



589 – Abbot's Barn, Lower South Wraxall, Bradford-on-Avon
BA15 2RS

STRUCTURAL REPORT

Proposed internal alterations

August 2017

Version	Date	Description
1.0	04/08/17	First issue - final for construction.



Appendix A : Structural Drawings



PROJECT
ABBOT'S BARN

PROJECT No.
589

DATE
4/8/17

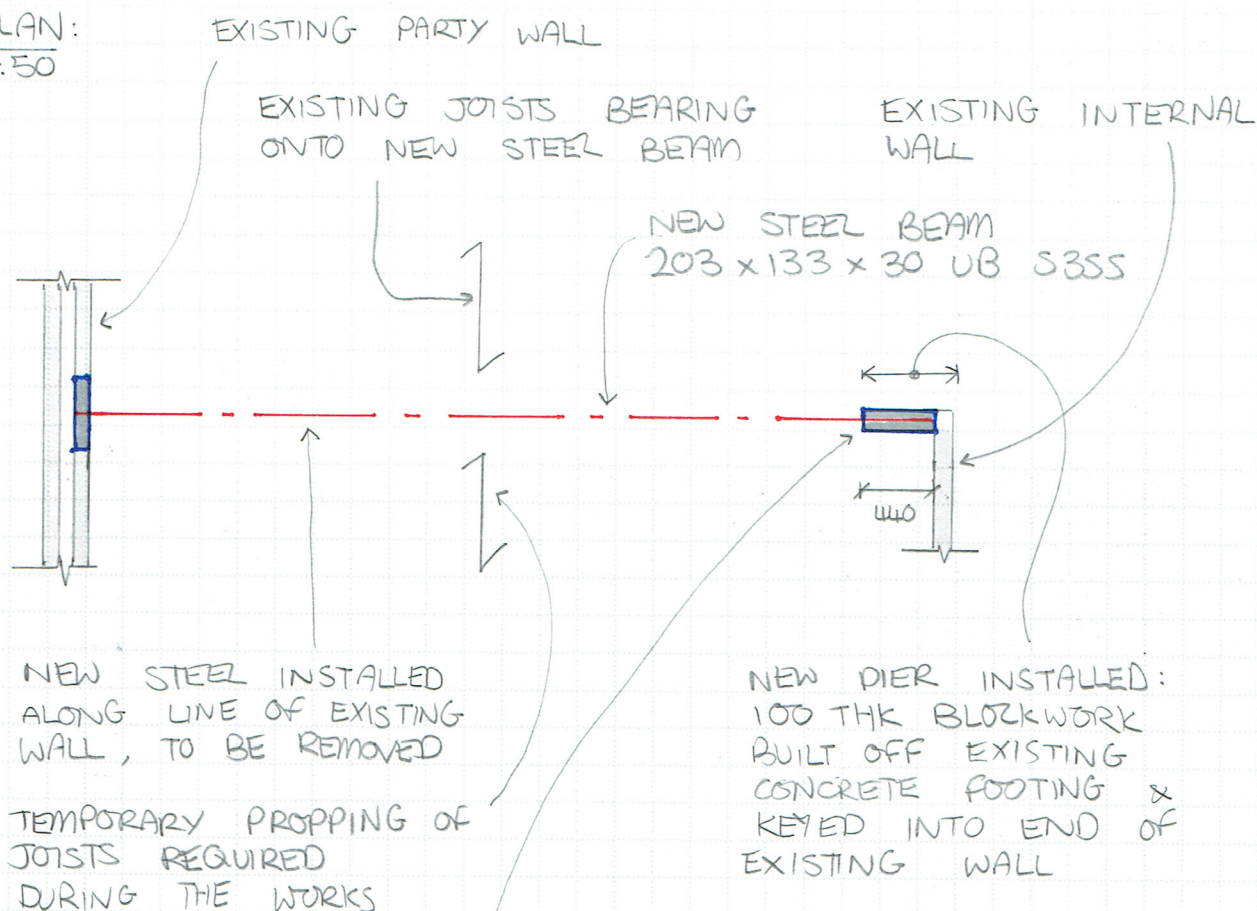
REV
1

NAME
STEEL BEAM

PAGE No.
SK01

AUTHOR
RP

PLAN:
1:50

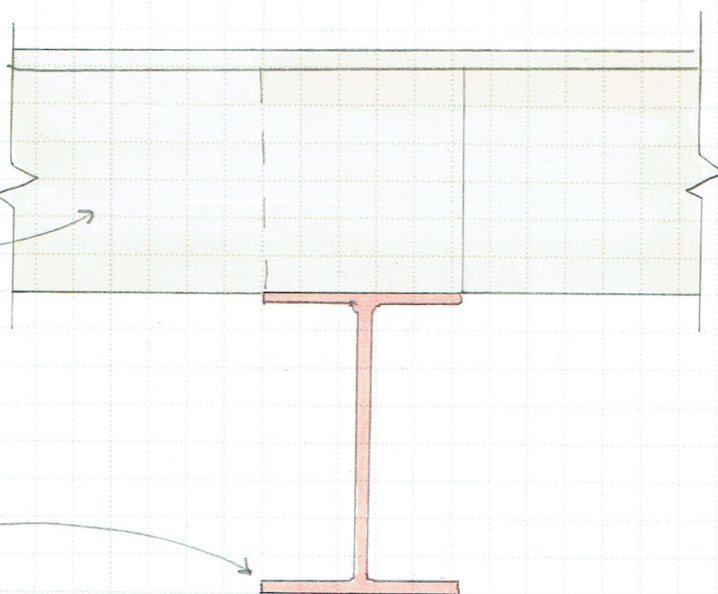


NEW PADSTONE TO EACH
END OF STEEL BEAM:
440 L x 100 W x 215 H
CONCRETE

DETAIL:
1:5

EXISTING JOISTS

STEEL BEAM
203 x 133 x 30 UB





Appendix B : Structural Calculations

	PROJECT ABBOT'S BARN	PROJECT No. 589	DATE 4/8/17	REV
	NAME STEEL BEAM	PAGE No.	AUTHOR RP	

- New steel beam to support FF joists, spanning over new opening in existing wall.

Span = 5.2m

Applied load: FF joists width $\frac{2.9+2.4}{2} = 2.65\text{m}$

$$\begin{aligned} DL &= 0.6 \times 2.65 = 1.6 \text{ kN/m} \\ LL &= 2.0 \times 2.65 = 5.3 \text{ kN/m} \end{aligned}$$

USE 203 x 133 x 30 UB S355

- BEARING

Bearing onto party wall.

100 THK block
on 600 WD footing. approx 450 OP
assume allowable bearing pressure = 75 kN/m^2

applied load from steel: $D = 6 \text{ kN}$
 $LL = 14 \text{ kN}$

load spreads down through wall

UDL on wall: assume 3m of masonry $DL = 18 \times 0.1 \times 3 = 5.4 \text{ kN/m}$
bearing area at foundation level = $3\text{m} \times 0.6\text{m}$

multiply load by 2 to account for adjacent property:

$$6 \text{ kN} + 14 \text{ kN} \times 2 = 40 \text{ kN}$$

$$UDL = 5.4 + 5.4 \times 2 = 21.6 \text{ kN/m}$$

$$40 \div (3 \times 0.6) + 21.6 \div 0.6 = 58.2 \text{ kN/m}^2 < 75 \therefore \text{OK}$$

USE 440 x 100 x 215 PADSTONE (calc overhead)

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STEEL MEMBER ANALYSIS & DESIGN (EN1993)

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

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

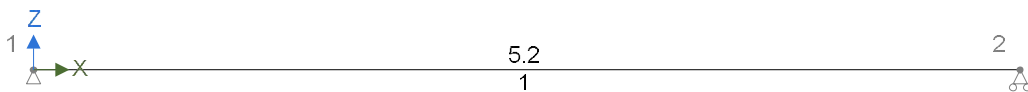
Tedds calculation version 4.2.00

ANALYSIS

Tedds calculation version 1.0.19

Geometry

Geometry (m) - Steel (EC3) - UKB 203x133x30



Span	Length (m)	Section	Start Support	End Support
1	5.2	UKB 203x133x30	Pinned	Roller Pin X
UKB 203x133x30: $A = 38 \text{ cm}^2$, $I_y = 2896 \text{ cm}^4$, $I_z = 385 \text{ cm}^4$, $A_y = 23 \text{ cm}^2$, $A_z = 13 \text{ cm}^2$				
Steel (EC3): Density 7850 kg/m^3 , Youngs 210 kN/mm^2 , Shear 80.8 kN/mm^2 , Thermal $0.000012 \text{ }^\circ\text{C}^{-1}$				

Loading

Self weight included

Permanent - Loading (kN/m)



Imposed - Loading (kN/m)



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Load combination factors

Load combination	Self Weight	Permanent	Imposed
Default (Strength)	1.35	1.35	1.50
Default (Service)	1.00	1.00	1.00

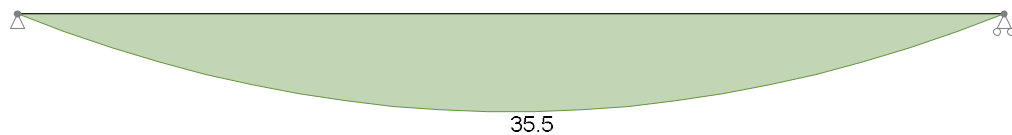
Member Loads

Member	Load case	Load Type	Orientation	Description
Beam	Permanent	UDL	GlobalZ	1.6 kN/m
Beam	Imposed	UDL	GlobalZ	5.3 kN/m

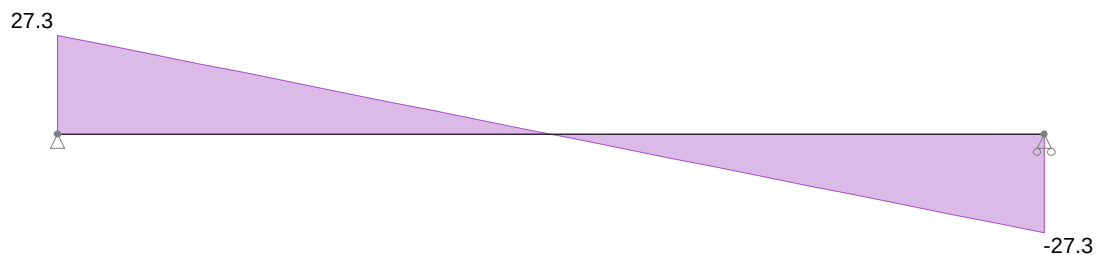
Results

Forces

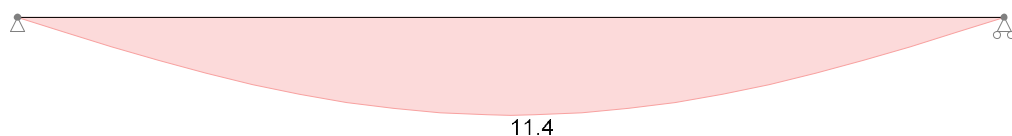
Strength combinations - Moment envelope (kNm)



Strength combinations - Shear envelope (kN)



Service combinations - Deflection envelope (mm)



Partial factors - Section 6.1

Resistance of cross-sections	$\gamma_{M0} = 1$
Resistance of members to instability	$\gamma_{M1} = 1$
Resistance of tensile members to fracture	$\gamma_{M2} = 1.1$

Beam design

Section details

Section type	UKB 203x133x30 (Tata Steel Advance)
Steel grade - EN 10025-2:2004	S355
Nominal thickness of element	$t_{nom} = \max(t_f, t_w) = 9.6 \text{ mm}$
Nominal yield strength	$f_y = 355 \text{ N/mm}^2$

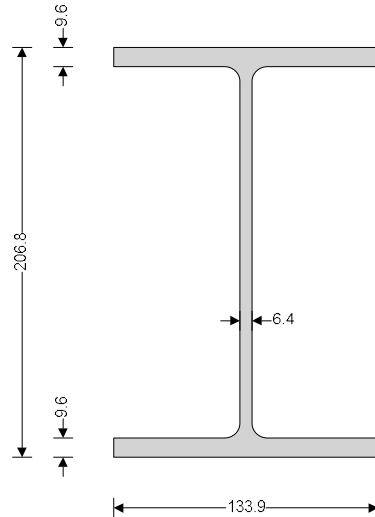
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Nominal ultimate tensile strength

$$f_u = 470 \text{ N/mm}^2$$

Modulus of elasticity

$$E = 210000 \text{ N/mm}^2$$



UKB 203x133x30 (Tata Steel Advance)

Section depth, h , 206.8 mm

Section breadth, b , 133.9 mm

Mass of section, Mass, 30 kg/m

Flange thickness, t_f , 9.6 mm

Web thickness, t_w , 6.4 mm

Root radius, r , 7.6 mm

Area of section, A , 3821 mm²

Radius of gyration about y-axis, i_y , 87.051 mm

Radius of gyration about z-axis, i_z , 31.728 mm

Elastic section modulus about y-axis, $W_{el,y}$, 280036 mm³

Elastic section modulus about z-axis, $W_{el,z}$, 57454 mm³

Plastic section modulus about y-axis, $W_{pl,y}$, 314365 mm³

Plastic section modulus about z-axis, $W_{pl,z}$, 88224 mm³

Second moment of area about y-axis, I_y , 28955684 mm⁴

Second moment of area about z-axis, I_z , 3846542 mm⁴

Lateral restraint

Both flanges have lateral restraint at supports only

Classification of cross sections - Section 5.5

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

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

Width of section

$$c = d = 172.4 \text{ mm}$$

$$c / t_w = 26.9 = 33.1 \times \varepsilon \leq 72 \times \varepsilon \quad \text{Class 1}$$

Outstand flanges - Table 5.2 (sheet 2 of 3)

Width of section

$$c = (b - t_w - 2 \times r) / 2 = 56.2 \text{ mm}$$

$$c / t_f = 5.8 = 7.2 \times \varepsilon \leq 9 \times \varepsilon \quad \text{Class 1}$$

Section is class 1

Check design at start of span

Check shear - Section 6.2.6

Height of web

$$h_w = h - 2 \times t_f = 187.6 \text{ mm} \quad \eta = 1.000$$

$$h_w / t_w = 29.3 = 36 \times \varepsilon / \eta < 72 \times \varepsilon / \eta$$

Shear buckling resistance can be ignored

Design shear force

$$V_{y,Ed} = 27.3 \text{ kN}$$

Shear area - cl 6.2.6(3)

$$A_v = \max(A - 2 \times b \times t_f + (t_w + 2 \times r) \times t_f, \eta \times h_w \times t_w) = 1458 \text{ mm}^2$$

Design shear resistance - cl 6.2.6(2)

$$V_{c,y,Rd} = V_{pl,y,Rd} = A_v \times (f_y / \sqrt{3}) / \gamma_{M0} = 298.7 \text{ kN}$$

$$V_{y,Ed} / V_{c,y,Rd} = 0.091$$

PASS - Design shear resistance exceeds design shear force

Check design 2600 mm along span

Check bending moment - Section 6.2.5

Design bending moment

$$M_{y,Ed} = 35.5 \text{ kNm}$$

Design bending resistance moment - eq 6.13

$$M_{c,y,Rd} = M_{pl,y,Rd} = W_{pl,y} \times f_y / \gamma_{M0} = 111.6 \text{ kNm}$$

$$M_{y,Ed} / M_{c,y,Rd} = 0.318$$

PASS - Design bending resistance moment exceeds design bending moment

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Slenderness ratio for lateral torsional buckling

Correction factor - Table 6.6

$$k_c = 0.94$$

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

Poissons ratio

$$\nu = 0.3$$

Shear modulus

$$G = E / [2 \times (1 + \nu)] = 80769 \text{ N/mm}^2$$

Unrestrained effective length

$$L = 1.0 \times L_{m1_s1_seg1_T} = 5200 \text{ mm}$$

Elastic critical buckling moment

$$M_{cr} = C_1 \times \pi^2 \times E \times I_z / L^2 \times \sqrt{(I_w / I_z + L^2 \times G \times I_t / (\pi^2 \times E \times I_z))} = 65 \text{ kNm}$$

Slenderness ratio for lateral torsional buckling

$$\bar{\lambda}_{LT} = \sqrt{(W_{pl,y} \times f_y / M_{cr})} = 1.31$$

Limiting slenderness ratio

$$\bar{\lambda}_{LT,0} = 0.4$$

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

Check buckling resistance - Section 6.3.2.1

Buckling curve - Table 6.5

b

Imperfection factor - Table 6.3

$$\alpha_{LT} = 0.34$$

Correction factor for rolled sections

$$\beta = 0.75$$

LTB reduction determination factor

$$\phi_{LT} = 0.5 \times [1 + \alpha_{LT} \times (\bar{\lambda}_{LT} - \bar{\lambda}_{LT,0}) + \beta \times \bar{\lambda}_{LT}^2] = 1.299$$

LTB reduction factor - eq 6.57

$$\chi_{LT} = \min(1 / [\phi_{LT} + \sqrt{(\phi_{LT}^2 - \beta \times \bar{\lambda}_{LT}^2)}], 1, 1 / \bar{\lambda}_{LT}^2) = 0.518$$

Modification factor

$$f = \min(1 - 0.5 \times (1 - k_c) \times [1 - 2 \times (\bar{\lambda}_{LT} - 0.8)^2], 1) = 0.986$$

Modified LTB reduction factor - eq 6.58

$$\chi_{LT,mod} = \min(\chi_{LT} / f, 1, 1 / \bar{\lambda}_{LT}^2) = 0.526$$

Design buckling resistance moment - eq 6.55

$$M_{b,y,Rd} = \chi_{LT,mod} \times W_{pl,y} \times f_y / \gamma_{M1} = 58.7 \text{ kNm}$$

$$M_{y,Ed} / M_{b,y,Rd} = 0.605$$

PASS - Design buckling resistance moment exceeds design bending moment

Consider Combination 2 - Default (Service)

Check design 2600 mm along span

Check y-y axis deflection - Section 7.2.1

Maximum deflection

$$\delta_y = 11.4 \text{ mm}$$

Allowable deflection

$$\delta_{y,Allowable} = L_{m1_s1} / 360 = 14.4 \text{ mm}$$

$$\delta_y / \delta_{y,Allowable} = 0.789$$

PASS - Allowable deflection exceeds design deflection

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MASONRY BEARING DESIGN (EN1996)

MASONRY BEARING DESIGN

In accordance with EN1996-1-1:2005 + A1:2012, incorporating Corrigenda February 2006 and July 2009 and the UK National Annex.

Tedds calculation version 1.0.07

Masonry panel details

Panel length	L = 4000 mm
Panel height	h = 2300 mm
Thickness of load bearing leaf	t = 100 mm
Effective height	h_{ef} = 2300 mm
Effective thickness	t_{ef} = 125 mm

Masonry material details

Unit type	Aggregate concrete - Group 1
Compressive strength of masonry unit	f_c = 3.6 N/mm ²
Height of unit	h_u = 215 mm
Width of unit	w_u = 100 mm
Conditioning factor	k = 1.0
- Conditioning to the air dry condition in accordance with cl.7.3.2	
Shape factor - Table A.1	d_{sf} = 1.38
Mean compressive strength of masonry unit	$f_b = f_c \times k \times d_{sf}$ = 4.97 N/mm ²
Specific weight of units	γ = 18 kN/m ³
Mortar type	M4 - General Purpose
Compressive strength of mortar	f_m = 4.0 N/mm ²
Compressive strength factor - Tbl. NA 4	K = 0.75
Characteristic compressive strength of the masonry - eq. 3.1	$f_k = K \times f_b^{0.7} \times f_m^{0.3}$ = 3.49 N/mm ²
Short term secant modulus of elasticity factor	K_E = 1000
Modulus of elasticity - cl.3.7.2	$E_w = K_E \times f_k$ = 3491 N/mm ²

Design compressive strength of masonry

Category of manufacturing control	Category II
Class of execution control	Class 2
Partial factor for material strength in direct or flexural compression	

$$\gamma_M = \mathbf{3.00}$$

Cross-sectional area of wall	$A = L \times t$ = 0.40 m ²
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Design compressive strength of masonry	$f_d = f_k / \gamma_M$ = 1.16 N/mm ²
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Partial safety factors for design loads

Partial safety factor for permanent load	γ_{fG} = 1.35
Partial safety factor for variable load	γ_{fQ} = 1.50

Superimposed vertical loading details

Permanent UDL at top of wall	g_k = 5.40 kN/m
Variable UDL at top of wall	q_k = 0.00 kN/m
Eccentricity of permanent UDL load	e_{gu} = 0 mm
Eccentricity of variable UDL load	e_{qu} = 0 mm

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Slenderness ratio of masonry wall - Section 5.5.1.4

Slenderness ratio limit

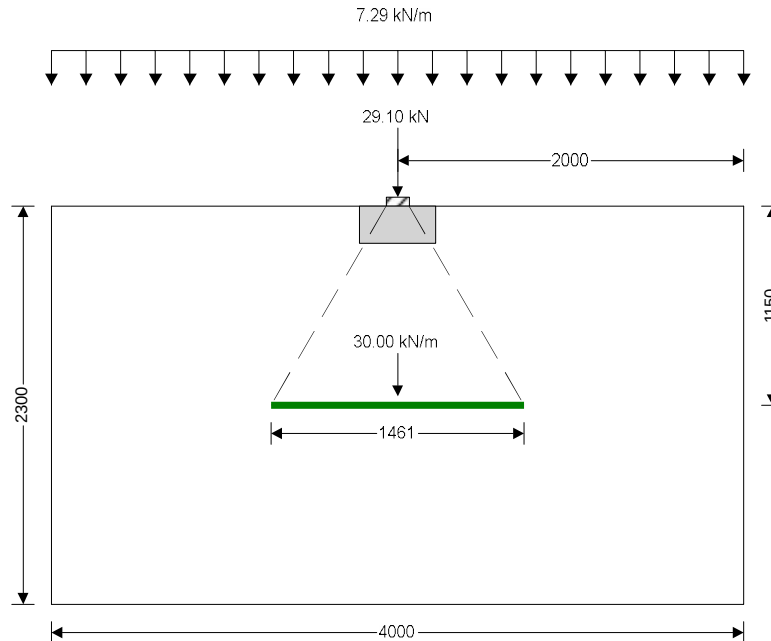
$$\lambda_{lim} = 27$$

Slenderness ratio

$$\lambda = h_{ef} / t_{ef} = 18.4$$

PASS - Slenderness ratio is less than slenderness limit

Concentrated Load 1 details - steel beam



Permanent concentrated load

$$G_{kc1} = 6.00 \text{ kN}$$

Variable concentrated load

$$Q_{kc1} = 14.00 \text{ kN}$$

Eccentricity of concentrated load

$$e_{c1} = 20 \text{ mm}$$

Length of concentrated load

$$L_{c1} = 133 \text{ mm}$$

Width of concentrated load

$$w_{c1} = 100 \text{ mm}$$

Height of concentrated load

$$h_{c1} = 2300 \text{ mm}$$

Distance of load to right vertical edge

$$r_{11} = 1934 \text{ mm}$$

Distance of load to nearest vertical edge

$$a_{11} = 1934 \text{ mm}$$

Walls subjected to concentrated loads - Section 6.1.3

Eccentricity check

$$e_{c1} \leq t / 4$$

PASS - Eccentricity of load is less than $t/4$

Area of bearing

$$A_{b1} = L_{c1} \times w_{c1} = 13300 \text{ mm}^2$$

Effective length of bearing at mid-height

$$l_{efm1} = L_{c1} + h_{c1} \times \tan(30) = 1461 \text{ mm}$$

Effective bearing area

$$A_{ef1} = l_{efm1} \times t = 146091 \text{ mm}^2$$

Bearing area ratio check

$$A_{ratio1} = \text{Min}(A_{b1} / A_{ef1}, 0.45) = 0.09$$

Initial enhancement factor

$$\beta_{init1} = \text{Max}((1 + 0.3 \times a_{11} / h_{c1}) \times (1.5 - 1.1 \times A_{ratio1}), 1.0) = 1.75$$

Maximum enhancement factor

$$\beta_{max1} = \text{Min}(1.25 + a_{11} / (2 \times h_{c1}), 1.5) = 1.50$$

Enhancement factor for concentrated loads

$$\beta_1 = \text{Min}(\beta_{init1}, \beta_{max1}) = 1.50$$

Design value of the concentrated load

$$N_{Edc1} = G_{kc1} \times \gamma_{fG} + Q_{kc1} \times \gamma_{fQ} = 29.10 \text{ kN}$$

Design value concentrated load resistance

$$N_{Rdc1} = \beta_1 \times A_{b1} \times f_d = 23.22 \text{ kN}$$

Applied concentrated load exceeds design resistance, spreader required!

Design of spreader beam

Type of spreader

Concrete padstone

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Type of bearing onto spreader	Point load
Location of load from RHS of spreader	$P_{11} = 220 \text{ mm}$
Length of spreader	$L_{sp1} = 440 \text{ mm}$
Height of spreader	$h_{sp1} = 215 \text{ mm}$
Width of spreader	$w_{sp1} = 100 \text{ mm}$
Eccentricity of load on spreader	$e_{sp1} = 20 \text{ mm}$
Modulus of elasticity	$E_{sp1} = 29962 \text{ N/mm}^2$
Second moment of area	$I_{sp1} = 1/12 \times w_{sp1} \times h_{sp1}^3 = 82819792 \text{ mm}^4$
Modulus of the wall	$k_0 = E_w / h = 1.52 \text{ N/mm}^2/\text{mm}$
Winkler's constant	$K_{c1} = k_0 \times w_{sp1} = 151.80 \text{ N/mm/mm}$
Characteristic of the system	$\alpha_1 = (K_{c1} / (4 \times E_{sp1} \times I_{sp1}))^{1/4} = 0.00198 \text{ mm}^{-1}$
Classification of spreader	$\alpha L_1 = \alpha_1 \times L_{sp1} = 0.87 \text{ Medium}$
Krilov's functions for the spreader length	$B_{\alpha 1} = 1/2 \times (\cosh(\alpha L_1) \times \sin(180 \times \alpha L_1 / \pi) + \sinh(\alpha L_1) \times \cos(180 \times \alpha L_1 / \pi)) = 0.85$ $C_{\alpha 1} = 1/2 \times \sinh(\alpha L_1) \times \sin(180 \times \alpha L_1 / \pi) = 0.38$ $D_{\alpha 1} = 1/4 \times (\cosh(\alpha L_1) \times \sin(180 \times \alpha L_1 / \pi) - \sinh(\alpha L_1) \times \cos(180 \times \alpha L_1 / \pi)) = 0.11$
Krilov's functions at the point load	$A_{\alpha P11} = \cosh(\alpha_1 \times P_{11}) \times \cos(180 \times \alpha_1 \times P_{11} / \pi) = 0.99$ $B_{\alpha P11} = 1/2 \times (\cosh(\alpha_1 \times P_{11}) \times \sin(180 \times \alpha_1 \times P_{11} / \pi) + \sinh(\alpha_1 \times P_{11}) \times \cos(180 \times \alpha_1 \times P_{11} / \pi)) = 0.43$
Using method of initial conditions	
Initial moment of LH edge	$M_{01} = 0 \text{ kNm}$
Initial shear of LH edge	$V_{01} = 0 \text{ kN}$
Which gives	$(4 \times \alpha_1^2 \times C_{\alpha 1} \times \delta_{01} + 4 \times \alpha_1 \times D_{\alpha 1} \times \Phi_{01}) \times E_{sp1} \times I_{sp1} - B_{\alpha P11} / \alpha_1 \times N_{Edc1} = 0.00 \text{ kNm}$
and	$(4 \times \alpha_1^3 \times B_{\alpha 1} \times \delta_{01} + 4 \times \alpha_1^2 \times C_{\alpha 1} \times \Phi_{01}) \times E_{sp1} \times I_{sp1} - A_{\alpha P11} \times N_{Edc1} = 0.00 \text{ kN}$
Therefore,	
Initial deflection of LH edge	$\delta_{01} = 0.43101 \text{ mm}$
Initial rotation of LH edge	$\Phi_{01} = 0.000047$
Location of maximum deflection	$x_{def1} = 220 \text{ mm}$
Krilov's functions at the spreader length	$A_{\alpha xdef1} = \cosh(\alpha_1 \times x_{def1}) \times \cos(180 \times \alpha_1 \times x_{def1} / \pi) = 0.99$ $B_{\alpha xdef1} = 1/2 \times (\cosh(\alpha_1 \times x_{def1}) \times \sin(180 \times \alpha_1 \times x_{def1} / \pi) + \sinh(\alpha_1 \times x_{def1}) \times \cos(180 \times \alpha_1 \times x_{def1} / \pi)) = 0.43$
Distance of point load right of loaction	$p_{1def1} = 0 \text{ mm}$
Krilov's functions at the spreader length	$D_{\alpha p1def1} = 1/4 \times (\cosh(\alpha_1 \times p_{1def1}) \times \sin(180 \times \alpha_1 \times p_{1def1} / \pi) - \sinh(\alpha_1 \times p_{1def1}) \times \cos(180 \times \alpha_1 \times p_{1def1} / \pi)) = 0.00$
Particular integral due to load	$\delta'_1 = D_{\alpha p1def1} / \alpha_1^3 \times N_{Edc1} / (I_{sp1} \times E_{sp1}) = 0.000 \text{ mm}$
Maximum deflection	$\delta_{max1} = A_{\alpha xdef1} \times \delta_{01} + B_{\alpha xdef1} \times \Phi_{01} / \alpha_1 + \delta'_1 = 0.439 \text{ mm}$
Location of maximum moment	$x_{M1} = 220 \text{ mm}$
Krilov's functions at the spreader length	$C_{\alpha xM1} = 1/2 \times \sinh(\alpha_1 \times x_{M1}) \times \sin(180 \times \alpha_1 \times x_{M1} / \pi) = 0.09$ $D_{\alpha xM1} = 1/4 \times (\cosh(\alpha_1 \times x_{M1}) \times \sin(180 \times \alpha_1 \times x_{M1} / \pi) - \sinh(\alpha_1 \times x_{M1}) \times \cos(180 \times \alpha_1 \times x_{M1} / \pi)) = 0.01$
Distance of point load right of loaction	$p_{1M1} = 0 \text{ mm}$

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Krilov's functions at the spreader length

$$B_{ap1M1} = 1/2 \times (\cosh(\alpha_1 \times p_{1M1}) \times \sin(180 \times \alpha_1 \times p_{1M1} / \pi) + \sinh(\alpha_1 \times p_{1M1}) \times \cos(180 \times \alpha_1 \times p_{1M1} / \pi)) = \mathbf{0.00}$$

Particular integral due to load

$$M'_1 = -B_{ap1M1} / \alpha_1 \times N_{Edc1} = \mathbf{0.00 \text{ kNm}}$$

Maximum moment

$$M_{Edsp1} = (4 \times \alpha_1^2 \times C_{\alpha xM1} \times \delta_{01} + 4 \times \alpha_1 \times D_{\alpha xM1} \times \Phi_{01}) \times (I_{sp1} \times E_{sp1}) + M'_1 = \mathbf{1.60 \text{ kNm}}$$

Location of maximum shear

$$x_{V1} = \mathbf{220 \text{ mm}}$$

Krilov's functions at the spreader length

$$B_{\alpha xV1} = 1/2 \times (\cosh(\alpha_1 \times x_{V1}) \times \sin(180 \times \alpha_1 \times x_{V1} / \pi) + \sinh(\alpha_1 \times x_{V1}) \times \cos(180 \times \alpha_1 \times x_{V1} / \pi)) = \mathbf{0.43}$$

$$C_{\alpha xV1} = 1/2 \times \sinh(\alpha_1 \times x_{V1}) \times \sin(180 \times \alpha_1 \times x_{V1} / \pi) = \mathbf{0.09}$$

Distance of point load right of loaction

$$p_{1V1} = \mathbf{0 \text{ mm}}$$

Krilov's functions at the spreader length

$$A_{ap1V1} = \cosh(\alpha_1 \times p_{1V1}) \times \cos(180 \times \alpha_1 \times p_{1V1} / \pi) = \mathbf{1.00}$$

Particular integral due to load

$$V'_1 = -A_{ap1V1} \times N_{Edc1} = \mathbf{-29.10 \text{ kN}}$$

Shear at concentrated point load

$$V_1 = (4 \times \alpha_1^3 \times B_{\alpha xV1} \times \delta_{01} + 4 \times \alpha_1^2 \times C_{\alpha xV1} \times \Phi_{01}) \times (I_{sp1} \times E_{sp1}) + V'_1 = \mathbf{-14.55 \text{ kN}}$$

Maximum shear

$$V_{Edsp1} = \text{Max}(\text{Abs}(V_1), N_{Edc1} - \text{Abs}(V_1)) = \mathbf{14.55 \text{ kN}}$$

Maximum allowable stress under spreader

$$\sigma_{Rdsp1} = 1.5 \times f_d = \mathbf{1.75 \text{ N/mm}^2}$$

Maximum reaction

$$N_{Edsp1} = K_{c1} \times \delta_{max1} = \mathbf{66.61 \text{ kN/m}}$$

Design stress

$$\sigma_{Edsp1} = 2 \times N_{Edsp1} / (3 \times (w_{sp1} / 2 - e_{sp1})) = \mathbf{1.48 \text{ N/mm}^2}$$

PASS - Design stress under spreader is less than the allowable bearing stress

Walls subjected to mainly vertical loading - Section6.1.2

Eccentricity of permanent UDL at mid-height below concentrated load

$$e_{gmu1} = e_{gu} \times h_{c1} / (2 \times h) = \mathbf{0.0 \text{ mm}}$$

Eccentricity of variable UDL at mid-height below concentrated load

$$e_{qmu1} = e_{qu} \times h_{c1} / (2 \times h) = \mathbf{0.0 \text{ mm}}$$

Eccentricity of concentrated load at mid-height

$$e_{mc1} = e_{c1} / 2 = \mathbf{10.0 \text{ mm}}$$

Initial eccentricity - cl.5.5.1.1(4)

$$e_{init} = h / 450 = \mathbf{5.1 \text{ mm}}$$

Concentrated load at mid-height as UDL

$$N_{mc1} = N_{Edc1} / l_{efm1} = \mathbf{19.92 \text{ kN/m}}$$

Vertical load at mid-height

$$N_{Ed1} = (g_k + \gamma \times t \times (h - h_{c1} / 2)) \times \gamma_{fG} + q_k \times \gamma_{fQ} + N_{mc1} = \mathbf{30.00 \text{ kN/m}}$$

Design moment at mid-height

$$M_{Ed1} = g_k \times \gamma_{fG} \times e_{gmu1} + q_k \times \gamma_{fQ} \times e_{qmu1} + N_{mc1} \times e_{mc1} = \mathbf{0.20 \text{ kNm/m}}$$

Eccentricities due to loads - eq. 6.7

$$e_{m1} = \text{Abs}(M_{Ed1}) / N_{Ed1} + e_{init} = \mathbf{11.8 \text{ mm}}$$

Slenderness ratio limit for creep eccentricity

$$\lambda_c = \mathbf{27}$$

Eccentricity due to creep

$$e_{k1} = \mathbf{0.0 \text{ mm}}$$

Eccentricity at mid-height - eq. 6.6

$$e_{mk1} = \text{Max}(e_{m1} + e_{k1}, 0.05 \times t) = \mathbf{11.8 \text{ mm}}$$

From eq. G2

$$A_{11} = 1 - 2 \times e_{mk1} / t = \mathbf{0.76}$$

From eq. G3

$$u_1 = (h_{ef} / t_{ef} \times (1 / K_E)^{1/2} - 0.063) / (0.73 - 1.17 \times e_{mk1} / t) = \mathbf{0.88}$$

Capacity reduction factor - eq. G1

$$\Phi_{m1} = A_{11} \times \exp(-(u_1^2) / 2) = \mathbf{0.52}$$

Design vertical resistance of panel - eq.6.2

$$N_{Rd1} = \Phi_{m1} \times t \times f_d = \mathbf{60.68 \text{ kN/m}}$$

PASS - Design value of vertical resistance exceeds applied vertical load

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MASONRY BEARING DESIGN (EN1996)

MASONRY BEARING DESIGN

In accordance with EN1996-1-1:2005 + A1:2012, incorporating Corrigenda February 2006 and July 2009 and the UK National Annex.

Tedds calculation version 1.0.07

Masonry panel details

Panel length	L = 2000 mm
Panel height	h = 2300 mm
Thickness of load bearing leaf	t = 100 mm
Effective height	h_{ef} = 2300 mm
Effective thickness	t_{ef} = 100 mm

Masonry material details

Unit type	Aggregate concrete - Group 1
Compressive strength of masonry unit	f_c = 3.6 N/mm²
Height of unit	h_u = 215 mm
Width of unit	w_u = 100 mm
Conditioning factor	k = 1.0
- Conditioning to the air dry condition in accordance with cl.7.3.2	
Shape factor - Table A.1	d_{sf} = 1.38
Mean compressive strength of masonry unit	f_b = f_c × k × d_{sf} = 4.97 N/mm²
Specific weight of units	γ = 18 kN/m³
Mortar type	M4 - General Purpose
Compressive strength of mortar	f_m = 4.0 N/mm²
Compressive strength factor - Tbl. NA 4	K = 0.75
Characteristic compressive strength of the masonry - eq. 3.1	f_k = K × f_b^{0.7} × f_m^{0.3} = 3.49 N/mm²
Short term secant modulus of elasticity factor	K_E = 1000
Modulus of elasticity - cl.3.7.2	E_w = K_E × f_k = 3491 N/mm²

Design compressive strength of masonry

Category of manufacturing control	Category II
Class of execution control	Class 2
Partial factor for material strength in direct or flexural compression	

$$\gamma_M = 3.00$$

Cross-sectional area of wall	A = L × t = 0.20 m²
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Design compressive strength of masonry	f_d = f_k / γ_M = 1.16 N/mm²
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Partial safety factors for design loads

Partial safety factor for permanent load	γ_{fG} = 1.35
Partial safety factor for variable load	γ_{fQ} = 1.50

Superimposed vertical loading details

Permanent UDL at top of wall	g_k = 5.40 kN/m
Variable UDL at top of wall	q_k = 0.00 kN/m
Eccentricity of permanent UDL load	e_{gu} = 0 mm
Eccentricity of variable UDL load	e_{qu} = 0 mm



Slenderness ratio of masonry wall - Section 5.5.1.4

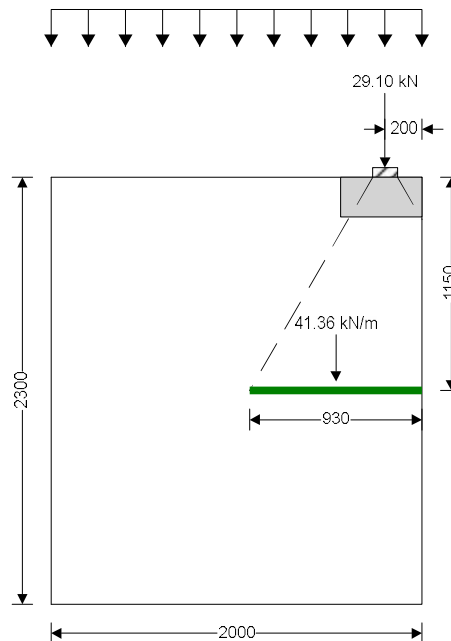
Slenderness ratio limit $\lambda_{lim} = 27$

Slenderness ratio $\lambda = h_{ef} / t_{ef} = \mathbf{23.0}$

PASS - Slenderness ratio is less than slenderness limit

Concentrated Load 1 details - steel beam on internal wall

7.29 kN/m



Permanent concentrated load $G_{kC1} = 6.00 \text{ kN}$

Variable concentrated load $Q_{kc1} = 14.00 \text{ kN}$

Eccentricity of concentrated load $e_{c1} = 15 \text{ mm}$

Length of concentrated load $L_{c1} = 133 \text{ mm}$

Width of concentrated load $w_{c1} = \mathbf{100 \text{ mm}}$

Height of concentrated load $h_{c1} = 2300 \text{ mm}$

Distance of load to right vertical edge $r_{11} = 134 \text{ mm}$ Distance of load to nearest vertical edge $a_{11} = 134 \text{ mm}$

Walls subjected to concentrated loads - Section 6.1.3

Eccentricity check $e_{c1} \leq t / 4$

PASS - Eccentricity of load is less than $t/4$

Area of bearing $A_{b1} = L_{c1} \times w_{c1} = \mathbf{13300 \text{ mm}^2}$

Effective length of bearing at mid-height $l_{efm1} = L_{c1} + h_{c1} / 2 \times \tan(30) + r_{11} = \mathbf{930 \text{ mm}}$

Effective bearing area $A_{ef1} = l_{efm1} \times t = \mathbf{93045 \text{ mm}^2}$

Bearing area ratio check $A_{ratio1} = \text{Min}(A_{b1} / A_{eff1}, 0.45) = \mathbf{0.14}$

Initial enhancement factor $\beta_{init1} = \text{Max}((1 + 0.3 \times a_{11} / h_{c1}) \times (1.5 - 1.1 \times A_{ratio1}), 1.0) = \mathbf{1.37}$

Maximum enhancement factor $\beta_{\max 1} = \text{Min}(1.25 + a_{11} / (2 \times h_{c1}), 1.5) = \mathbf{1.28}$

Enhancement factor for concentrated loads $\beta_1 = \text{Min}(\beta_{\text{init}1}, \beta_{\text{max}1}) = \mathbf{1.28}$

$$\text{Design value of the concentrated load } N_{\text{Edc1}} = G_{\text{kc1}} \times \gamma_{\text{FG}} + O_{\text{kc1}} \times \gamma_{\text{FO}} = \mathbf{29.10 \text{ kN}}$$
$$N_{Rdc1} = \beta_1 \times A_{b1} \times f_d = \mathbf{19.80 \text{ kN}}$$

Applied concentrated load exceeds design resistance, spreader required!

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Design of spreader beam

Type of spreader
Type of bearing onto spreader
Location of load from RHS of spreader
Length of spreader
Height of spreader
Width of spreader
Eccentricity of load on spreader
Modulus of elasticity
Second moment of area
Modulus of the wall
Winkler's constant
Characteristic of the system
Classification of spreader
Krilov's functions for the spreader length

Concrete padstone

Point load

$P_{11} = 200$ mm
 $L_{sp1} = 440$ mm
 $h_{sp1} = 215$ mm
 $w_{sp1} = 100$ mm
 $e_{sp1} = 15$ mm
 $E_{sp1} = 29962$ N/mm²
 $I_{sp1} = 1/12 \times w_{sp1} \times h_{sp1}^3 = 82819792$ mm⁴
 $k_0 = E_w / h = 1.52$ N/mm²/mm
 $K_{c1} = k_0 \times w_{sp1} = 151.80$ N/mm/mm
 $\alpha_1 = (K_{c1} / (4 \times E_{sp1} \times I_{sp1}))^{1/4} = 0.00198$ mm⁻¹
 $\alpha L_1 = \alpha_1 \times L_{sp1} = 0.87$ Medium
 $B_{\alpha 11} = 1/2 \times (\cosh(\alpha L_1) \times \sin(180 \times \alpha L_1 / \pi) + \sinh(\alpha L_1) \times \cos(180 \times \alpha L_1 / \pi)) = 0.85$
 $C_{\alpha 11} = 1/2 \times \sinh(\alpha L_1) \times \sin(180 \times \alpha L_1 / \pi) = 0.38$
 $D_{\alpha 11} = 1/4 \times (\cosh(\alpha L_1) \times \sin(180 \times \alpha L_1 / \pi) - \sinh(\alpha L_1) \times \cos(180 \times \alpha L_1 / \pi)) = 0.11$
 $A_{\alpha P11} = \cosh(\alpha_1 \times P_{11}) \times \cos(180 \times \alpha_1 \times P_{11} / \pi) = 1.00$
 $B_{\alpha P11} = 1/2 \times (\cosh(\alpha_1 \times P_{11}) \times \sin(180 \times \alpha_1 \times P_{11} / \pi) + \sinh(\alpha_1 \times P_{11}) \times \cos(180 \times \alpha_1 \times P_{11} / \pi)) = 0.40$
 $M_{01} = 0$ kNm
 $V_{01} = 0$ kN
 $(4 \times \alpha_1^2 \times C_{\alpha 11} \times \delta_{01} + 4 \times \alpha_1 \times D_{\alpha 11} \times \Phi_{01}) \times E_{sp1} \times I_{sp1} - B_{\alpha P11} / \alpha_1 \times N_{Edc1} = 0.00$ kNm
 $(4 \times \alpha_1^3 \times B_{\alpha 11} \times \delta_{01} + 4 \times \alpha_1^2 \times C_{\alpha 11} \times \Phi_{01}) \times E_{sp1} \times I_{sp1} - A_{\alpha P11} \times N_{Edc1} = 0.00$ kN
Therefore,
Initial deflection of LH edge
Initial rotation of LH edge

Location of maximum deflection
Krilov's functions at the spreader length

Distance of point load right of loaction
Krilov's functions at the spreader length

Particular integral due to load
Maximum deflection

Location of maximum moment
Krilov's functions at the spreader length

$x_{def1} = 440$ mm
 $A_{\alpha xdef1} = \cosh(\alpha_1 \times x_{def1}) \times \cos(180 \times \alpha_1 \times x_{def1} / \pi) = 0.90$
 $B_{\alpha xdef1} = 1/2 \times (\cosh(\alpha_1 \times x_{def1}) \times \sin(180 \times \alpha_1 \times x_{def1} / \pi) + \sinh(\alpha_1 \times x_{def1}) \times \cos(180 \times \alpha_1 \times x_{def1} / \pi)) = 0.85$
 $p_{1def1} = 200$ mm
 $D_{\alpha p1def1} = 1/4 \times (\cosh(\alpha_1 \times p_{1def1}) \times \sin(180 \times \alpha_1 \times p_{1def1} / \pi) - \sinh(\alpha_1 \times p_{1def1}) \times \cos(180 \times \alpha_1 \times p_{1def1} / \pi)) = 0.01$
 $\delta'_1 = D_{\alpha p1def1} / \alpha_1^3 \times N_{Edc1} / (I_{sp1} \times E_{sp1}) = 0.016$ mm
 $\delta_{max1} = A_{\alpha xdef1} \times \delta_{01} + B_{\alpha xdef1} \times \Phi_{01} / \alpha_1 + \delta'_1 = 0.550$ mm
 $x_{M1} = 240$ mm
 $C_{\alpha xM1} = 1/2 \times \sinh(\alpha_1 \times x_{M1}) \times \sin(180 \times \alpha_1 \times x_{M1} / \pi) = 0.11$
 $D_{\alpha xM1} = 1/4 \times (\cosh(\alpha_1 \times x_{M1}) \times \sin(180 \times \alpha_1 \times x_{M1} / \pi) - \sinh(\alpha_1 \times x_{M1}) \times \cos(180 \times \alpha_1 \times x_{M1} / \pi)) = 0.02$

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Distance of point load right of loaction	$p_{1M1} = 0 \text{ mm}$
Krilov's functions at the spreader length	$B_{ap1M1} = 1/2 \times (\cosh(\alpha_1 \times p_{1M1}) \times \sin(180 \times \alpha_1 \times p_{1M1} / \pi) + \sinh(\alpha_1 \times p_{1M1}) \times \cos(180 \times \alpha_1 \times p_{1M1} / \pi)) = 0.00$
Particular integral due to load	$M'_1 = -B_{ap1M1} / \alpha_1 \times N_{Edc1} = 0.00 \text{ kNm}$
Maximum moment	$M_{Edsp1} = (4 \times \alpha_1^2 \times C_{axM1} \times \delta_{01} + 4 \times \alpha_1 \times D_{axM1} \times \Phi_{01}) \times (I_{sp1} \times E_{sp1}) + M'_1 = 1.57 \text{ kNm}$
Location of maximum shear	$x_{V1} = 240 \text{ mm}$
Krilov's functions at the spreader length	$B_{axV1} = 1/2 \times (\cosh(\alpha_1 \times x_{V1}) \times \sin(180 \times \alpha_1 \times x_{V1} / \pi) + \sinh(\alpha_1 \times x_{V1}) \times \cos(180 \times \alpha_1 \times x_{V1} / \pi)) = 0.47$ $C_{axV1} = 1/2 \times \sinh(\alpha_1 \times x_{V1}) \times \sin(180 \times \alpha_1 \times x_{V1} / \pi) = 0.11$
Distance of point load right of loaction	$p_{1V1} = 0 \text{ mm}$
Krilov's functions at the spreader length	$A_{ap1V1} = \cosh(\alpha_1 \times p_{1V1}) \times \cos(180 \times \alpha_1 \times p_{1V1} / \pi) = 1.00$
Particular integral due to load	$V'_1 = -A_{ap1V1} \times N_{Edc1} = -29.10 \text{ kN}$
Shear at concentrated point load	$V_1 = (4 \times \alpha_1^3 \times B_{axV1} \times \delta_{01} + 4 \times \alpha_1^2 \times C_{axV1} \times \Phi_{01}) \times (I_{sp1} \times E_{sp1}) + V'_1 = -15.19 \text{ kN}$
Maximum shear	$V_{Edsp1} = \text{Max}(\text{Abs}(V_1), N_{Edc1} - \text{Abs}(V_1)) = 15.19 \text{ kN}$
Maximum allowable stress under spreader	$\sigma_{Rdsp1} = 1.5 \times f_d = 1.75 \text{ N/mm}^2$
Maximum reaction	$N_{Edsp1} = K_{c1} \times \delta_{max1} = 83.45 \text{ kN/m}$
Design stress	$\sigma_{Edsp1} = N_{Edsp1} / W_{sp1} \times (1 + 6 \times e_{sp1} / W_{sp1}) = 1.59 \text{ N/mm}^2$

PASS - Design stress under spreader is less than the allowable bearing stress

Walls subjected to mainly vertical loading - Section6.1.2

Eccentricity of permanent UDL at mid-height below concentrated load

$$e_{gmu1} = e_{gu} \times h_{c1} / (2 \times h) = 0.0 \text{ mm}$$

Eccentricity of variable UDL at mid-height below concentrated load

$$e_{qmu1} = e_{qu} \times h_{c1} / (2 \times h) = 0.0 \text{ mm}$$

Eccentricity of concentrated load at mid-height

$$e_{mc1} = e_{c1} / 2 = 7.5 \text{ mm}$$

Initial eccentricity - cl.5.5.1.1(4)

$$e_{init} = h / 450 = 5.1 \text{ mm}$$

Concentrated load at mid-height as UDL

$$N_{mc1} = N_{Edc1} / l_{efm1} = 31.28 \text{ kN/m}$$

Vertical load at mid-height

$$N_{Ed1} = (g_k + \gamma \times t \times (h - h_{c1} / 2)) \times \gamma_{fG} + q_k \times \gamma_{fQ} + N_{mc1} = 41.36 \text{ kN/m}$$

Design moment at mid-height

$$M_{Ed1} = g_k \times \gamma_{fG} \times e_{gmu1} + q_k \times \gamma_{fQ} \times e_{qmu1} + N_{mc1} \times e_{mc1} = 0.23 \text{ kNm/m}$$

Eccentricities due to loads - eq. 6.7

$$e_{m1} = \text{Abs}(M_{Ed1}) / N_{Ed1} + e_{init} = 10.8 \text{ mm}$$

Slenderness ratio limit for creep eccentricity

$$\lambda_c = 27$$

Eccentricity due to creep

$$e_{k1} = 0.0 \text{ mm}$$

Eccentricity at mid-height - eq. 6.6

$$e_{mk1} = \text{Max}(e_{m1} + e_{k1}, 0.05 \times t) = 10.8 \text{ mm}$$

From eq. G2

$$A_{11} = 1 - 2 \times e_{mk1} / t = 0.78$$

From eq. G3

$$u_1 = (h_{ef} / t_{ef} \times (1 / K_E)^{1/2} - 0.063) / (0.73 - 1.17 \times e_{mk1} / t) = 1.10$$

Capacity reduction factor - eq. G1

$$\Phi_{m1} = A_{11} \times \exp(-(u_1^2) / 2) = 0.43$$

Design vertical resistance of panel - eq.6.2

$$N_{Rd1} = \Phi_{m1} \times t \times f_d = 49.84 \text{ kN/m}$$

PASS - Design value of vertical resistance exceeds applied vertical load