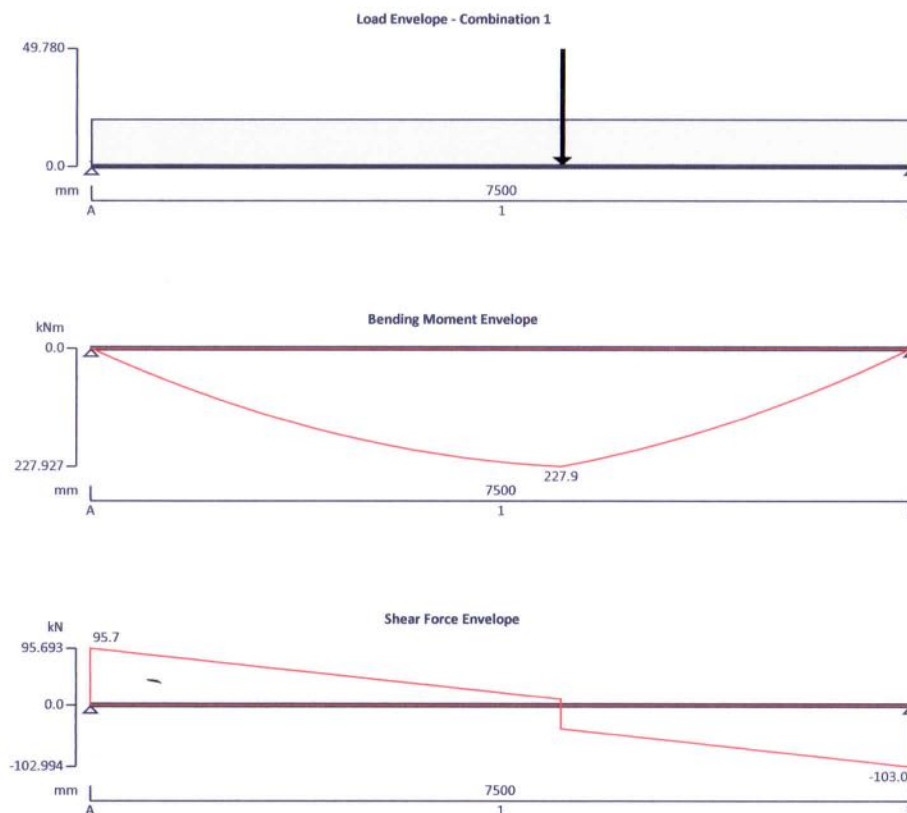
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STEEL BEAM ANALYSIS & DESIGN (BS5950)

STEEL BEAM ANALYSIS & DESIGN (BS5950)

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

TEDDS calculation version 3.0.07



Support conditions

Support A

Vertically restrained

Rotationally free

Support B

Vertically restrained

Rotationally free

Applied loading

Beam loads

Dead self weight of beam $\times 1$

Dead full UDL 4.1 kN/m

Imposed full UDL 8.13 kN/m

Dead point load 26.3 kN at 4300 mm

Imposed point load 8.1 kN at 4300 mm

Load combinations

Load combination 1

Support A

Dead $\times 1.40$

Imposed $\times 1.60$

Dead $\times 1.40$

Imposed $\times 1.60$

Support B

Dead $\times 1.40$

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Imposed $\times 1.60$

Analysis results

Maximum moment

$$M_{\max} = 227.9 \text{ kNm}$$

$$M_{\min} = 0 \text{ kNm}$$

Maximum moment span 1 segment 1

$$M_{s1_seg1_max} = 177.2 \text{ kNm}$$

$$M_{s1_seg1_min} = 0 \text{ kNm}$$

Maximum moment span 1 segment 2

$$M_{s1_seg2_max} = 227.9 \text{ kNm}$$

$$M_{s1_seg2_min} = 0 \text{ kNm}$$

Maximum moment span 1 segment 3

$$M_{s1_seg3_max} = 195.4 \text{ kNm}$$

$$M_{s1_seg3_min} = 0 \text{ kNm}$$

Maximum shear

$$V_{\max} = 95.7 \text{ kN}$$

$$V_{\min} = -103 \text{ kN}$$

Maximum shear span 1 segment 1

$$V_{s1_seg1_max} = 95.7 \text{ kN}$$

$$V_{s1_seg1_min} = 0 \text{ kN}$$

Maximum shear span 1 segment 2

$$V_{s1_seg2_max} = 46.1 \text{ kN}$$

$$V_{s1_seg2_min} = -53.4 \text{ kN}$$

Maximum shear span 1 segment 3

$$V_{s1_seg3_max} = 0 \text{ kN}$$

$$V_{s1_seg3_min} = -103 \text{ kN}$$

Deflection segment 4

$$\delta_{\max} = 11.6 \text{ mm}$$

$$\delta_{\min} = 0 \text{ mm}$$

Maximum reaction at support A

$$R_{A_max} = 95.7 \text{ kN}$$

$$R_{A_min} = 95.7 \text{ kN}$$

Unfactored dead load reaction at support A

$$R_{A_Dead} = 29.6 \text{ kN}$$

Unfactored imposed load reaction at support A

$$R_{A_Imposed} = 33.9 \text{ kN}$$

Maximum reaction at support B

$$R_{B_max} = 103 \text{ kN}$$

$$R_{B_min} = 103 \text{ kN}$$

Unfactored dead load reaction at support B

$$R_{B_Dead} = 33.4 \text{ kN}$$

Unfactored imposed load reaction at support B

$$R_{B_Imposed} = 35.1 \text{ kN}$$

PADSTONES 1-12.

Section details

Section type

2 x UKB 305x165x40

Steel grade

S275

From table 9: Design strength p_y

Thickness of element

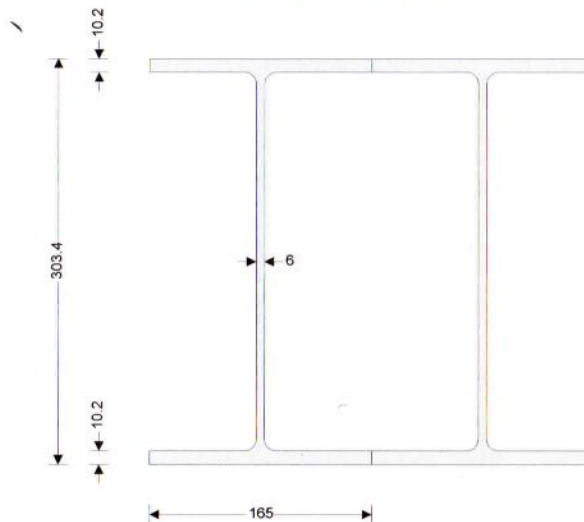
$$\max(T, t) = 10.2 \text{ mm}$$

Design strength

$$p_y = 275 \text{ N/mm}^2$$

Modulus of elasticity

$$E = 205000 \text{ N/mm}^2$$



Lateral restraint

Span 1 has lateral restraint at supports plus third points

Effective length factors

Effective length factor in major axis

$$K_x = 1.00$$

Effective length factor in minor axis

$$K_y = 1.00$$

Effective length factor for lateral-torsional buckling

$$K_{LTA} = 1.00$$

$$K_{LTB} = 1.00$$

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Classification of cross sections - Section 3.5

$$\varepsilon = \sqrt{[275 \text{ N/mm}^2 / p_y]} = 1.00$$

Internal compression parts - Table 11

Depth of section

$$d = 265.2 \text{ mm}$$

$$d / t = 44.2 \times \varepsilon \leq 80 \times \varepsilon \quad \text{Class 1 plastic}$$

Outstand flanges - Table 11

Width of section

$$b = B / 2 = 82.5 \text{ mm}$$

$$b / T = 8.1 \times \varepsilon \leq 9 \times \varepsilon \quad \text{Class 1 plastic}$$

Section is class 1 plastic

Shear capacity - Section 4.2.3

Design shear force

$$F_v = \max(\text{abs}(V_{\max}), \text{abs}(V_{\min})) = 103 \text{ kN}$$

$$d / t < 70 \times \varepsilon$$

Web does not need to be checked for shear buckling

Shear area

$$A_v = t \times D = 1820 \text{ mm}^2$$

Design shear resistance

$$P_v = 0.6 \times N \times p_y \times A_v = 600.7 \text{ kN}$$

PASS - Design shear resistance exceeds design shear force

Moment capacity at span 1 segment 2 - Section 4.2.5

Design bending moment

$$M = \max(\text{abs}(M_{s1_seg2_max}), \text{abs}(M_{s1_seg2_min})) = 227.9 \text{ kNm}$$

Moment capacity low shear - cl.4.2.5.2

$$M_c = N \times \min(p_y \times S_{xx}, 1.2 \times p_y \times Z_{xx}) = 342.7 \text{ kNm}$$

Effective length for lateral-torsional buckling - Section 4.3.5

Effective length for lateral torsional buckling

$$L_E = 1.0 \times L_{s1_seg2} = 2500 \text{ mm}$$

Slenderness ratio

$$\lambda = L_E / r_{yy} = 64.779$$

Equivalent slenderness - Section 4.3.6.7

Buckling parameter

$$u = 0.889$$

Torsional index

$$x = 30.971$$

Slenderness factor

$$v = 1 / [1 + 0.05 \times (\lambda / x)^2]^{0.25} = 0.952$$

Ratio - cl.4.3.6.9

$$\beta_W = 1.000$$

Equivalent slenderness - cl.4.3.6.7

$$\lambda_{LT} = u \times v \times \lambda \times \sqrt{[\beta_W]} = 54.801$$

Limiting slenderness - Annex B.2.2

$$\lambda_{L0} = 0.4 \times (\pi^2 \times E / p_y)^{0.5} = 34.310$$

$\lambda_{LT} > \lambda_{L0}$ - Allowance should be made for lateral-torsional buckling

Bending strength - Section 4.3.6.5

Robertson constant

$$\alpha_{LT} = 7.0$$

Perry factor

$$\eta_{LT} = \max(\alpha_{LT} \times (\lambda_{LT} - \lambda_{L0}) / 1000, 0) = 0.143$$

Euler stress

$$p_E = \pi^2 \times E / \lambda_{LT}^2 = 673.7 \text{ N/mm}^2$$

$$\phi_{LT} = (p_y + (\eta_{LT} + 1) \times p_E) / 2 = 522.7 \text{ N/mm}^2$$

Bending strength - Annex B.2.1

$$p_b = p_E \times p_y / (\phi_{LT} + (\phi_{LT}^2 - p_E \times p_y)^{0.5}) = 226.2 \text{ N/mm}^2$$

Equivalent uniform moment factor - Section 4.3.6.6

Moment at quarter point of segment

$$M_2 = 202.1 \text{ kNm}$$

Moment at centre-line of segment

$$M_3 = 219.2 \text{ kNm}$$

Moment at three quarter point of segment

$$M_4 = 224.9 \text{ kNm}$$

Maximum moment in segment

$$M_{abs} = 227.9 \text{ kNm}$$

Maximum moment governing buckling resistance

$$M_{LT} = M_{abs} = 227.9 \text{ kNm}$$

Equivalent uniform moment factor for lateral-torsional buckling

$$m_{LT} = \max(0.2 + (0.15 \times M_2 + 0.5 \times M_3 + 0.15 \times M_4) / M_{abs}, 0.44) = 0.962$$



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Buckling resistance moment - Section 4.3.6.4

Buckling resistance moment

$$M_b = N \times p_b \times S_{xx} = 281.8 \text{ kNm}$$

$$M_b / m_{LT} = 293 \text{ kNm}$$

PASS - Buckling resistance moment exceeds design bending moment

Check vertical deflection - Section 2.5.2

Consider deflection due to imposed loads

Limiting deflection

$$\delta_{lim} = L_{s1} / 360 = 20.833 \text{ mm}$$

Maximum deflection span 1

$$\delta = \max(\text{abs}(\delta_{max}), \text{abs}(\delta_{min})) = 11.589 \text{ mm}$$

PASS - Maximum deflection does not exceed deflection limit

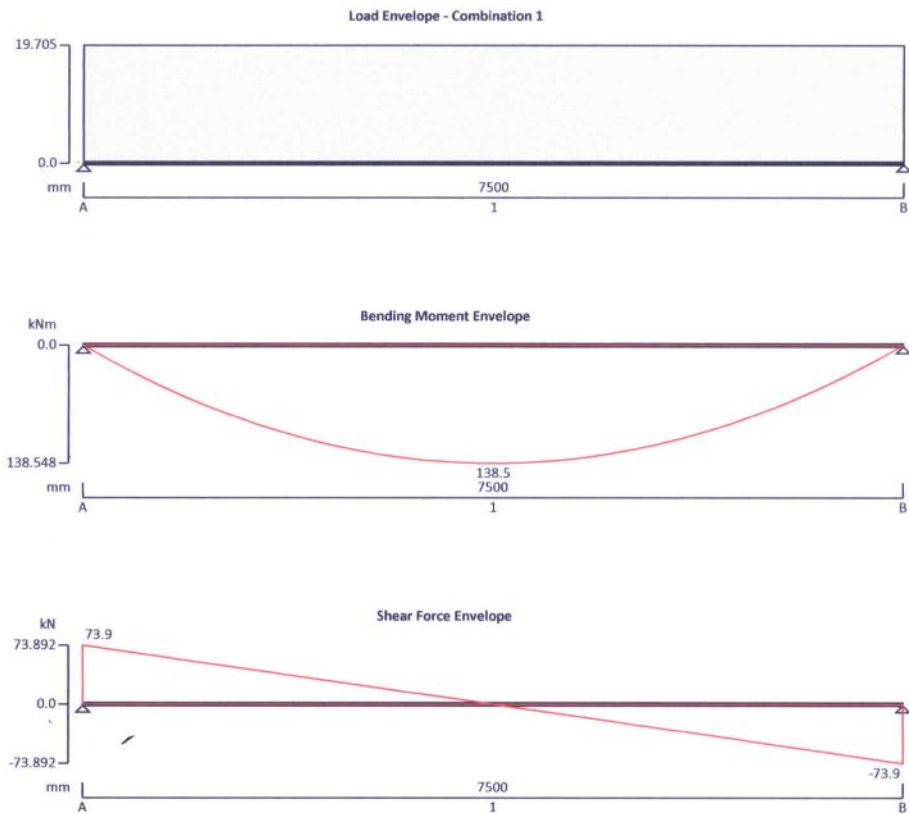
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STEEL BEAM ANALYSIS & DESIGN (BS5950)

STEEL BEAM ANALYSIS & DESIGN (BS5950)

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

TEDDS calculation version 3.0.07



Support conditions

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

Applied loading

Beam loads	Dead self weight of beam × 1
	Dead full UDL 4.1 kN/m
	Imposed full UDL 8.13 kN/m

Load combinations

Load combination 1	Support A	Dead × 1.40
		Imposed × 1.60
	Support B	Dead × 1.40
		Imposed × 1.60

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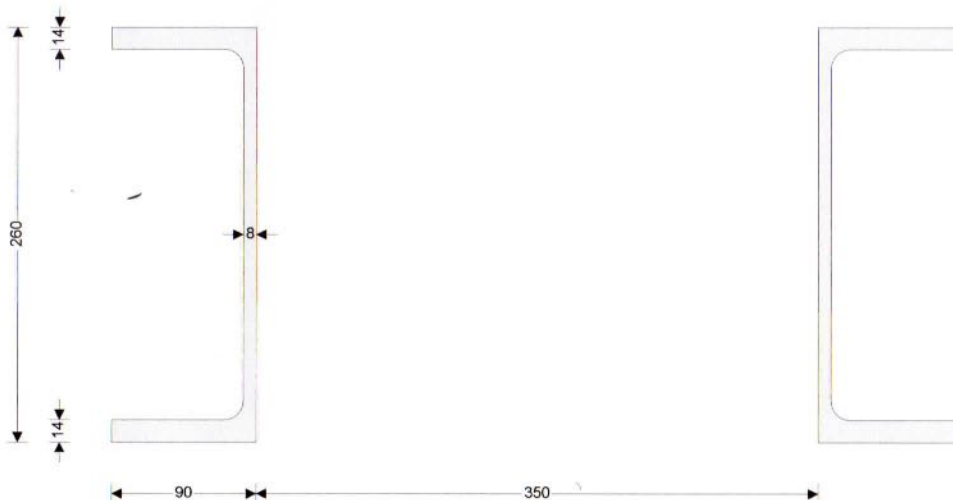
Analysis results

Maximum moment	$M_{max} = 138.5 \text{ kNm}$	$M_{min} = 0 \text{ kNm}$
Maximum moment span 1 segment 1	$M_{s1_seg1_max} = 123.2 \text{ kNm}$	$M_{s1_seg1_min} = 0 \text{ kNm}$
Maximum moment span 1 segment 2	$M_{s1_seg2_max} = 138.5 \text{ kNm}$	$M_{s1_seg2_min} = 0 \text{ kNm}$
Maximum moment span 1 segment 3	$M_{s1_seg3_max} = 123.2 \text{ kNm}$	$M_{s1_seg3_min} = 0 \text{ kNm}$
Maximum shear	$V_{max} = 73.9 \text{ kN}$	$V_{min} = -73.9 \text{ kN}$
Maximum shear span 1 segment 1	$V_{s1_seg1_max} = 73.9 \text{ kN}$	$V_{s1_seg1_min} = 0 \text{ kN}$
Maximum shear span 1 segment 2	$V_{s1_seg2_max} = 24.6 \text{ kN}$	$V_{s1_seg2_min} = -24.6 \text{ kN}$
Maximum shear span 1 segment 3	$V_{s1_seg3_max} = 0 \text{ kN}$	$V_{s1_seg3_min} = -73.9 \text{ kN}$
Deflection segment 4	$\delta_{max} = 17.3 \text{ mm}$	$\delta_{min} = 0 \text{ mm}$
Maximum reaction at support A	$R_{A_max} = 73.9 \text{ kN}$	$R_{A_min} = 73.9 \text{ kN}$
Unfactored dead load reaction at support A	$R_{A_Dead} = 17.9 \text{ kN}$	
Unfactored imposed load reaction at support A	$R_{A_Imposed} = 30.5 \text{ kN}$	
Maximum reaction at support B	$R_{B_max} = 73.9 \text{ kN}$	$R_{B_min} = 73.9 \text{ kN}$
Unfactored dead load reaction at support B	$R_{B_Dead} = 17.9 \text{ kN}$	
Unfactored imposed load reaction at support B	$R_{B_Imposed} = 30.5 \text{ kN}$	

PADSTONE 1-11.

Section details

Section type	2 x PFC 260x90x35 (BS4-1)
Steel grade	S275
From table 9: Design strength p_y	
Thickness of element	$\max(T, t) = 14.0 \text{ mm}$
Design strength	$p_y = 275 \text{ N/mm}^2$
Modulus of elasticity	$E = 205000 \text{ N/mm}^2$



Lateral restraint

Span 1 has lateral restraint at supports plus third points

Effective length factors

Effective length factor in major axis	$K_x = 1.00$
Effective length factor in minor axis	$K_y = 1.00$
Effective length factor for lateral-torsional buckling	$K_{LTA} = 1.00$
	$K_{LTB} = 1.00$

Classification of cross sections - Section 3.5

$$\varepsilon = \sqrt{[275 \text{ N/mm}^2 / p_y]} = 1.00$$

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Internal compression parts - Table 11

Depth of section $d = 208 \text{ mm}$
 $d / t = 26.0 \times \varepsilon \leq 80 \times \varepsilon$ Class 1 plastic

Outstand flanges - Table 11

Width of section $b = B = 90 \text{ mm}$
 $b / T = 6.4 \times \varepsilon \leq 9 \times \varepsilon$ Class 1 plastic
Section is class 1 plastic

Shear capacity - Section 4.2.3

Design shear force $F_v = \max(\text{abs}(V_{\max}), \text{abs}(V_{\min})) = 73.9 \text{ kN}$
 $d / t < 70 \times \varepsilon$
Web does not need to be checked for shear buckling
Shear area $A_v = t \times D = 2080 \text{ mm}^2$
Design shear resistance $P_v = 0.6 \times N \times p_y \times A_v = 686.4 \text{ kN}$
PASS - Design shear resistance exceeds design shear force

Moment capacity at span 1 segment 2 - Section 4.2.5

Design bending moment $M = \max(\text{abs}(M_{s1_seg2_max}), \text{abs}(M_{s1_seg2_min})) = 138.5 \text{ kNm}$
Moment capacity low shear - cl.4.2.5.2 $M_c = N \times \min(p_y \times S_{xx}, 1.2 \times p_y \times Z_{xx}) = 233.5 \text{ kNm}$

Effective length for lateral-torsional buckling - Section 4.3.5

Effective length for lateral torsional buckling $L_E = 1.0 \times L_{s1_seg2} = 2500 \text{ mm}$
Slenderness ratio $\lambda = L_E / r_{yy} = 88.704$

Equivalent slenderness - Section 4.3.6.7

Buckling parameter $u = 0.942$
Torsional index $x = 17.331$
Slenderness factor $v = 1 / [1 + 0.05 \times (\lambda / x)^2]^{0.25} = 0.811$
Ratio - cl.4.3.6.9 $\beta_W = 1.000$
Equivalent slenderness - cl.4.3.6.7 $\lambda_{LT} = u \times v \times \lambda \times \sqrt{[\beta_W]} = 67.776$
Limiting slenderness - Annex B.2.2 $\lambda_{LO} = 0.4 \times (\pi^2 \times E / p_y)^{0.5} = 34.310$
 $\lambda_{LT} > \lambda_{LO}$ - Allowance should be made for lateral-torsional buckling

Bending strength - Section 4.3.6.5

Robertson constant $\alpha_{LT} = 7.0$
Perry factor $\eta_{LT} = \max(\alpha_{LT} \times (\lambda_{LT} - \lambda_{LO}) / 1000, 0) = 0.234$
Euler stress $p_E = \pi^2 \times E / \lambda_{LT}^2 = 440.5 \text{ N/mm}^2$
 $\phi_{LT} = (p_y + (\eta_{LT} + 1) \times p_E) / 2 = 409.3 \text{ N/mm}^2$
Bending strength - Annex B.2.1 $p_b = p_E \times p_y / (\phi_{LT} + (\phi_{LT}^2 - p_E \times p_y)^{0.5}) = 193.9 \text{ N/mm}^2$

Equivalent uniform moment factor - Section 4.3.6.6

Moment at quarter point of segment $M_2 = 134.7 \text{ kNm}$
Moment at centre-line of segment $M_3 = 138.5 \text{ kNm}$
Moment at three quarter point of segment $M_4 = 134.7 \text{ kNm}$
Maximum moment in segment $M_{abs} = 138.5 \text{ kNm}$
Maximum moment governing buckling resistance $M_{LT} = M_{abs} = 138.5 \text{ kNm}$
Equivalent uniform moment factor for lateral-torsional buckling
 $m_{LT} = \max(0.2 + (0.15 \times M_2 + 0.5 \times M_3 + 0.15 \times M_4) / M_{abs}, 0.44) = 0.992$

Buckling resistance moment - Section 4.3.6.4

Buckling resistance moment $M_b = N \times p_b \times S_{xx} = 164.6 \text{ kNm}$

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$$M_b / m_{LT} = 166 \text{ kNm}$$

PASS - Buckling resistance moment exceeds design bending moment

Check vertical deflection - Section 2.5.2

Consider deflection due to imposed loads

Limiting deflection

$$\delta_{lim} = L_{s1} / 360 = 20.833 \text{ mm}$$

Maximum deflection span 1

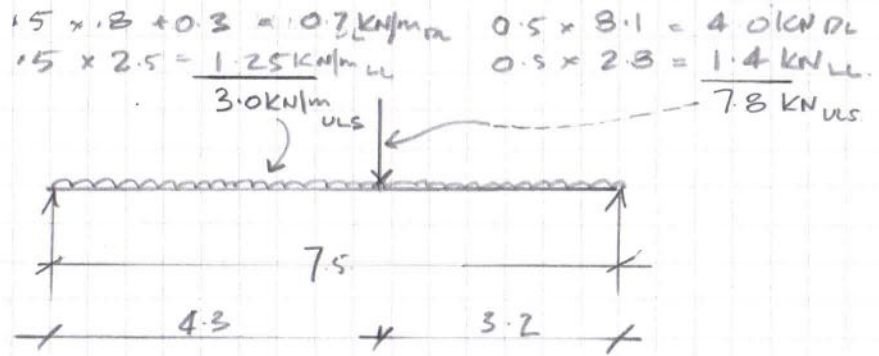
$$\delta = \max(\text{abs}(\delta_{max}), \text{abs}(\delta_{min})) = 17.277 \text{ mm}$$

PASS - Maximum deflection does not exceed deflection limit

restrained at at least $\frac{1}{3}$ points.

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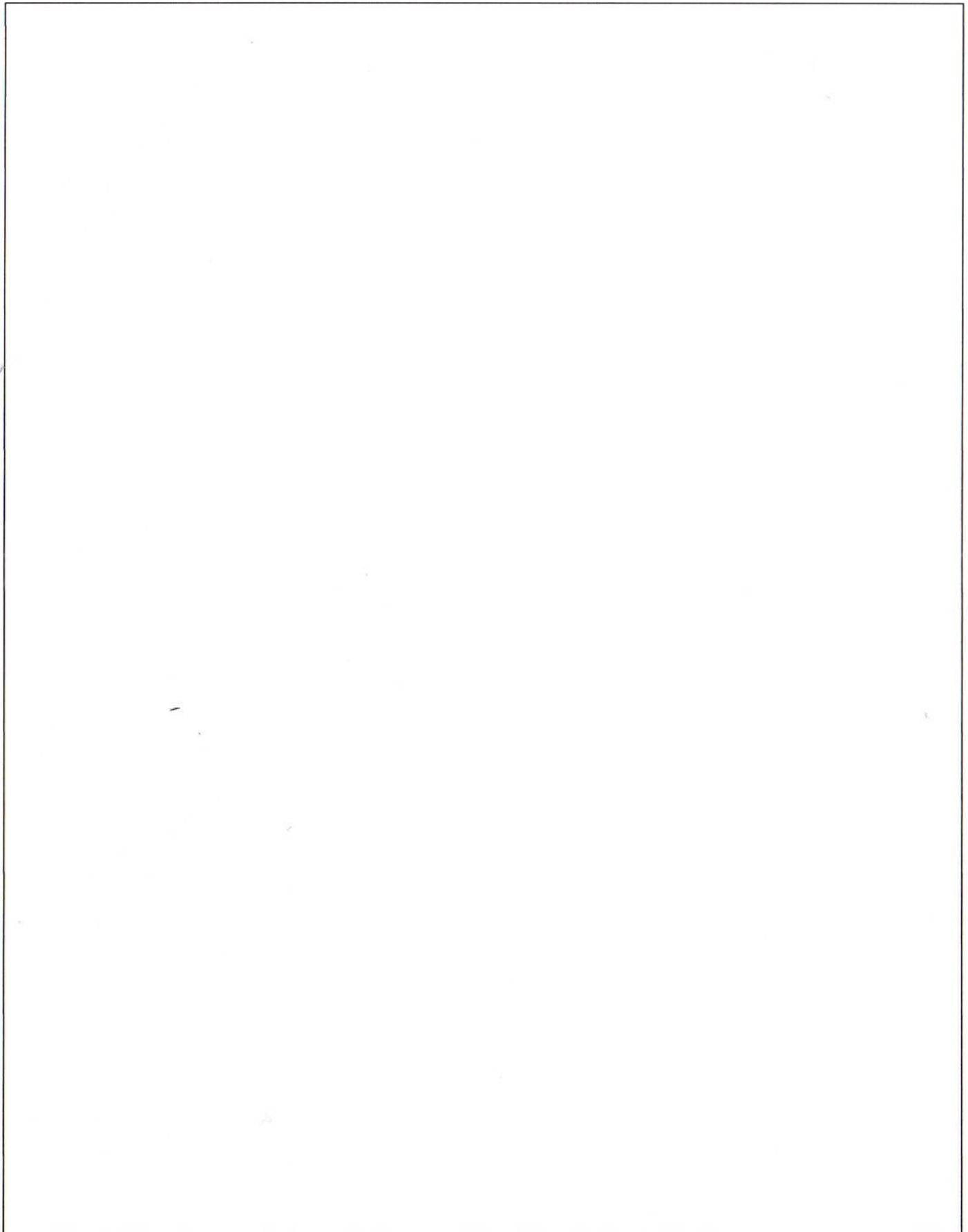
SUBS 1-04 (3)





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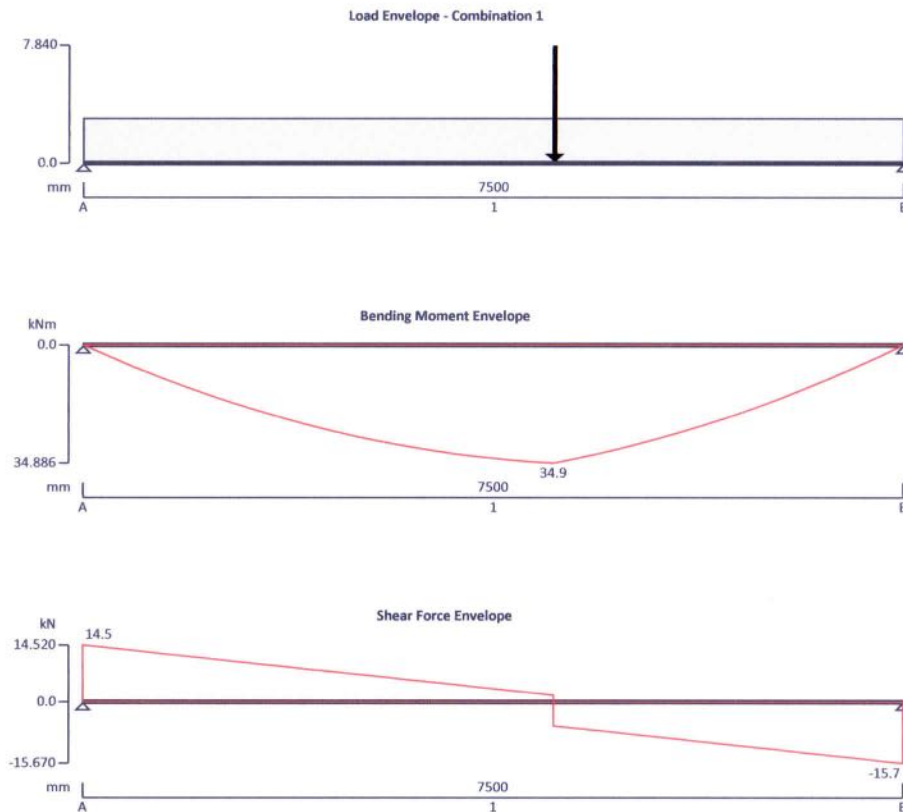
 SFK CONSULTING UNIT 26 BASEPOINT ENTERPRISE CENTRE ANDERSONS ROAD	Project Bruce Castle				Job Ref. 22129	
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STEEL BEAM ANALYSIS & DESIGN (BS5950)

STEEL BEAM ANALYSIS & DESIGN (BS5950)

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

TEDDS calculation version 3.0.07



Support conditions

Support A

Vertically restrained

Rotationally free

Support B

Vertically restrained

Rotationally free

Applied loading

Beam loads

Dead full UDL 0.7 kN/m

Imposed full UDL 1.25 kN/m

Dead point load 4 kN at 4300 mm

Imposed point load 1.4 kN at 4300 mm

Load combinations

Load combination 1

Support A

Dead $\times$ 1.40

Imposed $\times$ 1.60

Dead $\times$ 1.40

Imposed $\times$ 1.60

Support B

Dead $\times$ 1.40

Imposed $\times$ 1.60



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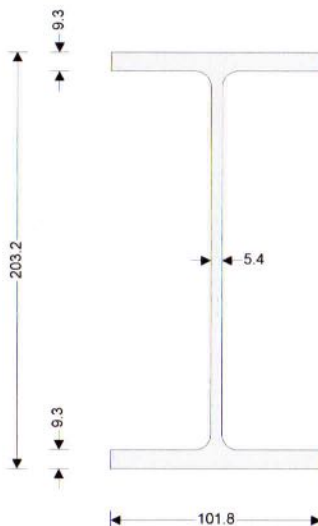
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2nd fl beams between joists in west bay				1 - 45	
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Analysis results

Maximum moment	$M_{max} = 34.9 \text{ kNm}$	$M_{min} = 0 \text{ kNm}$
Maximum moment span 1 segment 1	$M_{s1_seg1_max} = 27 \text{ kNm}$	$M_{s1_seg1_min} = 0 \text{ kNm}$
Maximum moment span 1 segment 2	$M_{s1_seg2_max} = 34.9 \text{ kNm}$	$M_{s1_seg2_min} = 0 \text{ kNm}$
Maximum moment span 1 segment 3	$M_{s1_seg3_max} = 29.9 \text{ kNm}$	$M_{s1_seg3_min} = 0 \text{ kNm}$
Maximum shear	$V_{max} = 14.5 \text{ kN}$	$V_{min} = -15.7 \text{ kN}$
Maximum shear span 1 segment 1	$V_{s1_seg1_max} = 14.5 \text{ kN}$	$V_{s1_seg1_min} = 0 \text{ kN}$
Maximum shear span 1 segment 2	$V_{s1_seg2_max} = 7.1 \text{ kN}$	$V_{s1_seg2_min} = -8.2 \text{ kN}$
Maximum shear span 1 segment 3	$V_{s1_seg3_max} = 0 \text{ kN}$	$V_{s1_seg3_min} = -15.7 \text{ kN}$
Deflection segment 4	$\delta_{max} = 14.7 \text{ mm}$	$\delta_{min} = 0 \text{ mm}$
Maximum reaction at support A	$R_{A_max} = 14.5 \text{ kN}$	$R_{A_min} = 14.5 \text{ kN}$
Unfactored dead load reaction at support A	$R_{A_Dead} = 4.3 \text{ kN}$	
Unfactored imposed load reaction at support A	$R_{A_Imposed} = 5.3 \text{ kN}$	
Maximum reaction at support B	$R_{B_max} = 15.7 \text{ kN}$	$R_{B_min} = 15.7 \text{ kN}$
Unfactored dead load reaction at support B	$R_{B_Dead} = 4.9 \text{ kN}$	
Unfactored imposed load reaction at support B	$R_{B_Imposed} = 5.5 \text{ kN}$	

Section details

Section type	UKB 203x102x23 (Tata Steel Advance)
Steel grade	S275
From table 9: Design strength p_y	
Thickness of element	$\max(T, t) = 9.3 \text{ mm}$
Design strength	$p_y = 275 \text{ N/mm}^2$
Modulus of elasticity	$E = 205000 \text{ N/mm}^2$



Lateral restraint

Span 1 has lateral restraint at supports plus third points

Effective length factors

Effective length factor in major axis	$K_x = 1.00$
Effective length factor in minor axis	$K_y = 1.00$
Effective length factor for lateral-torsional buckling	$K_{LTA} = 1.00$
	$K_{LTB} = 1.00$

Classification of cross sections - Section 3.5

$$\varepsilon = \sqrt{[275 \text{ N/mm}^2 / p_y]} = 1.00$$

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Internal compression parts - Table 11

Depth of section $d = 169.4 \text{ mm}$
 $d / t = 31.4 \times \epsilon \leq 80 \times \epsilon$ Class 1 plastic

Outstand flanges - Table 11

Width of section $b = B / 2 = 50.9 \text{ mm}$
 $b / T = 5.5 \times \epsilon \leq 9 \times \epsilon$ Class 1 plastic
Section is class 1 plastic

Shear capacity - Section 4.2.3

Design shear force $F_v = \max(\text{abs}(V_{\max}), \text{abs}(V_{\min})) = 15.7 \text{ kN}$
 $d / t < 70 \times \epsilon$
Web does not need to be checked for shear buckling
Shear area $A_v = t \times D = 1097 \text{ mm}^2$
Design shear resistance $P_v = 0.6 \times p_y \times A_v = 181.1 \text{ kN}$
PASS - Design shear resistance exceeds design shear force

Moment capacity at span 1 segment 2 - Section 4.2.5

Design bending moment $M = \max(\text{abs}(M_{s1_seg2_max}), \text{abs}(M_{s1_seg2_min})) = 34.9 \text{ kNm}$
Moment capacity low shear - cl.4.2.5.2 $M_c = \min(p_y \times S_{xx}, 1.2 \times p_y \times Z_{xx}) = 64.4 \text{ kNm}$

Effective length for lateral-torsional buckling - Section 4.3.5

Effective length for lateral torsional buckling $L_E = 1.0 \times L_{s1_seg2} = 2500 \text{ mm}$
Slenderness ratio $\lambda = L_E / r_{yy} = 105.890$

Equivalent slenderness - Section 4.3.6.7

Buckling parameter $u = 0.888$
Torsional index $x = 22.460$
Slenderness factor $v = 1 / [1 + 0.05 \times (\lambda / x)^2]^{0.25} = 0.830$
Ratio - cl.4.3.6.9 $\beta_w = 1.000$
Equivalent slenderness - cl.4.3.6.7 $\lambda_{LT} = u \times v \times \lambda \times \sqrt{\beta_w} = 78.009$
Limiting slenderness - Annex B.2.2 $\lambda_{L0} = 0.4 \times (\pi^2 \times E / p_y)^{0.5} = 34.310$
 $\lambda_{LT} > \lambda_{L0}$ - Allowance should be made for lateral-torsional buckling

Bending strength - Section 4.3.6.5

Robertson constant $\alpha_{LT} = 7.0$
Perry factor $\eta_{LT} = \max(\alpha_{LT} \times (\lambda_{LT} - \lambda_{L0}) / 1000, 0) = 0.306$
Euler stress $p_E = \pi^2 \times E / \lambda_{LT}^2 = 332.5 \text{ N/mm}^2$
 $\phi_{LT} = (p_y + (\eta_{LT} + 1) \times p_E) / 2 = 354.6 \text{ N/mm}^2$
Bending strength - Annex B.2.1 $p_b = p_E \times p_y / (\phi_{LT} + (\phi_{LT}^2 - p_E \times p_y)^{0.5}) = 169.4 \text{ N/mm}^2$

Equivalent uniform moment factor - Section 4.3.6.6

Moment at quarter point of segment $M_2 = 30.8 \text{ kNm}$
Moment at centre-line of segment $M_3 = 33.5 \text{ kNm}$
Moment at three quarter point of segment $M_4 = 34.4 \text{ kNm}$
Maximum moment in segment $M_{\text{abs}} = 34.9 \text{ kNm}$
Maximum moment governing buckling resistance $M_{LT} = M_{\text{abs}} = 34.9 \text{ kNm}$
Equivalent uniform moment factor for lateral-torsional buckling
 $m_{LT} = \max(0.2 + (0.15 \times M_2 + 0.5 \times M_3 + 0.15 \times M_4) / M_{\text{abs}}, 0.44) = 0.961$

Buckling resistance moment - Section 4.3.6.4

Buckling resistance moment $M_b = p_b \times S_{xx} = 39.6 \text{ kNm}$



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$$M_b / m_{LT} = 41.3 \text{ kNm}$$

PASS - Buckling resistance moment exceeds design bending moment

Check vertical deflection - Section 2.5.2

Consider deflection due to imposed loads

Limiting deflection

$$\delta_{lim} = L_{s1} / 360 = 20.833 \text{ mm}$$

Maximum deflection span 1

$$\delta = \max(\text{abs}(\delta_{max}), \text{abs}(\delta_{min})) = 14.7 \text{ mm}$$

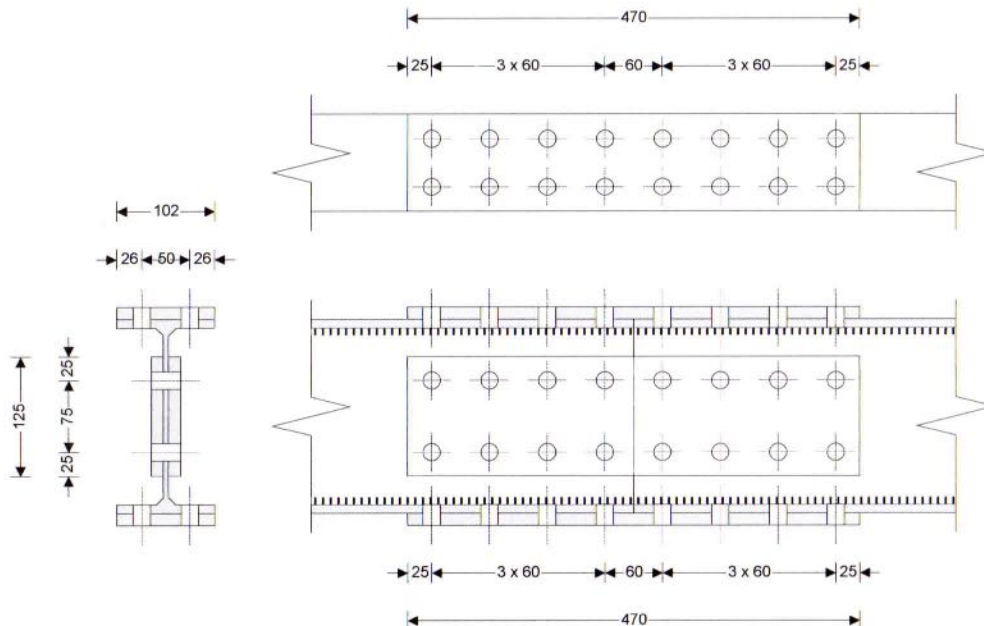
PASS - Maximum deflection does not exceed deflection limit

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BOLTED COVER PLATE SPLICE CONNECTION (BS5950)

BOLTED COVER PLATE SPLICE CONNECTION TO BS5950-1:2000

TEDDS calculation version 1.0.08



Connection loads

Design moment

$M = 35 \text{ kNm}$ ✓

Axial force in the member (compression +ve)

$N = 2 \text{ kN}$

Shear force in the member

$V = 15 \text{ kN}$ ✓

Steel beam details

Beam section classification

UB 203x102x23

Grade of steel section

S275

Section bearing strength

$p_{bs_s} = 460 \text{ N/mm}^2$

General connection details

Grade of steel plate

S275

Plate bearing strength

$p_{bs_p} = 460 \text{ N/mm}^2$

Bolt classification

M16 (Torqued General Grade HSFG)

Hole diameter

$D_h = 18 \text{ mm}$

Bolt slip factor

$\mu = 0.50$

Hole type factor

$K_s = 1.0$

Flange plate details – plates bolted to one side of each flange

Thickness of flange plates

$t_{fp} = 13 \text{ mm}$

Width of flange plates

$b_{fp} = 102 \text{ mm}$

Length of flange plates

$l_{fp} = 470 \text{ mm}$

Flange bolting details

Rows of flange bolts on each side of joint

$n_{fb_r} = 4$

Bolts per row

$n_{fb_p} = 2$

Total number of flange bolts each side of joint

$n_{fb} = n_{fb_r} \times n_{fb_p} = 8$

Spacing between rows of bolts

$S_f = 60 \text{ mm}$



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Spacing between rows of bolts across joint	$S_{fc} = 60 \text{ mm}$
Spacing at end of flange plates	$S_{fe} = 25 \text{ mm}$
Lateral spacing between central bolts	$S_{flc} = 50 \text{ mm}$
Lateral spacing at edge of flange plates	$S_{fle} = 26 \text{ mm}$

Web plate details - plates bolted to both sides of the web

Thickness of web plates	$t_{wp} = 13 \text{ mm}$
Width of web plates	$b_{wp} = 470 \text{ mm}$
Length of web plates	$l_{wp} = 125 \text{ mm}$

Web bolting details

Rows of web bolts	$n_{wb_r} = 2$
Bolts per row each side of joint	$n_{wb_p} = 4$
Total number of web bolts each side of joint	$n_{wb} = n_{wb_r} \times n_{wb_p} = 8$
Spacing between rows of bolts	$S_w = 75 \text{ mm}$
Spacing at end of web plates	$S_{we} = 25 \text{ mm}$
Lateral spacing between bolts	$S_{wl} = 60 \text{ mm}$
Lateral spacing between central bolts	$S_{wlc} = 60 \text{ mm}$
Lateral spacing at edge of web plates	$S_{wle} = 25 \text{ mm}$

Step 1 - Distribution of forces in member flanges

Forces in member tension flange	$T = [M / (D_b - T_b)] - N / 2 = 180 \text{ kN}$
Forces in member compression flange	$C = [M / (D_b - T_b)] + N / 2 = 182 \text{ kN}$
Force in the flange	$F_f = \max(T, C) = 182 \text{ kN}$

Step 2 - Calculate distribution of forces in member flanges

Check area of flange

Design strength of section	$p_{ys} = 275 \text{ N/mm}^2$
Minimum required effective flange area	$F_f / p_{ys} = 660 \text{ mm}^2$
Effective net area coefficient	$K_e = 1.2$
Effective flange area	$A_{ef} = \min(K_e \times [B_b - (n_{fb_p} \times D_h)] \times T_b, B_b \times T_b) = 734 \text{ mm}^2$
PASS - Effective flange area is adequate	

Check area of flange plates

Design strength of plates	$p_{yp} = 275 \text{ N/mm}^2$
Minimum required effective flange plate area	$F_f / p_{yp} = 660 \text{ mm}^2$
Effective flange plate area	$A_{ep} = \min(K_e \times [b_{fp} - (n_{fb_p} \times D_h)] \times t_{fp}, b_{fp} \times t_{fp}) = 990 \text{ mm}^2$
PASS - Effective flange plate area is adequate	

Step 3 - Design of flange bolts

Slip resistance of the bolt per interface	$S_{fb} = 1.1 \times K_s \times \mu \times P_p = 50.7 \text{ kN}$
Bearing capacity of the bolt in the flange	$P_{bg_s} = 1.5 \times d \times T_b \times p_{bs_s} = 102.7 \text{ kN}$
Bearing capacity of the bolt in the plate	$P_{bg_p} = 1.5 \times d \times t_{fp} \times p_{bs_p} = 138.0 \text{ kN}$
Average flange bolt end distance	$S_{fe_{ave}} = S_{fe} + (n_{fb_r} - 1) \times S_f / 2 = 115 \text{ mm}$
Bearing capacity limit of the bolt in the plate	$P_{bg_{p_{lim}}} = 0.5 \times S_{fe_{ave}} \times t_{fp} \times p_{bs_p} = 330.6 \text{ kN}$
Bolt capacity	$P_s = \min(S_{fb}, P_{bg_s}, P_{bg_p}, P_{bg_{p_{lim}}}) = 50.7 \text{ kN}$
Number of bolts required	$n_{fb_{req}} = F_f / P_s = 3.6$
Number of bolts provided	$n_{fb} = 8$

PASS - Flange plate bolting is adequate

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Step 4 - Design of web plates and bolts

Check web plate in shear

Shear force in web plates

$$V = 15 \text{ kN}$$

Gross shear area

$$A_v = n_{wp} \times l_{wp} \times t_{wp} = 3125 \text{ mm}^2$$

Net shear area (allowing for bolt holes)

$$A_{v_net} = n_{wp} \times (l_{wp} - n_{wb_r} \times D_h) \times t_{wp} = 2225 \text{ mm}^2$$

Net shear area limit

$$0.85 \times A_v / K_e = 2214 \text{ mm}^2$$

$A_{v_net} \geq 0.85 \times A_v / K_e$ - Bolt holes may be ignored in the shear area

Gross shear capacity of web plates

$$p_{v_gross} = 0.6 \times A_v \times p_{yp} = 516 \text{ kN}$$

Length of block shear face

$$L_v = S_{we} + (n_{wb_r} - 1) \times S_w = 100 \text{ mm}$$

Length of block tension face

$$L_t = S_{wle} + (n_{wb_p} - 1) \times S_{wl} = 205 \text{ mm}$$

Block shear coefficient

$$k = \text{if}(n_{wb_p} > 1, 2.5, 0.5) = 2.5$$

Block shear capacity of web plates

$$p_{v_block} = 0.6 \times p_{yp} \times t_{wp} \times n_{wp} \times [L_v + K_e \times (L_t - k \times D_h)] = 1205 \text{ kN}$$

Shear capacity of web plates

$$p_v = \min(p_{v_gross}, p_{v_block}) = 516 \text{ kN}$$

PASS - Effective web plate area is adequate in shear

Check web plate in bending

Second moment of area of web plate

$$I = (t_{wp} \times l_{wp}^3 / 12) - (n_{wb_r} \times t_{wp} \times D_h^3 / 12) - (t_{wp} \times D_h \times K \times S_w^2)$$

$$I = 1389543 \text{ mm}^4$$

Distance from joint to centroid of bolt group

$$a = [((n_{wb_p} - 1) \times S_{wl}) + S_{wlc}] / 2 = 120 \text{ mm}$$

Moment in web plate

$$M_{wp} = V \times a = 1.8 \text{ kNm}$$

Moment capacity of web plates

$$M_{cap} = p_{yp} \times n_{wp} \times I / (l_{wp} / 2) = 12.2 \text{ kNm}$$

PASS - Effective web plate area is adequate in bending

Check web plate bolts

Moment of inertia of bolt group

$$I_{bg} = 47250 \text{ mm}^2$$

Force on bolt due to direct shear

$$F_v = V / n_{wb} = 1.9 \text{ kN}$$

Vertical force on bolt due to moment

$$F_{mv} = M_{wp} \times x / I_{bg} = 3.4 \text{ kN}$$

Horizontal force on bolt due to moment

$$F_{mh} = M_{wp} \times y / I_{bg} = 1.4 \text{ kN}$$

Resultant bolt load

$$F_r = \sqrt{(F_v + F_{mv})^2 + F_{mh}^2} = 5.5 \text{ kN}$$

Angle of the resultant bolt load

$$\theta = \text{atan}(F_{mh} / (F_v + F_{mv})) = 15.1 \text{ deg}$$

Minimum edge distance

$$e_r = \min(S_{we} / \cos(\theta), S_{wle} / \cos(90 - \theta)) = 26 \text{ mm}$$

Edge distance factor for web plate bearing

$$K_{edge} = \min(e_r / (3 \times d), 1) = 0.5$$

Slip resistance of the bolt per interface

$$S_{fb} = 1.1 \times K_s \times \mu \times P_p = 50.7 \text{ kN}$$

Bearing capacity of the bolt in the web

$$P_{bg_s} = 1.5 \times d \times t_b \times p_{bs_s} = 59.6 \text{ kN}$$

Bearing capacity of the bolt in the plate

$$P_{bg_p} = 1.5 \times K_{edge} \times d \times t_{wp} \times n_{wp} \times p_{bs_p} = 148.9 \text{ kN}$$

Bolt capacity

$$P_s = \min(n_{wp} \times S_{fb}, P_{bg_s}, P_{bg_p}) = 59.6 \text{ kN}$$

PASS - Web plate bolting is adequate

Connection summary

Beam classification

UB 203x102x23

Bolt classification

M16 (Torqued General Grade HSFG)

Flange plates

470 mm x 102 mm x 12.5 mm to the outside of each flange

Flange bolting

16 No. total per flange - 4 No. rows of 2 No. bolts on each side of the joint

Web plates

125 mm x 470 mm x 12.5 mm on each side of the web

Web bolting

16 No. total - 2 No. rows of 4 No. bolts on each side of the joint

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	CALC SHEET	M _f KN m _f	V _f KN _f
SPLICE TYPE A. 2nd FL. IN BETWEEN JOISTS. 203x102x23UB	1-48 to 1-50	35 (1-44)	15 (1-44)
SPLICE TYPE B 1st FL. PRIMARY BEAMS. 260x90x35PFC	1- ~~13~~ 13. to 1-	138 (1-35)	74 (1-35)
SPLICE TYPE C. 2nd FL. PRIMARY BEAMS 305-165-40UB	1-52. to 1-58	227 (1-31)	103 (1-31)



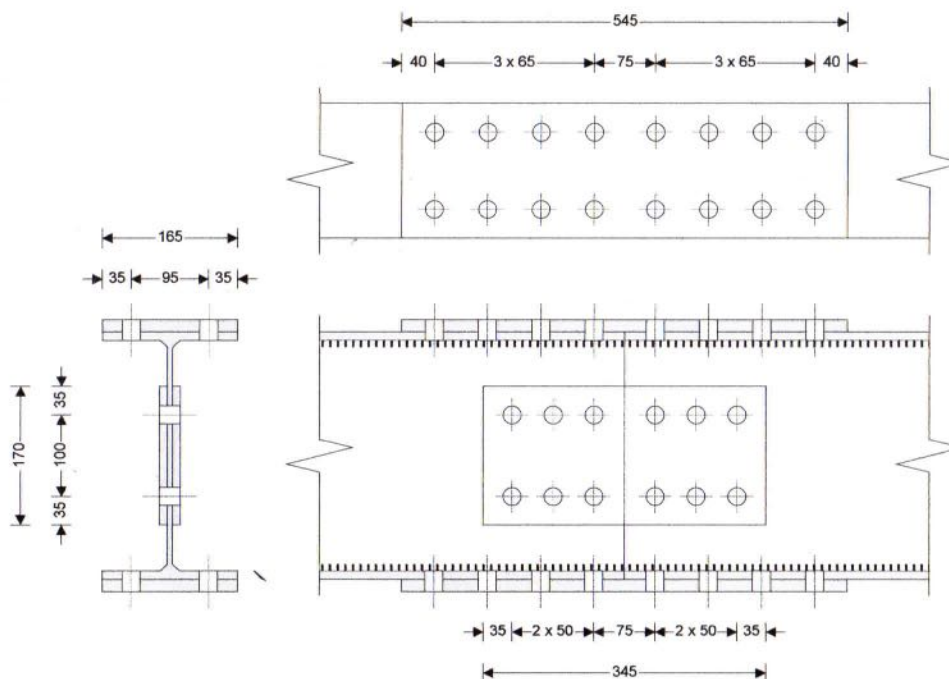
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BOLTED COVER PLATE SPLICE CONNECTION (BS5950)

BOLTED COVER PLATE SPLICE CONNECTION TO BS5950-1:2000

TEDDS calculation version 1.0.08



Connection loads

Design moment

$M = 114 \text{ kNm}$

Axial force in the member (compression +ve)

$N = 2 \text{ kN}$

Shear force in the member

$V = 52 \text{ kN}$

Steel beam details

Beam section classification

UB 305x165x40

Grade of steel section

S275

Section bearing strength

$p_{bs_s} = 460 \text{ N/mm}^2$

General connection details

Grade of steel plate

S275

Plate bearing strength

$p_{bs_p} = 460 \text{ N/mm}^2$

Bolt classification

M20 (Torqued General Grade HSFG)

Hole diameter

$D_h = 22 \text{ mm}$

Bolt slip factor

$\mu = 0.50$

Hole type factor

$K_s = 1.0$

Flange plate details – plates bolted to one side of each flange

Thickness of flange plates

$t_{fp} = 15 \text{ mm}$

Width of flange plates

$b_{fp} = 165 \text{ mm}$

Length of flange plates

$l_{fp} = 545 \text{ mm}$

Flange bolting details

Rows of flange bolts on each side of joint

$n_{fb_r} = 4$

Bolts per row

$n_{fb_p} = 2$

Total number of flange bolts each side of joint

$n_{fb} = n_{fb_r} \times n_{fb_p} = 8$



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Spacing between rows of bolts	$S_r = 65 \text{ mm}$
Spacing between rows of bolts across joint	$S_{fc} = 75 \text{ mm}$
Spacing at end of flange plates	$S_{fe} = 40 \text{ mm}$
Lateral spacing between central bolts	$S_{flc} = 95 \text{ mm}$
Lateral spacing at edge of flange plates	$S_{fle} = 35 \text{ mm}$

Web plate details - plates bolted to both sides of the web

Thickness of web plates	$t_{wp} = 10 \text{ mm}$
Width of web plates	$b_{wp} = 345 \text{ mm}$
Length of web plates	$l_{wp} = 170 \text{ mm}$

Web bolting details

Rows of web bolts	$n_{wb_r} = 2$
Bolts per row each side of joint	$n_{wb_p} = 3$
Total number of web bolts each side of joint	$n_{wb} = n_{wb_r} \times n_{wb_p} = 6$
Spacing between rows of bolts	$S_w = 100 \text{ mm}$
Spacing at end of web plates	$S_{we} = 35 \text{ mm}$
Lateral spacing between bolts	$S_{wl} = 50 \text{ mm}$
Lateral spacing between central bolts	$S_{wlc} = 75 \text{ mm}$
Lateral spacing at edge of web plates	$S_{wle} = 35 \text{ mm}$

Step 1 - Distribution of forces in member flanges

Forces in member tension flange	$T = [M / (D_b - T_b)] - N / 2 = 388 \text{ kN}$
Forces in member compression flange	$C = [M / (D_b - T_b)] + N / 2 = 390 \text{ kN}$
Force in the flange	$F_f = \max(T, C) = 390 \text{ kN}$

Step 2 - Calculate distribution of forces in member flanges

Check area of flange

Design strength of section	$p_{ys} = 275 \text{ N/mm}^2$
Minimum required effective flange area	$F_f / p_{ys} = 1418 \text{ mm}^2$
Effective net area coefficient	$K_e = 1.2$
Effective flange area	$A_{ef} = \min(K_e \times [B_b - (n_{fb_p} \times D_h)] \times T_b, B_b \times T_b) = 1481 \text{ mm}^2$
PASS - Effective flange area is adequate	

Check area of flange plates

Design strength of plates	$p_{yp} = 275 \text{ N/mm}^2$
Minimum required effective flange plate area	$F_f / p_{yp} = 1418 \text{ mm}^2$
Effective flange plate area	$A_{ep} = \min(K_e \times [b_{fp} - (n_{fb_p} \times D_h)] \times t_{fp}, b_{fp} \times t_{fp}) = 2178 \text{ mm}^2$
PASS - Effective flange plate area is adequate	

Step 3 - Design of flange bolts

Slip resistance of the bolt per interface	$S_{fb} = 1.1 \times K_s \times \mu \times P_p = 79.2 \text{ kN}$
Bearing capacity of the bolt in the flange	$P_{bg_s} = 1.5 \times d \times T_b \times p_{bs_s} = 140.8 \text{ kN}$
Bearing capacity of the bolt in the plate	$P_{bg_p} = 1.5 \times d \times t_{fp} \times p_{bs_p} = 207.0 \text{ kN}$
Average flange bolt end distance	$S_{fe_{ave}} = S_{fe} + (n_{fb_r} - 1) \times S_r / 2 = 138 \text{ mm}$
Bearing capacity limit of the bolt in the plate	$P_{bg_p_{lim}} = 0.5 \times S_{fe_{ave}} \times t_{fp} \times p_{bs_p} = 474.4 \text{ kN}$
Bolt capacity	$P_s = \min(S_{fb}, P_{bg_s}, P_{bg_p}, P_{bg_p_{lim}}) = 79.2 \text{ kN}$
Number of bolts required	$n_{fb_{req}} = F_f / P_s = 4.9$
Number of bolts provided	$n_{fb} = 8$

PASS - Flange plate bolting is adequate

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Step 4 - Design of web plates and bolts

Check web plate in shear

Shear force in web plates

$$V = 52 \text{ kN}$$

Gross shear area

$$A_v = n_{wp} \times l_{wp} \times t_{wp} = 3400 \text{ mm}^2$$

Net shear area (allowing for bolt holes)

$$A_{v_net} = n_{wp} \times (l_{wp} - n_{wb_r} \times D_h) \times t_{wp} = 2520 \text{ mm}^2$$

Net shear area limit

$$0.85 \times A_v / K_e = 2408 \text{ mm}^2$$

$A_{v_net} \geq 0.85 \times A_v / K_e$ - Bolt holes may be ignored in the shear area

Gross shear capacity of web plates

$$p_{v_gross} = 0.6 \times A_v \times p_{yp} = 561 \text{ kN}$$

Length of block shear face

$$L_v = S_{we} + (n_{wb_r} - 1) \times S_w = 135 \text{ mm}$$

Length of block tension face

$$L_t = S_{wle} + (n_{wb_p} - 1) \times S_{wl} = 135 \text{ mm}$$

Block shear coefficient

$$k = \text{if}(n_{wb_p} > 1, 2.5, 0.5) = 2.5$$

Block shear capacity of web plates

$$p_{v_block} = 0.6 \times p_{yp} \times t_{wp} \times n_{wp} \times [L_v + K_e \times (L_t - k \times D_h)] = 762 \text{ kN}$$

Shear capacity of web plates

$$p_v = \min(p_{v_gross}, p_{v_block}) = 561 \text{ kN}$$

PASS - Effective web plate area is adequate in shear

Check web plate in bending

Second moment of area of web plate

$$I = (t_{wp} \times l_{wp}^3 / 12) - (n_{wb_r} \times t_{wp} \times D_h^3 / 12) - (t_{wp} \times D_h \times K \times S_w^2)$$

$$I = 2976420 \text{ mm}^4$$

Distance from joint to centroid of bolt group

$$a = [((n_{wb_p} - 1) \times S_{wl}) + S_{wlc}] / 2 = 87 \text{ mm}$$

Moment in web plate

$$M_{wp} = V \times a = 4.6 \text{ kNm}$$

Moment capacity of web plates

$$M_{cap} = p_{yp} \times n_{wp} \times I / (l_{wp} / 2) = 19.3 \text{ kNm}$$

PASS - Effective web plate area is adequate in bending

Check web plate bolts

Moment of inertia of bolt group

$$I_{bg} = 25000 \text{ mm}^2$$

Force on bolt due to direct shear

$$F_v = V / n_{wb} = 8.7 \text{ kN}$$

Vertical force on bolt due to moment

$$F_{mv} = M_{wp} \times x / I_{bg} = 9.1 \text{ kN}$$

Horizontal force on bolt due to moment

$$F_{mh} = M_{wp} \times y / I_{bg} = 9.1 \text{ kN}$$

Resultant bolt load

$$F_r = \sqrt{(F_v + F_{mv})^2 + F_{mh}^2} = 20.0 \text{ kN}$$

Angle of the resultant bolt load

$$\theta = \text{atan}(F_{mh} / (F_v + F_{mv})) = 27.1 \text{ deg}$$

Minimum edge distance

$$e_r = \min(S_{we} / \cos(\theta), S_{wle} / \cos(90 - \theta)) = 39 \text{ mm}$$

Edge distance factor for web plate bearing

$$K_{edge} = \min(e_r / (3 \times d), 1) = 0.7$$

Slip resistance of the bolt per interface

$$S_{fb} = 1.1 \times K_s \times \mu \times P_p = 79.2 \text{ kN}$$

Bearing capacity of the bolt in the web

$$P_{bg_s} = 1.5 \times d \times t_b \times p_{bs_s} = 82.8 \text{ kN}$$

Bearing capacity of the bolt in the plate

$$P_{bg_p} = 1.5 \times K_{edge} \times d \times t_{wp} \times n_{wp} \times p_{bs_p} = 180.9 \text{ kN}$$

Bolt capacity

$$P_s = \min(n_{wp} \times S_{fb}, P_{bg_s}, P_{bg_p}) = 82.8 \text{ kN}$$

PASS - Web plate bolting is adequate

Connection summary

Beam classification

UB 305x165x40

Bolt classification

M20 (Torqued General Grade HSFG)

Flange plates

545 mm x 165 mm x 15 mm to the outside of each flange

Flange bolting

16 No. total per flange - 4 No. rows of 2 No. bolts on each side of the joint

Web plates

170 mm x 345 mm x 10 mm on each side of the web

Web bolting

12 No. total - 2 No. rows of 3 No. bolts on each side of the joint