Canham Consulting

Structural Civil Building

Engineers

Structural Calculations

Project Ref:	**220007-CCL-XX-XX-CA-S-0100-P02**
Site:	**Great Plumstead Pavilion, Church Road, Great Plumstead, Norwich, NR13 5AB**
Client:	**Great and Little Plumstead Parish Council**
Project Overview:	**New single storey pavilion building.**
Prepared By:	**Jack Powell BEng (Hons) CEng MIStructE**
Checked By:	**Nigel Evans IEng IMIStructE C.Build E MCABE**
Revision History:	**May 2024 – First Issue**
	June 2024 – Second Issue

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Design Information

Project Overview	New single storey pavilion building.

Intended Building Usage	Leisure

Subsoil Conditions	From BGS website assumed to be Happisburgh Glacigenic Formation - diamicton. Assumed allowable ground bearing pressure of 100kN/m^2 TBC on site prior to pouring concrete.	**S.I Report by**	None

Specialist Design by Others	Glulam roof structure and ICF walls.		
Relevant British Standards, Codes of Practice and Design Standards Used In This Project			
BS 648:1964 - Weights of Materials	☒	**BS 5268 Pt 2:1996 -** Timber	☒
BS 6399 Pt 1:1996 - Building Loads	☒	**BS 5628 Pt 1: 1992 -** Masonry	☒
BS 6399 Pt 2:2002 - Wind Loading	☒	**Building Regs Approved Doc. A: 2004 edition incorporating 2004, 2010 and 2013 amendments.**	☒
BS 6399 Pt 3:1988 - Snow Loading	☒		
BS 5950 Pt 1:2000 - Steelwork	☒	**NHBC Standards**	☒
BS 8110 Pt 1:1997 - Concrete	☒	**Product and Material Data Sheets**	☒

CDM 2015 Design Considerations	**Principal Designer:**	Others		
This design has been considered under CDM 2015 - to eliminate, reduce or control foreseeable risks that may arise during construction, and the maintenance and use of a building once it is built.				
***Risks -** Consider those difficult to manage, unusual, or not likely to be obvious to a suitably-experienced contractor or designer*				
Area of Hazard	**Assessed** (Yes | No | N/A)	**Method of Hazard Elimination/Reduction**	**Residual Risk**	
			Risk	Significant
Access to work area	Yes	To be arranged by Client	☒	☐
Demolition	Yes		☒	☐
Services	Yes	Contractor to identify services prior to construction	☒	☐
Installation	Yes	Design elements as light as possible. Deliver elements as close as possible to installation point. Use mechanical plant for lifting were possible	☒	☐
Excavation	Yes	Contractor to provide all req'd shoring and propping	☒	☐
Sequencing	Yes	Contractor's choice	☒	☐
Falls from height	Yes	Contractor to provide scaffold, fall arrest and edge protection where required	☒	☐
Post-tensioning/ pre-tensioning	No		☐	☐
Materials	Yes	Contractor to adopt safe working practices. Wear PPE to prevent contact with hazardous materials.	☒	☐
Hotworks	Yes	Prefabricate elements offsite where possible	☒	☐
Future Maintenance	No		☐	☐
Final Demolition	No		☐	☐
Does this project require a separate Designers Risk Assessment?			Yes ☐	No ☒

Client and Contractor Notices

The following calculations and details should be read in conjunction with any available architectural drawings.

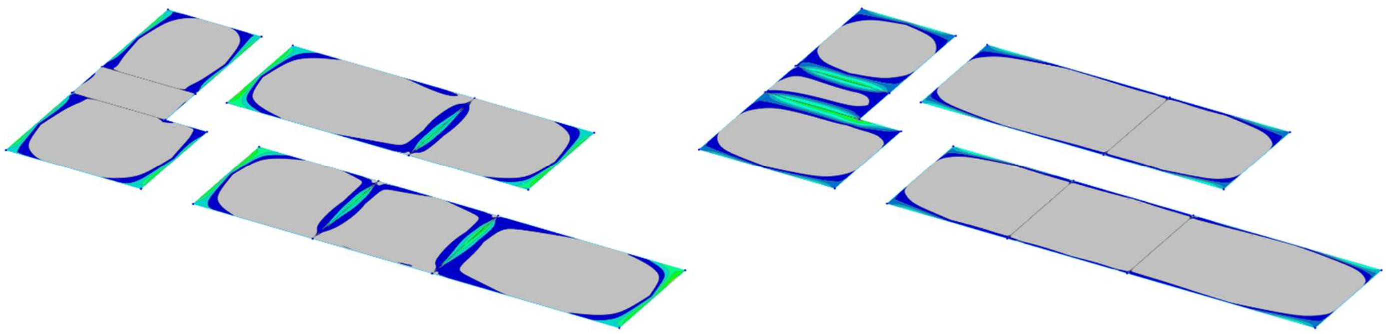
All dimensions within these calculations are for design purposes only. These are used in place of site derived dimensions and may vary from actual dimensions of structural members. The design dimensions must not be used for the procurement of any structural elements. Accurate on-site measurements for overall element sizes must be carried out prior to fabrication of any structural elements, by the contractor or specialist supplier. This must include the bearing lengths specified within this calculations package.

Design Statement

Canham Consulting (CCL) have been appointed to provide structural calculations in relation to the proposed works at the above address.

Within this calculation package are structural calculations for the steel and concrete beams over the window and door openings to the external walls, and foundation raft slabs.

The raft slabs have been analysed and designed in Masterseries using finite element analysis. The slabs have been modelled as 200mm thick with 450mm wide x 250mm thick thickenings. The ground support has been modelled as an area spring stiffness equivalent to 50kN/m².



Where additional reinforcement is required across the thickenings, this has been provided with additional loose bars alongside the mesh, lapping a minimum of 1.5m to ensure the minimum area of steel is provided. This applied in both directions.

The edge thickenings have also been designed to span over a 2m soft spot with additional reinforcement in the bottom of the thickening.

It should be noted that the roof structures are to be independent of each other whilst giving the appearance of a single structure.

Please refer to architectural drawings by David Bullen Limited, and the following structural calculations and CCL drawing numbers:

- 220007-CCL-XX-XX-DR-S-0600
- 220007-CCL-XX-XX-DR-S-0700
- 220007-CCL-XX-XX-DR-S-0701
- 220007-CCL-XX-XX-DR-S-0702
- 220007-CCL-XX-XX-DR-S-1000
- 220007-CCL-XX-XX-SH-S-0701

Contents

Loading Schedule:

Vaulted Roof		
PV panels =	0.20	
Metal profile roofing =	0.10	
Timber cross members =	0.05	
Glulam rafters =	0.10	
22mm OSB =	0.20	
Insulation =	0.05	
Plaster & Skim =	0.18	
	0.88	
Roof Angle =	35	
Plan Dead =	**1.07**	**kN/m^2**
Plan Live (No access) =	**0.50**	**kN/m^2**

ICF Walls		
Plaster & Skim =	0.18	
140mm Concrete =	3.50	
Insulation =	0.05	
Facing Brick =	2.25	
Plan Dead =	**5.98**	**kN/m^2**

Internal Walls		
Plaster & Skim =	0.36	
100mm Blockwork =	1.50	
Plan Dead =	**1.86**	**kN/m^2**

Raft Slab		
150mm Thk slab =	3.60	
Plan Dead =	**0.00**	**kN/m^2**
Plan Live =	**1.50**	**kN/m^2**

Design of Concrete Lintel within ICF Walls.

Maximum span of lintel required: $L := 2.50\ m$

Try section size: Width: $b := 140\ mm$

Depth: $t := 200\ mm$

Assume global cover: $c := 50\ mm$

Concrete strength: $f_{cu} := 35\ \frac{N}{mm^2}$

Main reinforcement yield strength: $f_y := 500\ \frac{N}{mm^2}$

<u>Loading:</u> <u>Dead Loads</u> <u>Live Loads</u>

Vaulted roof: $G_{k2} := 1.07\ \frac{kN}{m^2} \cdot \frac{12.0\ m}{2} = 6.42\ \frac{kN}{m}$ $Q_{k2} := 0.50\ \frac{kN}{m^2} \cdot \frac{12.0\ m}{2} = 3.00\ \frac{kN}{m}$

Wall: $G_{k9} := 5.98\ \frac{kN}{m^2} \cdot 0.50\ m = 2.99\ \frac{kN}{m}$

S/W: $G_{k10} := 25\ \frac{kN}{m^3} \cdot b \cdot t = 0.70\ \frac{kN}{m}$

Total Dead Load: $G_k := \sum G_k$ $G_k = 10.11\ \frac{kN}{m}$

Total Live Load: $Q_k := \sum Q_k$ $Q_k = 3.00\ \frac{kN}{m}$

Ultimate UDL: $w_u := \left(G_k \cdot 1.4 + Q_k \cdot 1.6\right)$ $w_u = 18.95\ \frac{kN}{m}$

Service UDL: $w_s := \left(G_k + Q_k\right)$ $w_s = 13.11\ \frac{kN}{m}$

<u>Check section for Bending:</u>

Moment: $M_s := \frac{w_u \cdot L^2}{8}$ $M_s = 14.81\ kN \cdot m$

<u>Bottom Reinforcement:</u>

Moment: $M_s = 14.81\ kN \cdot m$

Assumed reinforcement sizes for effective depth:

Main bar diameter: $\phi_{b;Lintel} = 16.00\ mm$

Effective depth: $d_b := t - c - \frac{\phi_{b;Lintel}}{2} = 158.00\ mm$ $d_b = 158.00\ mm$

$k := \frac{M_s}{f_{cu} \cdot b \cdot d_b^2}$ $k = 0.12$

$k_{check1} = \text{“k < 0.156”}$ Therefore, $k_{check2} = \text{“no compression steel required.”}$

$Z_{check1} = \text{“Z < 0.95 x d, therefore Z = d x (0.5 + (0.25 – k/0.9)^0.5)”}$ $Z = 132.70\ mm$

Area of steel required: $As_{req;b} := \dfrac{M_s}{0.87 \cdot f_y \cdot Z}$ $As_{req;b} = 256.53\ mm^2$

Try: Main bar diameter: $\phi_{b;Lintel} \equiv 16\ mm$

Number of main bars: $N_b := 2$

Area of steel provided: $As_{prov;b} := N_b \cdot \pi \cdot \left(\dfrac{\phi_{b;Lintel}}{2}\right)^2$ $As_{prov;b} = 402.12\ mm^2$

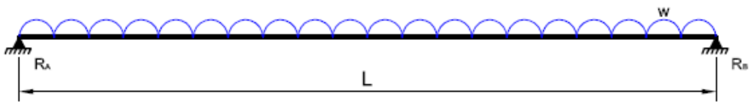
Utilisation ratios:

Bottom bars: $As_{prov;b} = 402.12\ mm^2$ > $As_{req;b} = 256.53\ mm^2$ $U_b := \dfrac{As_{prov;b}}{As_{req;b}} = 1.57$ $Steel\_Area = \text{“OK”}$

Min. area: $As_{prov;b} = 402.12\ mm^2$ > $A_{smin;req} = 36.40\ mm^2$ $U_m := \dfrac{As_{prov;b}}{A_{smin;req}} = 11.05$ $Min\_Area = \text{“OK”}$

Therefore provide minimum of 140mm wide x 200mm deep concrete (C28/35) lintel over openings, with 2H16 bars in the bottom, 50mm cover.

Design of Steel Beam over Gable Opening - Ref B1.



Span between supports: $L := 5.0\ m$

Loading:	Dead Loads	Live Loads
Vaulted roof:	$G_{k2} := 1.07\ \frac{kN}{m^2} \cdot 1.25\ m = 1.34\ \frac{kN}{m}$	$Q_{k2} := 0.50\ \frac{kN}{m^2} \cdot 1.25\ m = 0.63\ \frac{kN}{m}$
Wall:	$G_{k9} := 5.98\ \frac{kN}{m^2} \cdot \frac{2.50\ m}{2} = 7.48\ \frac{kN}{m}$	
S/W:	$G_{k10} := 0.43\ \frac{kN}{m} = 0.43\ \frac{kN}{m}$	

Total Dead Load: $G_k := \sum G_k$ $\qquad G_k = 9.24\ \frac{kN}{m}$

Total Live Load: $Q_k := \sum Q_k$ $\qquad Q_k = 0.63\ \frac{kN}{m}$

Ultimate UDL: $w_u := \left(G_k \cdot 1.4 + Q_k \cdot 1.6\right)$ $\qquad w_u = 13.94\ \frac{kN}{m}$

Service UDL: $w_s := \left(G_k + Q_k\right)$ $\qquad w_s = 9.87\ \frac{kN}{m}$

Check section for Bending:

Moment: $M := \frac{w_u \cdot L^2}{8}$ $\qquad M = 43.56\ kN \cdot m$

Check section for Deflection:

Young's Modulus: $E := 205000\ \frac{N}{mm^2}$

Limiting Deflection: Max 12mm for brittle finishes: $\delta := min\left(12\ mm, \frac{L}{500}\right)$ $\qquad \delta = 10.00\ mm$

Ixx Required: $I_{xxreq} := \frac{5 \cdot w_s \cdot L^4}{384 \cdot E \cdot \delta}$ $\qquad I_{xxreq} = 3917.17\ cm^4$

From TATA Steel "Bluebook" try: $Beam :=$ 254 x 146 x 43 $\qquad Effective\_Length :=$ 1.4L +2D

Moment capacity: $M_c = 201.00\ kN \cdot m$

Section: $D = 259.6\ mm$ $\quad B = 147.3\ mm$ $\quad$ Span to depth ratio: $\lambda := \frac{L}{D}$ $\qquad \lambda = 19.26$

Effective Length: $L_E = 7.52\ m$

Buckling capacity: $M_b = 58.31\ kN \cdot m$

Second Moment of Area: $I_{xx} = 6540.00\ cm^4$

Utilisation ratios:

Bending: $M_c = 201.00\ kN \cdot m$ > $M = 43.56\ kN \cdot m$ $U_b := \frac{M}{M_c} = 0.22$ $Bending = \text{"OK"}$

Buckling: $M_b = 58.31\ kN \cdot m$ > $M = 43.56\ kN \cdot m$ $U_b := \frac{M}{M_b} = 0.75$ $Buckling = \text{"OK"}$

Deflection: $I_{xx} = 6540.00\ cm^4$ > $I_{xxreq} = 3917.17\ cm^4$ $U_d := \frac{I_{xxreq}}{I_{xx}} = 0.60$ $Deflection = \text{"OK"}$

Therefore provide $Beam = \text{"254 x 146 x 43"}$ UB.

Reactions:

Ultimate: $R_{A;uls;B1} := \frac{w_u \cdot L}{2} = 34.85\ kN$ Dead: $R_{A;Gk;B1} := \frac{G_k \cdot L}{2} = 23.11\ kN$

Service: $R_{A;sls;B1} := \frac{w_s \cdot L}{2} = 24.67\ kN$ Live: $R_{A;Qk;B1} := \frac{Q_k \cdot L}{2} = 1.56\ kN$

Beam Bearings

Beam reaction: $F := R_{A;uls;B1}$ $F = 34.85\ kN$

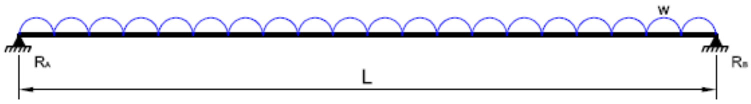
Masonry material: $\sigma_c :=$ Concrete $= 35.00\ \frac{N}{mm^2}$ Material Safety Factor: $\gamma_m := 3.50$

Bearing length of beam: $b_{bA} := 250\ mm$

Applied Bearing Stress: $\sigma_a := \frac{F}{t \cdot b_{bA}}$ $\sigma_a = 0.70\ \frac{N}{mm^2}$ $Bearing\_Stress = \text{"OK"}$

Therefore beam to have minimum 250mm end bearing on to concrete core.

Design of Steel Beam over Gable Opening - Ref B2.



Span between supports: $L := 4.5\ m$

<u>Loading:</u> <u>Dead Loads</u> <u>Live Loads</u>

Vaulted roof: $G_{k2} := 1.07\ \frac{kN}{m^2} \cdot \frac{12.0\ m}{2} = 6.42\ \frac{kN}{m}$ $Q_{k2} := 0.50\ \frac{kN}{m^2} \cdot \frac{12.0\ m}{2} = 3.00\ \frac{kN}{m}$

Wall: $G_{k9} := 5.98\ \frac{kN}{m^2} \cdot 0.50\ m = 2.99\ \frac{kN}{m}$

S/W: $G_{k10} := 0.46\ \frac{kN}{m} = 0.46\ \frac{kN}{m}$

Total Dead Load: $G_k := \sum G_k$ $G_k = 9.87\ \frac{kN}{m}$

Total Live Load: $Q_k := \sum Q_k$ $Q_k = 3.00\ \frac{kN}{m}$

Ultimate UDL: $w_u := \left(G_k \cdot 1.4 + Q_k \cdot 1.6\right)$ $w_u = 18.62\ \frac{kN}{m}$

Service UDL: $w_s := \left(G_k + Q_k\right)$ $w_s = 12.87\ \frac{kN}{m}$

<u>Check section for Bending:</u>

Moment: $M := \frac{w_u \cdot L^2}{8}$ $M = 47.13\ kN \cdot m$

<u>Check section for Deflection:</u>

Young's Modulus: $E := 205000\ \frac{N}{mm^2}$

Limiting Deflection: Max 12mm for brittle finishes: $\delta := min\left(12\ mm, \frac{L}{1000}\right)$ $\delta = 4.50\ mm$

Ixx Required: $I_{xxreq} := \frac{5 \cdot w_s \cdot L^4}{384 \cdot E \cdot \delta}$ $I_{xxreq} = 7449.05\ cm^4$

<u>From TATA Steel "Bluebook" try:</u> $Beam :=$ 305 x 165 x 46 $Effective\_Length :=$ 1.4L +2D

Moment capacity: $M_c = 256.00\ kN \cdot m$

Section: $D = 306.6\ mm$ $B = 165.7\ mm$ Span to depth ratio: $\lambda := \frac{L}{D}$ $\lambda = 14.68$

Effective Length: $L_E = 6.91\ m$

Buckling capacity: $M_b = 76.39\ kN \cdot m$

Second Moment of Area: $I_{xx} = 9900.00\ cm^4$

Utilisation ratios:

Bending: $M_c = 256.00\ kN \cdot m$ > $M = 47.13\ kN \cdot m$ $U_b := \frac{M}{M_c} = 0.18$ $Bending = \text{"OK"}$

Buckling: $M_b = 76.39\ kN \cdot m$ > $M = 47.13\ kN \cdot m$ $U_b := \frac{M}{M_b} = 0.62$ $Buckling = \text{"OK"}$

Deflection: $I_{xx} = 9900.00\ cm^4$ > $I_{xxreq} = 7449.05\ cm^4$ $U_d := \frac{I_{xxreq}}{I_{xx}} = 0.75$ $Deflection = \text{"OK"}$

Therefore provide $Beam = \text{"305 x 165 x 46"}$ UB.

Reactions:

Ultimate: $R_{A;uls;B2} := \frac{w_u \cdot L}{2} = 41.89\ kN$ Dead: $R_{A;Gk;B2} := \frac{G_k \cdot L}{2} = 22.21\ kN$

Service: $R_{A;sls;B2} := \frac{w_s \cdot L}{2} = 28.96\ kN$ Live: $R_{A;Qk;B2} := \frac{Q_k \cdot L}{2} = 6.75\ kN$

Beam Bearings

Beam reaction: $F := R_{A;uls;B2}$ $F = 41.89\ kN$

Masonry material: $\sigma_c :=$ Concrete $= 35.00\ \frac{N}{mm^2}$ Material Safety Factor: $\gamma_m := 3.50$

Bearing length of beam: $b_{bA} := 350\ mm$

Applied Bearing Stress: $\sigma_a := \frac{F}{t \cdot b_{bA}}$ $\sigma_a = 0.60\ \frac{N}{mm^2}$ $Bearing\_Stress = \text{"OK"}$

Therefore beam to have minimum 350mm end bearing on to concrete core.

Design of New Steel Column to B2 - Ref C1.

Height: $h := 3.00\ m$

Vertical Loading: | Ultimate Loads | Service Loads

Beam 2: $P_{u1} := R_{A;uls;B2} = 41.89\ kN$ $P_{s1} := R_{A;sls;B2} = 28.96\ kN$

Beam 2: $P_{u2} := R_{A;uls;B2} = 41.89\ kN$ $P_{s2} := R_{A;sls;B2} = 28.96\ kN$

Total Ultimate Load: $P_u := \sum P_u$ $P_u = 83.78\ kN$

Total Service Load: $P_s := \sum P_s$ $P_s = 57.92\ kN$

Consider section for for local capacity check and overall buckling check simplified approach:

Load eccentricity: $e_x := \frac{120\ mm}{2}$ $e_x = 60.00\ mm$

Moment: $M_x := e_x \cdot P_u$ $M_x = 5.03\ kN \cdot m$

Load eccentricity: $e_y := 100\ mm + \frac{80\ mm}{2}$ $e_y = 140.00\ mm$

Moment: $M_y := e_y \cdot P_{u1}$ $M_y = 5.86\ kN \cdot m$

Equivalent uniform moment factors: (Pinned based with nominal fixity at head β=0) $m_x := 1.00$ $m_y := 1.00$
(1.0 for nominal moments)

From TATA Steel "Bluebook": 120x80x5 RHS

Effective Height: $H_E := 1.0 \cdot h$ $H_E = 3.00\ m$

Buckling axial capacity Y - Y axis: $P_{cy} := 355\ kN$

Buckling Resistance: $M_{bs} := 17.1\ kN \cdot m$

Moment Capacity Y - Y axis: $M_{cy} := 19.9\ kN \cdot m$

Utilisation ratios:

Overall buckling check: $U_B := \frac{P_u}{P_{cy}} + \frac{m_x \cdot M_x}{M_{bs}} + \frac{m_y \cdot M_y}{M_{cy}} = 0.82$ $Buckling\_Check = \text{"OK"}$

Therefore provide 120 x 80 x 5.0 RHS S355 Column.

Reactions: Ultimate: $R_{A;uls;C1} := P_u$ $R_{A;uls;C1} = 83.78\ kN$

Service: $R_{A;sls;C1} := P_s$ $R_{A;sls;C1} = 57.92\ kN$

Line Load on Flank Walls.

Loading:	Dead Loads	Live Loads
Vaulted roof:	$G_{k2} := 1.07 \frac{kN}{m^2} \cdot \frac{12.0\ m}{2} = 6.42 \frac{kN}{m}$	$Q_{k2} := 0.50 \frac{kN}{m^2} \cdot \frac{12.0\ m}{2} = 3.00 \frac{kN}{m}$
Wall:	$G_{k9} := 5.98 \frac{kN}{m^2} \cdot 2.50\ m = 14.95 \frac{kN}{m}$	

Total Dead Load: $G_k := \sum G_k$ $\quad G_k = 21.37 \frac{kN}{m}$

Total Live Load: $Q_k := \sum Q_k$ $\quad Q_k = 3.00 \frac{kN}{m}$

Line Load on Inner Flank Walls.

Loading:	Dead Loads	Live Loads
Vaulted roof:	$G_{k2} := 1.07 \frac{kN}{m^2} \cdot \frac{12.0\ m}{2} = 6.42 \frac{kN}{m}$	$Q_{k2} := 0.50 \frac{kN}{m^2} \cdot \frac{12.0\ m}{2} = 3.00 \frac{kN}{m}$
Wall:	$G_{k9} := 5.98 \frac{kN}{m^2} \cdot 5.00\ m = 29.90 \frac{kN}{m}$	

Total Dead Load: $G_k := \sum G_k$ $\quad G_k = 36.32 \frac{kN}{m}$

Total Live Load: $Q_k := \sum Q_k$ $\quad Q_k = 3.00 \frac{kN}{m}$

Line Load on Gable Walls.

Loading:	Dead Loads	Live Loads
Vaulted roof:	$G_{k2} := 1.07 \frac{kN}{m^2} \cdot 1.25\ m = 1.34 \frac{kN}{m}$	$Q_{k2} := 0.50 \frac{kN}{m^2} \cdot 1.25\ m = 0.63 \frac{kN}{m}$
Wall:	$G_{k9} := 5.98 \frac{kN}{m^2} \cdot 3.50\ m = 20.93 \frac{kN}{m}$	

Total Dead Load: $G_k := \sum G_k$ $\quad G_k = 22.27 \frac{kN}{m}$

Total Live Load: $Q_k := \sum Q_k$ $\quad Q_k = 0.63 \frac{kN}{m}$

Design of Concrete Slab Thickening.

Maximum span of ground beam required: $L := 2.0\ m$

Try section size: Width: $b := 450\ mm$

Depth: $t := 450\ mm$

Assume global cover: $c := 40\ mm$

Concrete strength: $f_{cu} := 35\ \frac{N}{mm^2}$

Main reinforcement yield strength: $f_y := 500\ \frac{N}{mm^2}$

Shear reinforcement yield strength: $f_{yv} := 500\ \frac{N}{mm^2}$

Max Sagging Moment: $M_s := 23.97\ kN \cdot m$ $M_s = 23.97\ kN \cdot m$

Max Hogging Moment: $M_h := 23.97\ kN \cdot m$ $M_h = 23.97\ kN \cdot m$

Max Shear Force: $V := 71.6\ kN$ $V = 71.60\ kN$

Bottom Reinforcement:

Moment: $M_s = 23.97\ kN \cdot m$

Assumed reinforcement sizes for effective depth:

Main bar diameter: $\phi_b = 12.00\ mm$

Link diameter: $\phi_L = 10.00\ mm$

Effective depth: $d_b := t - c - \phi_L - \frac{\phi_b}{2} = 394.00\ mm$ $d_b = 394.00\ mm$

$k := \frac{M_s}{f_{cu} \cdot b \cdot {d_b}^2}$ $k = 0.01$

$k_{check1} = \text{“k < 0.156”}$ Therefore, $k_{check2} = \text{“no compression steel required.”}$

$Z_{check1} = \text{“Z > 0.95 x d, therefore make Z = 0.95 x d”}$ $Z = 374.30\ mm$

Area of steel required: $As_{req;b} := \frac{M_s}{0.87 \cdot f_y \cdot Z}$ $As_{req;b} = 147.22\ mm^2$

Try: Main bar diameter: $\phi_b \equiv 12\ mm$

Number of main bars: $N_b := 3$

Area of steel provided: $As_{prov;b} := N_b \cdot \pi \cdot \left(\frac{\phi_b}{2}\right)^2$ $As_{prov;b} = 339.29\ mm^2$

Utilisation ratios:

Bottom bars: $As_{prov;b} = 339.29\ mm^2$ > $As_{req;b} = 147.22\ mm^2$ $U_b := \frac{As_{prov;b}}{As_{req;b}} = 2.30$ $Steel\_Area = \text{“OK”}$

Min. area: $As_{prov;b} = 339.29\ mm^2$ > $A_{smin;req} = 263.25\ mm^2$ $U_m := \frac{As_{prov;b}}{A_{smin;req}} = 1.29$ $Min\_Area = \text{“OK”}$

Therefore provide 3H12 bars in the bottom, 40mm cover.

Top Reinforcement:

Moment: $M_h = 23.97\ kN \cdot m$

Assumed reinforcement sizes for effective depth:

Main bar diameter: $\phi_t = 12.00\ mm$

Link diameter: $\phi_L = 10.00\ mm$

Effective depth: $d_t := t - c - \phi_L - \dfrac{\phi_t}{2} = 394.00\ mm$ $d_t = 0.39\ m$

$k := \dfrac{M_h}{f_{cu} \cdot b \cdot d_t^{\ 2}}$ $k = 0.01$

$k_{check1} = \text{“k < 0.156”}$ Therefore, $k_{check2} = \text{“no compression steel required.”}$

$Z_{check1} = \text{“Z > 0.95 x d, therefore make Z = 0.95 x d”}$ $Z = 374.30\ mm$

Area of steel required: $As_{req;t} := \dfrac{M_h}{0.87 \cdot f_y \cdot Z}$ $As_{req;t} = 147.22\ mm^2$

Try: Main bar diameter: $\phi_t \equiv 12\ mm$

Number of main bars: $N_b := 3$

Area of steel provided: $As_{prov;t} := N_b \cdot \pi \cdot \left(\dfrac{\phi_t}{2}\right)^2$ $As_{prov;t} = 339.29\ mm^2$

Utilisation ratios:

Top bars: $As_{prov;t} = 339.29\ mm^2$ > $As_{req;t} = 147.22\ mm^2$ $U_b := \dfrac{As_{prov;t}}{As_{req;t}} = 2.30$ $Area = \text{“OK”}$

Min. area: $As_{prov;t} = 339.29\ mm^2$ > $A_{smin;req} = 263.25\ mm^2$ $U_m := \dfrac{As_{prov;t}}{A_{smin;req}} = 1.29$ $Min\_Area = \text{“OK”}$

Therefore provide 3H12 bars in the top, 40mm cover.

Check Shear:

Applied Shear: $V := V \cdot \frac{(L - 0\ mm)}{L}$ $V = 71.60\ kN$

Applied Shear Stress: $v := \frac{V}{b \cdot t}$ $v = 0.35\ \frac{N}{mm^2}$

Effective Depth : $d_b = 394.00\ mm$

Ratio: $R := \frac{100 \cdot As_{prov;b}}{b \cdot t}$ $R = 0.17$

Maximum permissible design shear stress: $v_b = 4.73\ \frac{N}{mm^2}$

$Links$ = "Minimum links for whole length of beam to provide a shear resistance of 0.4N/mm^2. Va = 0.4N/mm^2"

Therefore design shear stress from BS 8110-1, Table 3.8: $v_c = 0.34\ \frac{N}{mm^2}$

Shear stress resistance: $v_a = 0.40\ \frac{N}{mm^2}$

Max Shear Spacing: $S_v := 0.75 \cdot t = 337.50\ mm$ $S_v := 300\ mm$

Area of shear steel required: $Asv_{req} := \frac{b \cdot S_v \cdot v_a}{0.87 \cdot f_{yv}}$ $Asv_{req} = 124.14\ mm^2$

Try: Link bar diameter: $\phi_L \equiv 10\ mm$

Number of link bars: $N_L := 2$

Area of steel provided: $Asv_{prov} := N_L \cdot \pi \cdot \left(\frac{\phi_L}{2}\right)^2$ $Asv_{prov} = 157.08\ mm^2$

Utilisation ratios:

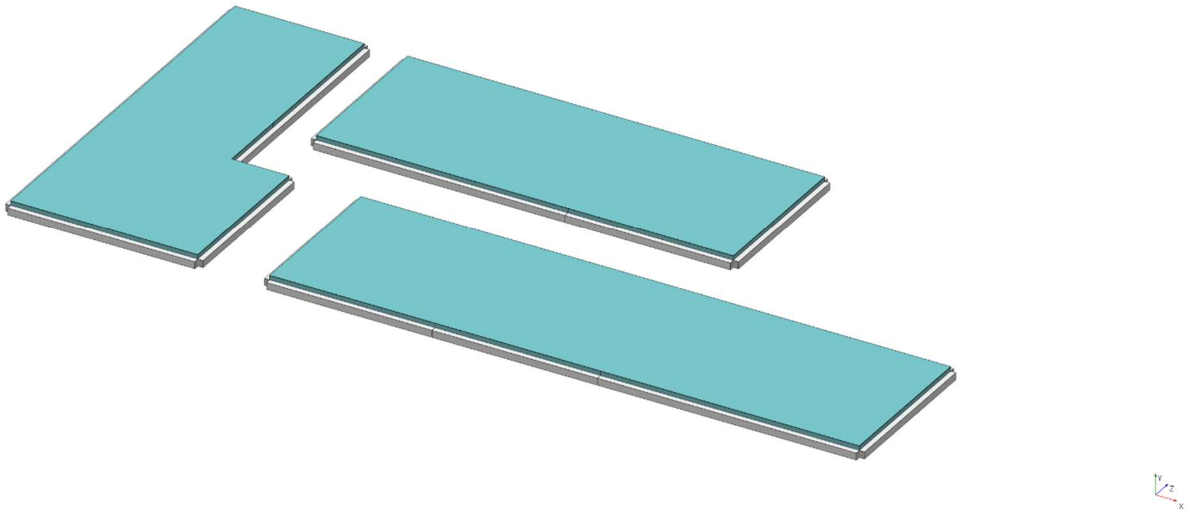
Shear: $Asv_{prov} = 157.08\ mm^2$ > $Asv_{req} = 124.14\ mm^2$ $U_b := \frac{Asv_{prov}}{Asv_{req}} = 1.27$ $Area$ = "OK"

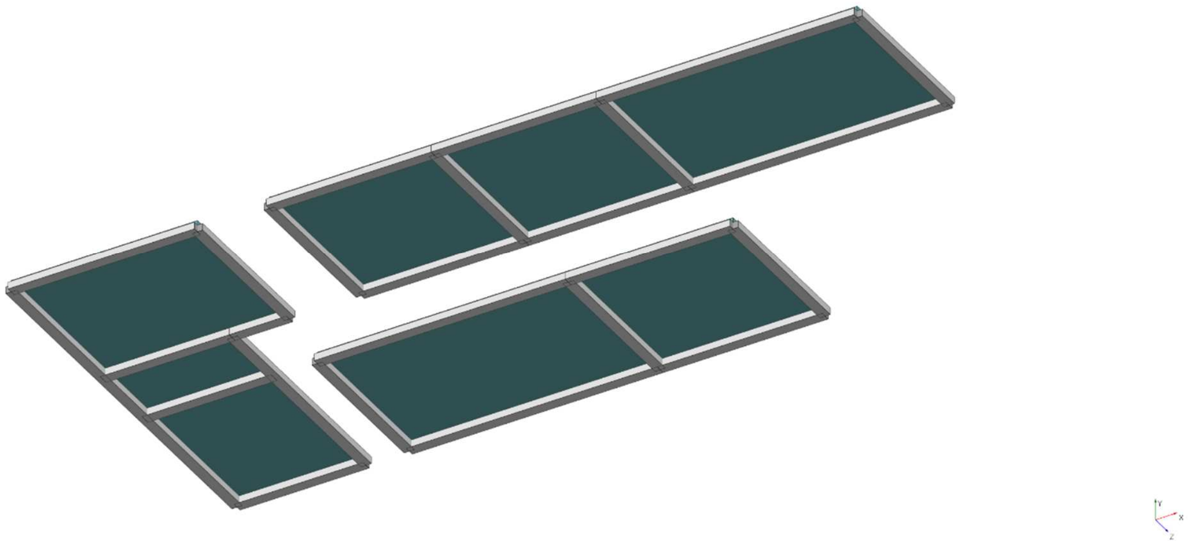
Therefore provide 2H10 Legs at 300mm c/c.

Summary:

450mm wide x 450mm C28/35 Concrete Thickening with 3H12 bars Top and 3H12 bars Bottom with 2H10 Legs at 300mm c/c. 40mm cover to all sides.

MasterFrame Data File





Loading Cases and Load Combination

Load Group Labels

Load Group UT	Unity Load Factor (All Cases)
Load Group D1	Dead Load
Load Group L1	Live Load

Load Case 001 : Dead plus Live (Ultimate)

Load Combination + 1.00 UT + 1.40 D1 + 1.60 L1

Load Case 002 : Dead Plus Live (Serviceability)

Load Combination + 1.00 UT + 1.00 D1 + 1.00 L1

Load Case 003 : Dead (Serviceability)

Load Combination + 1.00 UT + 1.00 D1

Load Case 004 : Live (Serviceability)

Load Combination + 1.00 UT + 1.00 L1

The Nodal Co-ordinates

Node	X (m)	Y (m)	Z (m)	Node	X (m)	Y (m)	Z (m)
1	0.000	0.000	0.000	2	8.127	0.000	0.000
3	11.254	0.000	0.000	4	18.441	0.000	0.000
5	25.627	0.000	0.000	6	36.877	0.000	0.000
7	0.000	0.000	6.873	8	11.254	0.000	6.873
9	18.441	0.000	6.873	10	25.627	0.000	6.873
11	36.877	0.000	6.873	12	5.627	0.000	6.873
13	8.127	0.000	6.873	14	0.000	0.000	10.000
15	5.627	0.000	10.000	16	7.504	0.000	10.000
17	18.441	0.000	10.000	18	25.627	0.000	10.000
19	0.000	0.000	16.873	20	5.627	0.000	16.873
21	7.504	0.000	16.873	22	18.441	0.000	16.873
23	25.627	0.000	16.873				

Member Properties

Members 1-4268

```
M   ...   ...          450x300 E 26.00E6 G 10.0E6
 A 1350E-4              Ix 101250E-8          Iy 227813E-8          J 405000E-8
```

Member Loading

Member Self Weight Density Load Included in Load Group D1, defined by Modulus of Elasticity

E kN/mm²	Density kN/m³
>= 200.00	77.01
>= 20.00	24.00
>= 2.00	10.00

Members 1-4268 - MasterFrame Pro Loads

```
D1 D      024.000             ( kN/m³)
```

Nodal Loading and Support Conditions

Finite Element Surfaces

FE Surface 001- Horizontal

Material Properties

Material - Isotropic - C35 Concrete - 200 mm thick

```
E 26.57E6     G 11.07E6     Pois. 0.2     Density 2350kg/ m³
```

Loads

Full Area	L1 Y -5 kN/m²
Line Load	D1 Y -21.37 kN/m Location - Load Points 14,13
Line Load	L1 Y -3 kN/m Location - Load Points 14,13
Line Load	D1 Y -36.32 kN/m Location - Load Points 15,21
Line Load	L1 Y -3 kN/m Location - Load Points 15,21
Line Load	D1 Y -22.27 kN/m Location - Load Points 1,14
Line Load	L1 Y -0.63 kN/m Location - Load Points 1,14
Line Load	D1 Y -22.27 kN/m Location - Load Points 13,19
Line Load	L1 Y -0.63 kN/m Location - Load Points 13,19
Line Load	D1 Y -22.27 kN/m Location - Load Points 20,2
Line Load	L1 Y -0.63 kN/m Location - Load Points 20,2
Line Load	D1 Y -36.32 kN/m Location - Load Points 16,20
Line Load	L1 Y -3 kN/m

	Location - Load Points 16,20
Line Load	L1 Y -3 kN/m
	Location - Load Points 17,7
Line Load	D1 Y -21.37 kN/m
	Location - Load Points 1,2
Line Load	L1 Y -3 kN/m
	Location - Load Points 1,2

Load Point Global Co-Ordinates

Load Point No.	X(m)	Y(m)	Z(m)
001	0000.000	0000.000	0000.000
002	0008.127	0000.000	0000.000
007	0036.877	0000.000	0006.873
013	0005.627	0000.000	0016.873
014	0000.000	0000.000	0016.873
015	0000.000	0000.000	0010.000
016	0000.000	0000.000	0006.873
017	0011.254	0000.000	0006.873
019	0005.627	0000.000	0006.873
020	0008.127	0000.000	0006.873
021	0005.627	0000.000	0010.000

Edge Supports

Edge support on member - Fixed 101000
Members 1, 9, 13, 16-17 & 20-23

FE Surface 002- Horizontal

Material Properties

Material - Isotropic - C35 Concrete - 200 mm thick

E 26.57E6 G 11.07E6 Pois. 0.2 Density 2350kg/ m³

Loads

Full Area	L1 Y -5 kN/m²
Line Load	D1 Y -21.37 kN/m
	Location - Load Points 1,2
Line Load	L1 Y -3 kN/m
	Location - Load Points 1,2
Line Load	D1 Y -36.32 kN/m
	Location - Load Points 4,5
Line Load	L1 Y -3 kN/m
	Location - Load Points 4,5
Line Load	D1 Y -22.27 kN/m
	Location - Load Points 4,1
Line Load	L1 Y -0.63 kN/m
	Location - Load Points 4,1
Line Load	D1 Y -22.27 kN/m
	Location - Load Points 2,5
Line Load	L1 Y -0.63 kN/m
	Location - Load Points 2,5

Load Point Global Co-Ordinates

Load Point No.	X(m)	Y(m)	Z(m)
001	0007.504	0000.000	0016.873
002	0025.627	0000.000	0016.873
004	0007.504	0000.000	0010.000
005	0025.627	0000.000	0010.000

Edge Supports

Edge support on member - Fixed 101000
Members 11-12, 14-15 & 24-25

FE Surface 003- Horizontal

Material Properties

Material - Isotropic - C35 Concrete - 200 mm thick

E 26.57E6 G 11.07E6 Pois. 0.2 Density 2350kg/ m³

Loads

Full Area	L1 Y -5 kN/m²
Line Load	D1 Y -22.27 kN/m
	Location - Load Points 1,2
Line Load	L1 Y -0.63 kN/m
	Location - Load Points 1,2
Line Load	D1 Y -36.32 kN/m
	Location - Load Points 2,3
Line Load	L1 Y -3 kN/m
	Location - Load Points 2,3

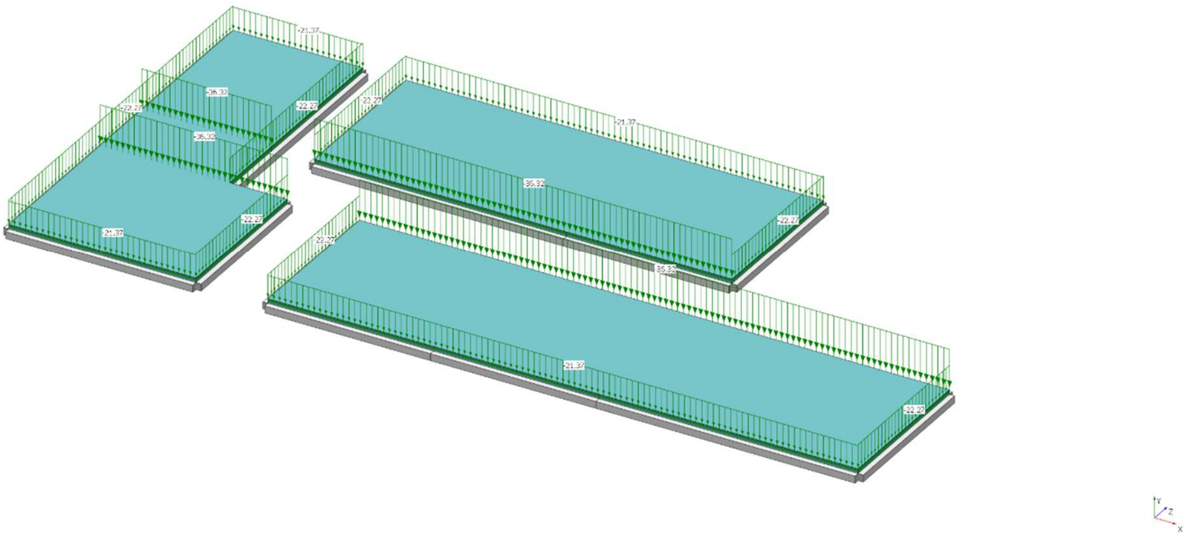
Line Load	D1 Y -22.27 kN/m Location - Load Points 3,4
Line Load	L1 Y -0.63 kN/m Location - Load Points 3,4
Line Load	D1 Y -21.37 kN/m Location - Load Points 1,4
Line Load	L1 Y -3 kN/m Location - Load Points 1,4

Load Point Global Co-Ordinates

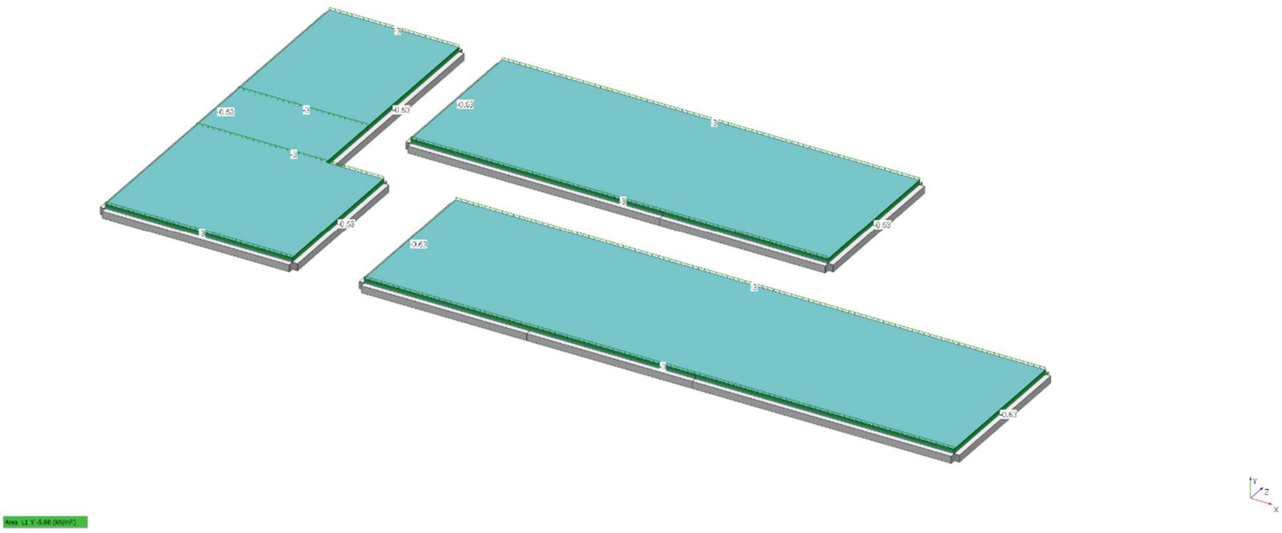
Load Point No.	X(m)	Y(m)	Z(m)
001	0011.254	0000.000	0000.000
002	0011.254	0000.000	0006.873
003	0036.877	0000.000	0006.873
004	0036.877	0000.000	0000.000

Edge Supports

Edge support on member - Fixed 101000
Members 2-4, 6-8 & 18-19

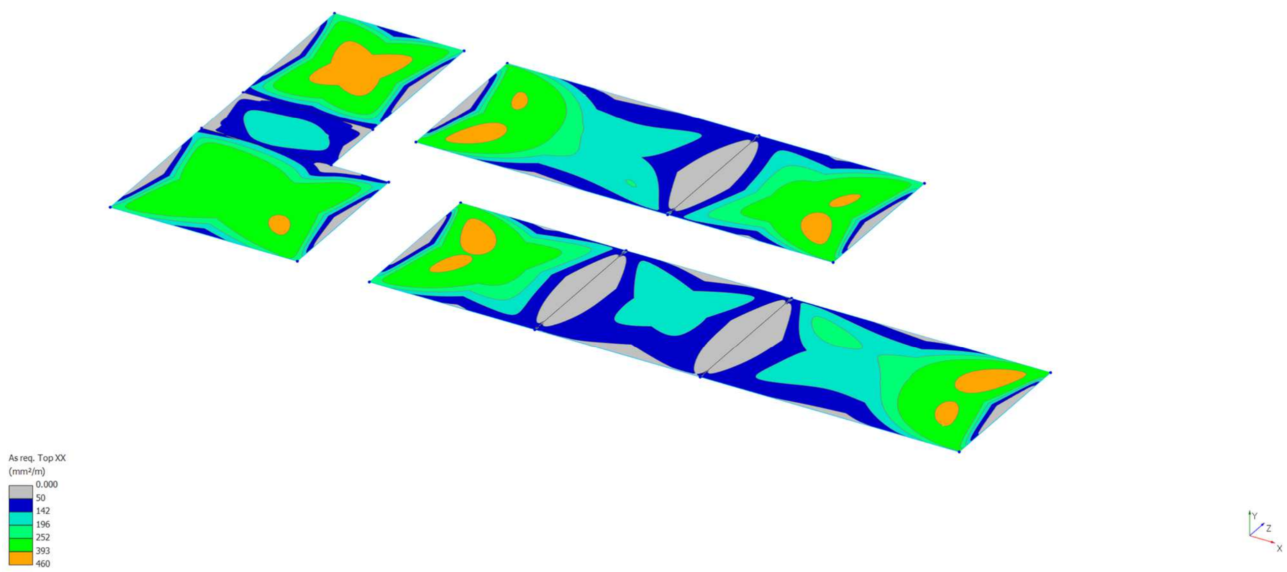


Load Diagram - 003 : Dead (Serviceability) - All Groups - Y Loads
Frame Geometry - Full Frame - 3D Front View

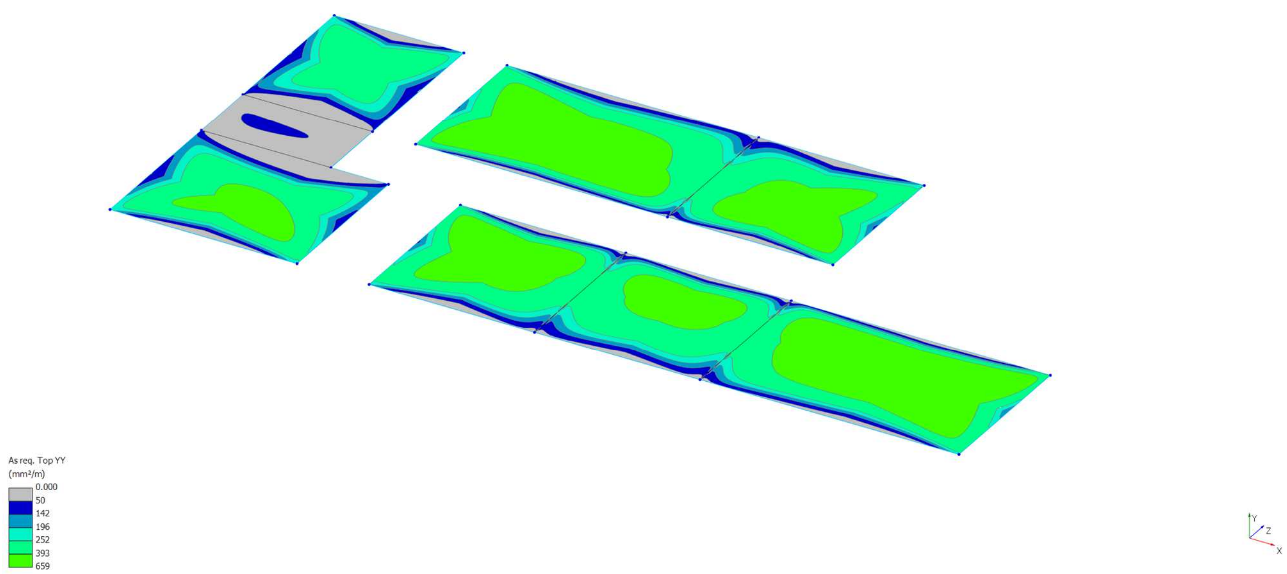


Load Diagram - 004 : Live (Serviceability) - All Groups - Y Loads
Frame Geometry - Full Frame - 3D Front View

MasterFrame : Graphics

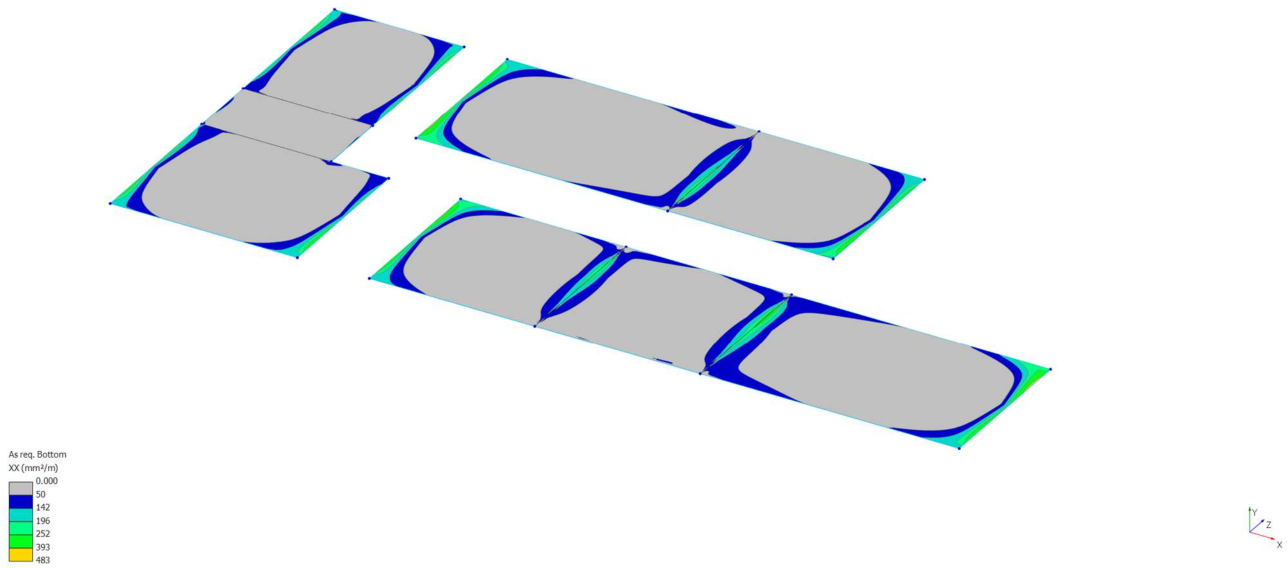


Load Case 001 : Dead plus Live (Ultimate)
Finite Element Colour Contour Diagram - Full Frame - 3D Front View
As req. Top XX (mm²/m)

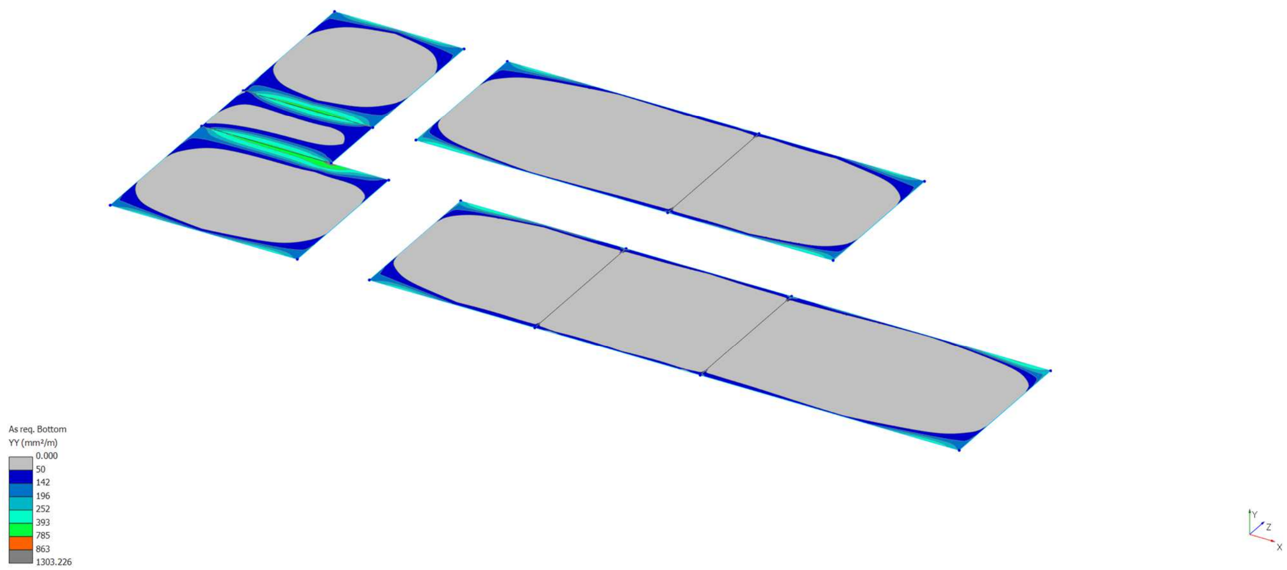


Load Case 001 : Dead plus Live (Ultimate)
Finite Element Colour Contour Diagram - Full Frame - 3D Front View
As req. Top YY (mm²/m)

MasterFrame : Graphics

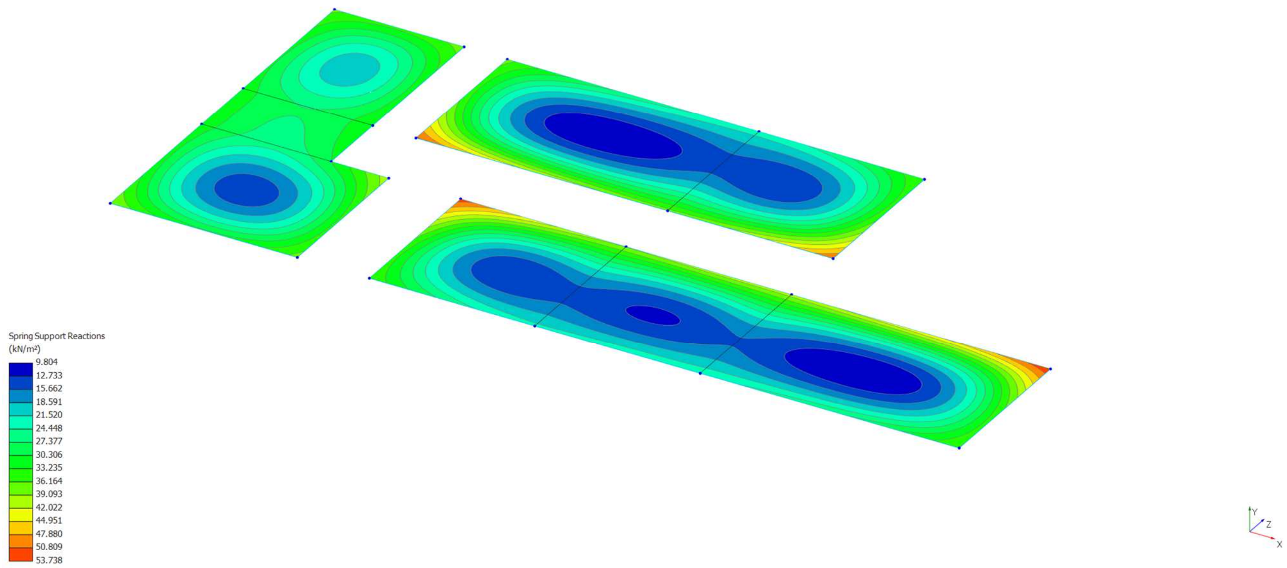


Load Case 001 : Dead plus Live (Ultimate)
Finite Element Colour Contour Diagram - Full Frame - 3D Front View
As req. Bottom XX (mm²/m)



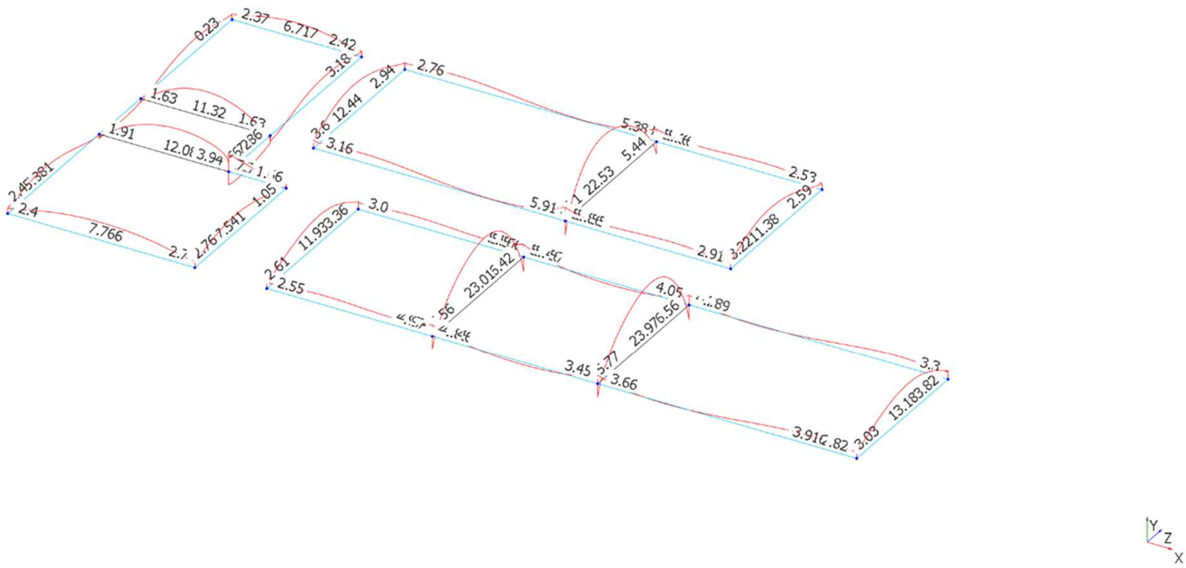
Load Case 001 : Dead plus Live (Ultimate)
Finite Element Colour Contour Diagram - Full Frame - 3D Front View
As req. Bottom YY (mm²/m)

MasterFrame : Graphics

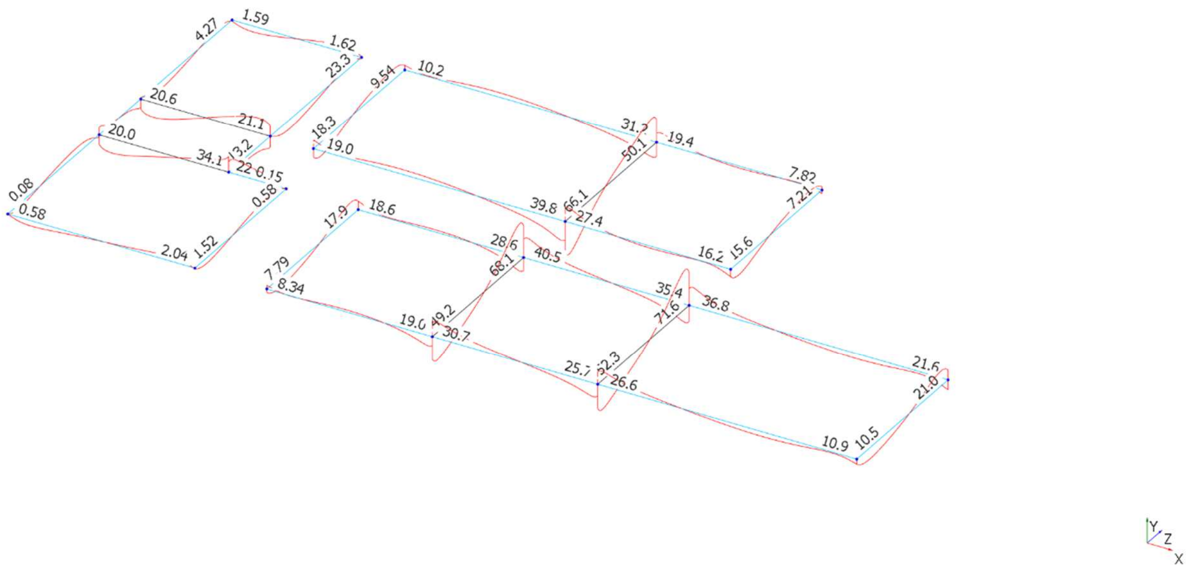


Load Case 002 : Dead Plus Live (Serviceability)
Finite Element Colour Contour Diagram - Full Frame - 3D Front View
Spring Support Reactions (kN/m²)

MasterFrame : Graphics



Load Case 002 : Dead Plus Live (Serviceability)
Bending Moment Diagram (Major Axis) - Full Frame - 3D Front View
Bending Moment Values (kN.m)



Load Case 002 : Dead Plus Live (Serviceability)
Shear Force Diagram (Major Axis) - Full Frame - 3D Front View
Shear Force Values (kN)

The Building Regulations

Building Regulations apply to most new buildings and many alterations of existing buildings in England and Wales, whether for domestic, commercial, and industrial use. Compliance is a legal requirement.

Unless specifically requested, we assume that the Lead Consultant, Contractor or Client will communicate and coordinate with Building Control or an Approved Inspector throughout the project. This will include the timely issue of information.

Planning Permission

Most projects will require Planning Permission.

Unless specifically requested, we assume that the Lead Consultant, Contractor or Client will communicate and coordinate with the Planning Authority.

There will be instances when we communicate with the Planning Officers, for example on projects which may be sensitive in terms of conservation. However, unless specifically stated the lead will be taken by others.

The Party Wall etc Act 1996

The Party Wall etc Act 1996 provides a framework for preventing and resolving disputes in relation to party walls, boundary walls and excavations near neighbouring buildings.

A building owner proposing to start work covered by the Act must give adjoining owners notice of their intentions in the way set down in the Act. Adjoining owners can agree or disagree with what is proposed. Where they disagree, the Act provides a mechanism for resolving disputes. The Act is separate from obtaining planning permission or building regulations approval.

Canham Consulting

The CDM Regulations 2015

The Construction (Design and Management) 2015 regulations set out what people involved in construction work need to do to protect themselves from harm and anyone the work affects, improving health and safety in the construction industry.

The role of CDM Co-ordinator from CDM 2007 has been removed within CDM 2015 with those duties being placed on other members of the project team, namely the Client, Principal Contractor and the new role of the Principal Designer. In effect the structure of the new regulations has been simplified and introduces early participation and additional duties from the appointed professionals and client in respect to Health and Safety matters on construction projects.

Domestic clients not previously encompassed by CDM must also take on duties, although domestic clients may discharge their duties onto the contractor or agreeing with the designer that they coordinate and manage the project through the construction phase rather than the role automatically passing to the contractor.

When working with domestic clients and being the only designer involved in the project, Canham Consulting will have fulfilled their duties as a Principal Designer under the CDM 2015 Regulations on completion of our appointment. On appointment of either a Contractor or Principal Contractor under The CDM Regulations 2015, they will become responsible for carrying out the Client's duties.

Extract from CDM 2015 - Table 1: A summary of roles and duties under CDM Dutyholders*

Clients - are organisations or individuals for whom a construction project is carried out.
Roles/Duties - Make suitable arrangements for managing a project. This includes making sure:

- other dutyholders are appointed;
- sufficient time and resources are allocated;

Making sure:

- relevant information is prepared and provided to other dutyholders;
- the principal designer and principal contractor carry out their duties;
- welfare facilities are provided.

Domestic clients - are people who have construction work carried out on their own home, or the home of a family member that is not done as part of a business, whether for profit or not.

Domestic clients are in scope of CDM 2015, but their duties as a client are normally transferred to:

- the contractor, on a single contractor project; or;
- the principal contractor, on a project involving more than one contractor.

However, the domestic client can choose to have a written agreement with the principal designer to carry out the client duties.

Designers – are those, who as part of a business, prepare or modify designs for a building, product or system relating to construction work.
Roles/Duties - When preparing or modifying designs, to eliminate, reduce or control foreseeable risks that may arise during construction; and the maintenance and use of a building once it is built.
Provide information to other members of the project team to help them fulfil their duties.

Principal designers – are designers appointed by the client in projects involving more than one contractor. They can be an organisation or an individual with sufficient knowledge, experience and ability to carry out the role.
Roles/Duties - Plan, manage, monitor and coordinate health and safety in the pre-construction phase of a project. This includes:

- identifying, eliminating or controlling foreseeable risks;
- ensuring designers carry out their duties;
- Prepare and provide relevant information to other dutyholders;

- Liaise with the principal contractor to help in the planning, management, monitoring and coordination of the construction phase.

Principal contractors – are contractors appointed by the client to coordinate the construction phase of a project where it involves more than one contractor.
Roles/Duties - Plan, manage, monitor and coordinate the construction phase of a project. This includes:

- liaising with the client and principal designer;
- preparing the construction phase plan;
- organising cooperation between contractors and coordinating their work.

Ensure:

- suitable site inductions are provided;
- reasonable steps are taken to prevent unauthorised access;
- workers are consulted and engaged in securing their health and safety; and
- welfare facilities are provided

Contractors - Are those who do the actual construction work and can be either an individual or a company
Roles/Duties - Plan, manage and monitor construction work under their control so that it is carried out without risks to health and safety;
For projects involving more than one contractor, coordinate their activities with others in the project team – in particular, comply with directions given to them by the principal designer or principal contractor;
For single-contractor projects, prepare a construction phase plan.

Workers - Are the people who work for or under the control of contractors on a construction site
Roles/Duties - They must:

- be consulted about matters which affect their health, safety and welfare;
- take care of their own health and safety and others who may be affected by their actions;
- report anything they see which is likely to endanger either their own or others' health and safety;
- cooperate with their employer, fellow workers, contractors and other dutyholders.

For more information on CDM 2015 please follow the link:
http://www.hse.gov.uk/construction/cdm/2015/index.htm
* Organisations or individuals can carry out the role of more than one dutyholder, provided they have the skills, knowledge, experience and (if an organisation) the organisational capability necessary to carry out those roles in a way that secures health and safety.

Temporary Works

We frequently design structures which are classified as 'Temporary Works'.

Temporary works (TW) are the parts of a construction project that are needed to enable the permanent works to be built. Usually the TW are removed after use - e.g. access scaffolds, props, shoring, excavation support, falsework and formwork, etc. Sometimes the TW is incorporated into the permanent works - e.g. haul road foundations and crane or piling platforms may be used for hardstanding or road foundations.

It is very important that the same degree of care and attention is given to the design and construction of temporary works (TW) as to the design and construction of the permanent works. As TW may be in place for only a short while there is a tendency to assume they are less important. This is incorrect. Lack of care with design, selection, assembly, etc leaves TW liable to fail or collapse. This places people at risk of injury and can cause the project to be delayed.

British Standard 5975 sets out one way of managing temporary works (TW) that has been found to work well on medium and large projects and uses the job title Temporary Works Coordinator (TWC). There is no legal requirement to use this job title or the BS recommended process, but you should remember that BS5975 provides an industry consensus view on what is considered to be good practice. The legal requirement is that the party in control must ensure that work is allocated and carried out in a manner that does not create unacceptable risk of harm to workers or members of the public. On projects with relatively simple TW needs, you may choose not to appoint a TWC. However, you must still make sure that TW are properly managed to ensure safety.

Unless specifically instructed to do so, Canham Consulting Ltd will not undertake the role of Temporary Works Coordinator.

Design Assumptions

Most designs will carry a certain degree of assumptions. As designers we will ensure that these are reasonable assumptions and frequently ask that these assumptions are verified by the Contractor on site.

In the absence of information regarding ground conditions we will often make educated assumptions which must be verified on site. It is the Clients and Contractors responsibility to ensure that our recommendations concerning investigation and verification of our assumptions are followed through. A typical example of this is as follows:

A modest residential project, incorporating an extension to an existing property. Our brief is to design a series of steel beams and supports. We will design the supports and foundation requirements, assuming a certain stratum. This is often stated as "assume medium dense / dense granular material with a net allowable bearing pressure of $100kN/m^2$ or greater, all to be confirmed by Contractor and Building Control on site, refer to Engineer for further information".

Where ground investigations have been undertaken, it is imperative to note that any investigation will only determine the ground conditions at the very location investigated. Differing ground conditions can and do exist elsewhere on the site and no assurance can be given this that is not the case. We will, however, make reasonable design assumptions and design with a degree of robustness and redundancy to accommodate slight variations in ground conditions.

We will typically make certain assumptions regarding forms of construction and existing materials, which must be confirmed and verified on site. This is particularly relevant with existing buildings; assumptions can be made with regards to wall construction, floor span and depth and roof construction. It is not always feasible for us to visit the site prior to undertaking our design and as such we will state any assumption which must be verified on site by the Client and Contractor.

Canham Consulting

Structural Civil Building

Engineers

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