JCP STRUCTURAL AND CIVIL ENGINEERS

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Structural Design Calculations

for

Proposed Workshop and Store

Site address
Amesbury Cricket Club,
Archers Way,
Amesbury,
SP4 7WQ

Oct 2023
Ref: 16469/DA/SC01

Brief:

JCP Engineers was appointed to carry out the structural design for the proposed workshop and store, a new building for the Amesbury Cricket Club on Archers Way, Amesbury.

Basis of the Design:

The scope of work covers the following structural design works:

- Specification of lintels over openings in all internal and external walls;
- Design of all associated masonry bearings;
- Design and detail reinforced concrete raft foundation

Robustness & Avoidance of Disproportionate Collapse:

The building is classed as a Consequence Class 2a building under the Building Regulations Part A with design guidance extract as follows:

In addition to the Consequence Class 1 measures, provide effective horizontal ties or effective anchorage of suspended floors to walls, as described in the Standards listed under paragraph 5.2 [not included here] for framed and load-bearing wall construction (the latter being defined in the paragraph 5.3 below [not included here]).

Design codes used:

BS 5268:	2002	"The Structural Use of Timber"
BS 5628:	1992	"Code of Practice for use of Masonry"
BS 5950:	2000	"The Structural Use of Steel in Building"
BS 5977:	1983	"Lintels"
BS 6399:	1997	"Loadings for Buildings"
BS 8004:	1986	"Code of Practice for Foundations"

Design software used:

Tekla Tedds 2023 — Structural Analysis & Design

Notes:

These calculations only apply to the structural elements included in these documents; if any discrepancies are found on site, the Engineer is to be informed. All architectural and building design requirements are to be provided by others.

Dimensions in these calculations are for design purposes only, having been scaled from copies of drawings. The building contractor is to obtain detailed dimensions from site measurements and is not to rely on those provided herein for fabrication purposes or procurement of materials.

The contractor is responsible for ensuring the stability of the structure at all times and that the works are carried out in strict compliance with all relevant Codes of Practice, Building Regulations and good building practice. All temporary support works required during the course of construction are the responsibility of the contractor.

Throughout the works the contractor / client is to be responsible for ensuring that the requirements of the Construction, Design and Management Regulations are complied with. Before any demolition works are undertaken, an R&D asbestos survey is recommended.

The Engineer believes that there are no foreseeable unusual risks that may arise during construction, maintenance, or use that a competent contractor could not foresee; however, due attention is to be given to ensuring a safe method of working and risk assessment is planned.

All structural steelwork and components are to be manufactured in accordance with the requirements of execution class EXC2 to BS EN 1090-2.

Architectural drawings:

992-05-C - Building Regs – GF & Roof plan
992-06-B - Building Regs – Section 01 & Details
992-07-A - Building Regs – Section 02 & Elevations

Structural drawings:

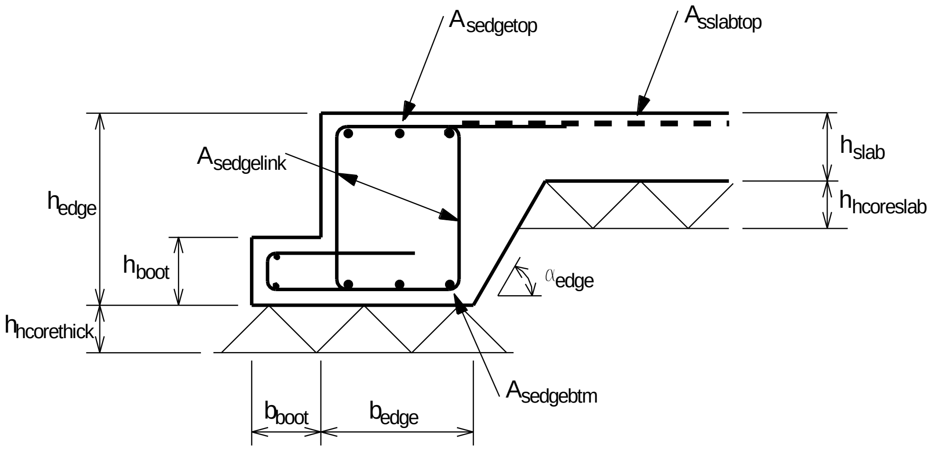
16469-001 - Substructure structural arrangement
16469-101 - Ground floor structural arrangement

GENERAL LOADINGS DATA

Pitched roof load			
Rafters	=	0.20	kN/m²
Roof covering - artificial slate	=	0.15	kN/m²
Battens, felt, insulation, etc	=	0.18	kN/m²
Total pitched roof dead load	=	0.53	kN/m²
Roof pitch	=	20	°
Dead load on plan	=	0.56	kN/m²
Roof imposed load	=	0.60	kN/m²
Total roof load	=	1.16	kN/m²
Ceiling load			
Joists	=	0.15	kN/m²
Plasterboard ceiling & skim coat	=	0.20	kN/m²
Total ceiling dead load	=	0.35	kN/m²
Ceiling imposed load	=	0.25	kN/m²
Total ceiling load	=	0.60	kN/m²
Cavity wall load			
Brickwork outer leaf	=	2.20	kN/m²
Blockwork inner leaf	=	2.00	kN/m²
Plasterboard & skim coat	=	0.20	kN/m²
Total cavity wall load	=	4.40	kN/m²
Ground Floor Load			
Screed 75mm	=	1.80	kN/m²
Insulation, etc	=	0.20	kN/m²
Total floor dead load	=	2.00	kN/m²
Floor imposed load	=	4.00	kN/m²
Total floor load	=	6.00	kN/m²

RAFT FOUNDATION

External wall	**Inner leaf**									
Dead udl from	inner wall		=	3.00	x	2.20	=	6.60	kN/m	
Dead udl from	pitched roof		=	6.10	x	0.56	=	3.42	kN/m	
Dead udl from	ceiling		=	6.10	x	0.35	=	2.14	kN/m	
		Total					=	12.15	kN/m	350mm
Imposed udl from	pitched roof		=	6.10	x	0.60	=	3.66	kN/m	
Imposed udl from	ceiling		=	6.10	x	0.25	=	1.53	kN/m	
		Total					=	5.19	kN/m	350mm
External wall	**Outer leaf**									
Dead udl from	outer wall		=	3.44	x	2.20	=	7.57	kN/m	
		Total					=	7.57	kN/m	148mm
Internal wall										
Dead udl from	cavity wall		=	3.00	x	4.40	=	13.20	kN/m	
		Total					=	13.20	kN/m	



Soil and raft definition

Soil definition

Allowable bearing pressure	q_{allow} = **75.0** kN/m^2
Number of types of soil forming sub-soil	**Two or more types**
Soil density	**Firm to loose**
Depth of hardcore beneath slab	$h_{hcoreslab}$ = **150** mm (Dispersal allowed for bearing pressure check)
Depth of hardcore beneath thickenings	$h_{hcorethick}$ = **150** mm (Dispersal allowed for bearing pressure check)
Density of hardcore	γ_{hcore} = **20.0** kN/m^3
Basic assumed diameter of local depression	$\phi_{depbasic}$ = **3500**mm

Diameter under slab modified for hardcore $\phi_{depslab} = \phi_{depbasic} - h_{hcoreslab} =$ **3350** mm
Diameter under thickenings modified for hardcore $\phi_{depthick} = \phi_{depbasic} - h_{hcorethick} =$ **3350** mm

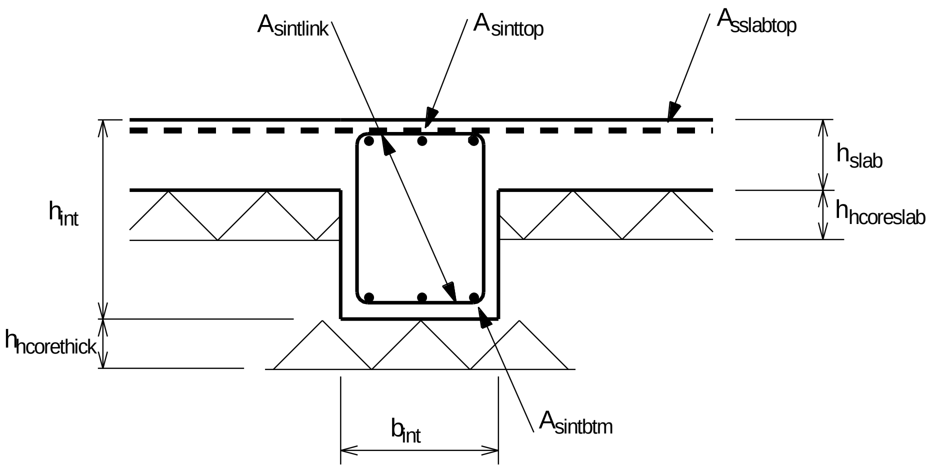
Raft slab definition

Max dimension/max dimension between joints $l_{max} =$ **16.500** m
Slab thickness $h_{slab} =$ **200** mm
Concrete strength $f_{cu} =$ **35** N/mm^2
Poissons ratio of concrete $\nu =$ **0.2**
Slab mesh reinforcement strength $f_{yslab} =$ **500** N/mm^2
Partial safety factor for steel reinforcement $\gamma_s =$ **1.15**
From C&CA document ‘Concrete ground floors’ Table 5
Minimum mesh required in top for shrinkage **A193**
Actual mesh provided in top **A393 ($A_{sslabtop}$ = 393 mm^2/m)**
Mesh bar diameter $\phi_{slabtop} =$ **10** mm
Cover to top reinforcement $c_{top} =$ **20** mm
Average effective depth of top reinforcement $d_{tslabav} = h_{slab} - c_{top} - \phi_{slabtop} =$ **170** mm
Minimum effective depth of top reinforcement $d_{tslabmin} = d_{tslabav} - \phi_{slabtop}/2 =$ **165** mm

Edge beam definition

Overall depth $h_{edge} =$ **600** mm
Width $b_{edge} =$ **450** mm
Depth of boot $h_{boot} =$ **375** mm
Width of boot $b_{boot} =$ **250** mm
Angle of chamfer to horizontal $\alpha_{edge} =$ **45** deg
Strength of main bar reinforcement $f_y =$ **500** N/mm^2
Strength of link reinforcement $f_{ys} =$ **500** N/mm^2
Reinforcement provided in top **4 H20 bars ($A_{sedgetop}$ = 1257 mm^2)**
Reinforcement provided in bottom **4 H20 bars ($A_{sedgebtm}$ = 1257 mm^2)**
Link reinforcement provided **2 H10 legs at 225 ctrs (A_{sv}/s_v = 0.698 mm)**
Bottom cover to links $c_{beam} =$ **35** mm
Effective depth of top reinforcement $d_{edgetop} = h_{edge} - c_{top} - \phi_{slabtop} - \phi_{edgelink} - \phi_{edgetop}/2 =$ **550** mm
Effective depth of bottom reinforcement $d_{edgebtm} = h_{edge} - c_{beam} - \phi_{edgelink} - \phi_{edgebtm}/2 =$ **545** mm
Boot main reinforcement **H10 bars at 225 ctrs (A_{sboot} = 349 mm^2/m)**
Effective depth of boot reinforcement $d_{boot} = h_{boot} - c_{beam} - \phi_{boot}/2 =$ **335** mm

Internal beam definition



Overall depth $h_{int} =$ **600** mm
Width $b_{int} =$ **450** mm

Strength of main bar reinforcement	f_y = **500** N/mm^2
Strength of link reinforcement	f_{ys} = **500** N/mm^2
Reinforcement provided in top	**3 H16 bars ($A_{sinttop}$ = 603 mm^2)**
Reinforcement provided in bottom	**3 H16 bars ($A_{sintbtm}$ = 603 mm^2)**
Link reinforcement provided	**2 H10 legs at 250 ctrs (A_{sv}/s_v = 0.628 mm)**
Effective depth of top reinforcement	$d_{inttop} = h_{int} - c_{top} - 2 \times \phi_{slabtop} - \phi_{inttop}/2$ = **552** mm
Effective depth of bottom reinforcement	$d_{intbtm} = h_{int} - c_{beam} - \phi_{intlink} - \phi_{intbtm}/2$ = **547** mm

<u>Internal slab design checks</u>

Basic loading

Slab self weight	w_{slab} = 24 kN/m^3 × h_{slab} = **4.8** kN/m^2
Hardcore	$w_{hcoreslab} = \gamma_{hcore} \times h_{hcoreslab}$ = **3.0** kN/m^2

Applied loading

Uniformly distributed dead load	w_{Dudl} = **2.0** kN/m^2
Uniformly distributed live load	w_{Ludl} = **4.0** kN/m^2

Slab load number 1

Load type	**Line load**
Dead load	w_{D1} = **6.6** kN/m
Live load	w_{L1} = **0.0** kN/m
Ultimate load	$w_{ult1} = 1.4 \times w_{D1} + 1.6 \times w_{L1}$ = **9.2** kN/m
Line load width	b_1 = **100** mm

Internal slab bearing pressure check

Total uniform load at formation level	$w_{udl} = w_{slab} + w_{hcoreslab} + w_{Dudl} + w_{Ludl}$ = **13.8** kN/m^2

Bearing pressure beneath load number 1

Effective loaded width	$x_1 = b_1 + 2 \times (h_{slab} + h_{hcoreslab} \times \tan(30))$ = **673** mm
Bearing pressure at formation level	$q_1 = (w_{D1} + w_{L1})/x_1 + w_{udl}$ = **23.6** kN/m^2

PASS - q <= q_{allow} - Applied bearing pressure is less than allowable

Internal slab bending and shear check

Applied bending moments

Effective span of slab	$l_{slab} = (\phi_{depslab} + d_{tslabav})/2$ = **1760** mm
Ultimate self weight udl	$w_{swult} = 1.4 \times w_{slab}$ = **6.7** kN/m^2
Approximate self weight cantilever moment at edge	$M_{esw} = (w_{swult} \times \pi \times l_{slab}^2) \times (l_{slab}/3) / (2 \times \pi \times l_{slab})$ = **3.5** kNm/m
Self weight shear force at edge	$V_{sw} = w_{swult} \times l_{slab} / 2$ = **5.9** kN/m

Moments due to applied uniformly distributed loads

Ultimate applied udl	$w_{udlult} = 1.4 \times w_{Dudl} + 1.6 \times w_{Ludl}$ = **9.2** kN/m^2
Approximate cantilever moment at edge	$M_{eudl} = (w_{udlult} \times \pi \times l_{slab}^2) \times (l_{slab}/3) / (2 \times \pi \times l_{slab})$ = **4.7** kNm/m
Shear force at edge	$V_{udl} = w_{udlult} \times l_{slab} / 2$ = **8.1** kN/m

Moment due to load number 1

Approximate equivalent udl	$w_{udl1} = 1.5 \times w_{ult1}/(2 \times 0.3 \times l_{slab})$ = **13.1** kN/m^2
Approximate cantilever moment at edge	$M_{e1} = (w_{udl1} \times \pi \times l_{slab}^2) \times (l_{slab}/3) / (2 \times \pi \times l_{slab})$ = **6.8** kNm/m
Shear force at edge	$V_1 = w_{udl1} \times l_{slab} / 2$ = **11.6** kN/m

Resultant moments and shears

Total moment at edge	$M_{\Sigma e}$ = **15.0** kNm/m
Total shear force	V_{Σ} = **25.6** kN/m

Reinforcement required in top

K factor	$K_{slabtop} = M_{\Sigma e}/(f_{cu} \times d_{tslabav}^2)$ = **0.015**
Lever arm	$z_{slabtop} = d_{tslabav} \times \min(0.95, 0.5 + \sqrt{(0.25 - K_{slabtop}/0.9)})$ = **161.5** mm
Area of steel required for bending	$A_{sslabtopbend} = M_{\Sigma e}/((1.0/\gamma_s) \times f_{yslab} \times z_{slabtop})$ = **214** mm^2/m
Minimum area of steel required	$A_{sslabmin} = 0.0013 \times h_{slab}$ = **260** mm^2/m
Area of steel required	$A_{sslabtopreq} = \max(A_{sslabtopbend}, A_{sslabmin})$ = **260** mm^2/m

PASS - $A_{sslabtopreq}$ <= $A_{sslabtop}$ - Area of reinforcement provided in top to span local depressions is adequate

Shear check

Applied shear stress	$v = V_{\Sigma}/d_{tslabmin}$ = **0.155** N/mm^2
Tension steel ratio	$\rho = 100 \times A_{sslabtop}/d_{tslabmin}$ = **0.238**
From BS8110-1:1997 - Table 3.8	

Design concrete shear strength v_c = **0.547** N/mm^2

PASS - $v \le v_c$ - Shear capacity of the slab is adequate

Internal slab deflection check

Basic allowable span to depth ratio $Ratio_{basic}$ = **7.0**

Moment factor $M_{factor} = M_{\Sigma e}/d_{tslabav}^2$ = **0.519** N/mm^2

Steel service stress $f_s = 2/3 \times f_{yslab} \times A_{sslabtopbend}/A_{sslabtop}$ = **181.129** N/mm^2

Modification factor $MF_{slab} = \min(2.0, 0.55 + [(477N/mm^2 - f_s)/(120 \times (0.9N/mm^2 + M_{factor}))])$

MF_{slab} = **2.000**

Modified allowable span to depth ratio $Ratio_{allow} = Ratio_{basic} \times MF_{slab}$ = **14.000**

Actual span to depth ratio $Ratio_{actual} = l_{slab}/d_{tslabav}$ = **10.353**

PASS - $Ratio_{actual} \le Ratio_{allow}$ - Slab span to depth ratio is adequate

<u>Edge beam design checks</u>

Basic loading

Hardcore $w_{hcorethick} = \gamma_{hcore} \times h_{hcorethick}$ = **3.0** kN/m^2

Edge beam

Rectangular beam element $w_{beam} = 24\ kN/m^3 \times h_{edge} \times b_{edge}$ = **6.5** kN/m

Boot element $w_{boot} = 24\ kN/m^3 \times h_{boot} \times b_{boot}$ = **2.3** kN/m

Chamfer element $w_{chamfer} = 24\ kN/m^3 \times (h_{edge} - h_{slab})^2/(2 \times \tan(\alpha_{edge}))$ = **1.9** kN/m

Slab element $w_{slabelmt} = 24\ kN/m^3 \times h_{slab} \times (h_{edge} - h_{slab})/\tan(\alpha_{edge})$ = **1.9** kN/m

Edge beam self weight $w_{edge} = w_{beam} + w_{boot} + w_{chamfer} + w_{slabelmt}$ = **12.6** kN/m

Edge load number 1

Load type **Longitudinal line load**

Dead load W_{Dedge1} = **12.2** kN/m

Live load W_{Ledge1} = **5.2** kN/m

Ultimate load $W_{ultedge1} = 1.4 \times W_{Dedge1} + 1.6 \times W_{Ledge1}$ = **25.4** kN/m

Longitudinal line load width b_{edge1} = **100** mm

Centroid of load from outside face of raft x_{edge1} = **350** mm

Edge load number 2

Load type **Longitudinal line load**

Dead load W_{Dedge2} = **7.6** kN/m

Live load W_{Ledge2} = **0.0** kN/m

Ultimate load $W_{ultedge2} = 1.4 \times W_{Dedge2} + 1.6 \times W_{Ledge2}$ = **10.6** kN/m

Longitudinal line load width b_{edge2} = **100** mm

Centroid of load from outside face of raft x_{edge2} = **148** mm

Edge load number 3

Load type **Transverse line load**

Dead load W_{Dedge3} = **13.2** kN/m

Live load W_{Ledge3} = **0.0** kN/m

Ultimate load $W_{ultedge3} = 1.4 \times W_{Dedge3} + 1.6 \times W_{Ledge3}$ = **18.5** kN/m

Transverse line load width b_{edge3} = **100** mm

Edge beam bearing pressure check

Effective bearing width of edge beam $b_{bearing} = b_{edge} + b_{boot} + (h_{edge} - h_{slab})/\tan(\alpha_{edge})$ = **1100** mm

Total uniform load at formation level $w_{udledge} = w_{Dudl}+w_{Ludl}+w_{edge}/b_{bearing}+w_{hcorethick}$ = **20.4** kN/m^2

Bearing pressure due to transverse line loads

Total dead transverse line load W_{Dtrans} = **13.2** kN/m

Total live transverse line load W_{Ltrans} = **0.0** kN/m

Total ultimate transverse line load $W_{ulttrans}$ = **18.5** kN/m

Minimum width of transverse line loads b_{trans} = **100** mm

Length of trans line load applied to edge beam $l_{transapp} = b_{edge} + (h_{edge} - h_{slab})/\tan(\alpha_{edge})$ = **850** mm

Total ult trans line load applied to edge beam $W_{ulttrans} = w_{ulttrans} \times l_{transapp}$ = **15.7** kN

Approx moment capacity of bottom steel $M_{edgebtm} = (1.0/\gamma_s) \times f_y \times 0.9 \times d_{edgebtm} \times A_{sedgebtm}$ = **268.0** kNm

Max allow dispersal based on moment capacity $p_{edgemom}=[2\times M_{edgebtm}+\sqrt{(4\times M_{edgebtm}^2+2\times W_{ulttrans}\times M_{edgebtm}\times b_{trans})}]/W_{ulttrans}$

$p_{edgemom}$ = **68293** mm

Limiting max dispersal to say 5 x beam depth $p_{edge} = \min(p_{edgemom}, 5 \times h_{edge})$ = 3000 mm

Total dispersal width of transverse line loads $l_{trans} = 2 \times p_{edge} + b_{trans}$ = **6100** mm

Bearing pressure due to trans line loads $q_{trans} = (w_{Dtrans} + w_{Ltrans}) \times l_{transapp}/(l_{trans} \times b_{bearing}) = \mathbf{1.7}$ kN/m^2

Centroid of longitudinal and equivalent line loads from outside face of raft

Load x distance for edge load 1 $Moment_1 = w_{ultedge1} \times x_{edge1} = \mathbf{8.9}$ kN

Load x distance for edge load 2 $Moment_2 = w_{ultedge2} \times x_{edge2} = \mathbf{1.6}$ kN

Sum of ultimate longitud'l and equivalent line loads $\Sigma UDL = \mathbf{36.0}$ kN/m

Sum of load x distances $\Sigma Moment = \mathbf{10.5}$ kN

Centroid of loads $x_{bar} = \Sigma Moment/\Sigma UDL = \mathbf{290}$ mm

Initially assume no moment transferred into slab due to load/reaction eccentricity

Sum of unfactored longitud'l and eff'tive line loads $\Sigma UDLsls = \mathbf{25.0}$ kN/m

Allowable bearing width $b_{allow} = 2 \times x_{bar} + 2 \times h_{hcoreslab} \times \tan(30) = \mathbf{754}$ mm

Bearing pressure due to line/point loads $q_{linepoint} = \Sigma UDLsls/ b_{allow} = \mathbf{33.2}$ kN/m^2

Total applied bearing pressure $q_{edge} = q_{linepoint} + q_{trans} + w_{udledge} = \mathbf{55.3}$ kN/m^2

PASS - q_{edge} <= q_{allow} - Allowable bearing pressure is not exceeded

Edge beam bending check

Divider for moments due to udl's $\beta_{udl} = \mathbf{10.0}$

Divider for moments due to point loads $\beta_{point} = \mathbf{6.0}$

Applied bending moments

Span of edge beam $l_{edge} = \phi_{depthick} + d_{edgetop} = \mathbf{3900}$ mm

Ultimate self weight udl $w_{edgeult} = 1.4 \times w_{edge} = \mathbf{17.6}$ kN/m

Ultimate slab udl (approx) $w_{edgeslab} = \max(0\ \text{kN/m}, 1.4 \times w_{slab} \times ((\phi_{depthick}/2 \times 3/4) - (b_{edge} + (h_{edge} - h_{slab})/\tan(\alpha_{edge}))))$

$w_{edgeslab} = \mathbf{2.7}$ kN/m

Self weight and slab bending moment $M_{edgesw} = (w_{edgeult} + w_{edgeslab}) \times l_{edge}^2/\beta_{udl} = \mathbf{30.9}$ kNm

Self weight shear force $V_{edgesw} = (w_{edgeult} + w_{edgeslab}) \times l_{edge}/2 = \mathbf{39.6}$ kN

Moments due to applied uniformly distributed loads

Ultimate udl (approx) $w_{edgeudl} = w_{udlult} \times \phi_{depthick}/2 \times 3/4 = \mathbf{11.6}$ kN/m

Bending moment $M_{edgeudl} = w_{edgeudl} \times l_{edge}^2/\beta_{udl} = \mathbf{17.6}$ kNm

Shear force $V_{edgeudl} = w_{edgeudl} \times l_{edge}/2 = \mathbf{22.5}$ kN

Moment and shear due to load number 1

Bending moment $M_{edge1} = w_{ultedge1} \times l_{edge}^2/\beta_{udl} = \mathbf{38.6}$ kNm

Shear force $V_{edge1} = w_{ultedge1} \times l_{edge}/2 = \mathbf{49.5}$ kN

Moment and shear due to load number 2

Bending moment $M_{edge2} = w_{ultedge2} \times l_{edge}^2/\beta_{udl} = \mathbf{16.2}$ kNm

Shear force $V_{edge2} = w_{ultedge2} \times l_{edge}/2 = \mathbf{20.7}$ kN

Moment and shear due to load number 3

Ultimate point load $W_{edge3} = w_{ultedge3} \times \phi_{depthick}/2 \times 3/4 = \mathbf{23.2}$ kN

Bending moment $M_{edge3} = W_{edge3} \times l_{edge}/\beta_{point} = \mathbf{15.1}$ kNm

Shear force $V_{edge3} = W_{edge3} = \mathbf{23.2}$ kN

Resultant moments and shears

Total moment (hogging and sagging) $M_{\Sigma edge} = \mathbf{118.4}$ kNm

Maximum shear force $V_{\Sigma edge} = \mathbf{155.7}$ kN

Reinforcement required in top

Width of section in compression zone $b_{edgetop} = b_{edge} + b_{boot} = \mathbf{700}$ mm

Average web width $b_w = b_{edge} + (h_{edge}/\tan(\alpha_{edge}))/2 = \mathbf{750}$ mm

K factor $K_{edgetop} = M_{\Sigma edge}/(f_{cu} \times b_{edgetop} \times d_{edgetop}^2) = \mathbf{0.016}$

Lever arm $z_{edgetop} = d_{edgetop} \times \min(0.95, 0.5 + \surd(0.25 - K_{edgetop}/0.9)) = \mathbf{523}$ mm

Area of steel required for bending $A_{sedgetopbend} = M_{\Sigma edge}/((1.0/\gamma_s) \times f_y \times z_{edgetop}) = \mathbf{521}$ mm^2

Minimum area of steel required $A_{sedgetopmin} = 0.0013 \times 1.0 \times b_w \times h_{edge} = \mathbf{585}$ mm^2

Area of steel required $A_{sedgetopreq} = \max(A_{sedgetopbend}, A_{sedgetopmin}) = \mathbf{585}$ mm^2

PASS - $A_{sedgetopreq}$ <= $A_{sedgetop}$ - Area of reinforcement provided in top of edge beams is adequate

Reinforcement required in bottom

Width of section in compression zone $b_{edgebtm} = b_{edge} + (h_{edge} - h_{slab})/\tan(\alpha_{edge}) + 0.1 \times l_{edge} = \mathbf{1240}$ mm

K factor $K_{edgebtm} = M_{\Sigma edge}/(f_{cu} \times b_{edgebtm} \times d_{edgebtm}^2) = \mathbf{0.009}$

Lever arm $z_{edgebtm} = d_{edgebtm} \times \min(0.95, 0.5 + \surd(0.25 - K_{edgebtm}/0.9)) = \mathbf{518}$ mm

Area of steel required for bending $A_{sedgebtmbend} = M_{\Sigma edge}/((1.0/\gamma_s) \times f_y \times z_{edgebtm}) = \mathbf{526}$ mm^2

Minimum area of steel required $A_{sedgebtmmin} = 0.0013 \times 1.0 \times b_w \times h_{edge} = \mathbf{585}$ mm^2
Area of steel required $A_{sedgebtmreq} = \max(A_{sedgebtmbend}, A_{sedgebtmmin}) = \mathbf{585}$ mm^2

PASS - $A_{sedgebtmreq} <= A_{sedgebtm}$ - Area of reinforcement provided in bottom of edge beams is adequate

Edge beam shear check

Applied shear stress $v_{edge} = V_{\Sigma edge}/(b_w \times d_{edgetop}) = \mathbf{0.377}$ N/mm^2
Tension steel ratio $\rho_{edge} = 100 \times A_{sedgetop}/(b_w \times d_{edgetop}) = \mathbf{0.305}$
From BS8110-1:1997 - Table 3.8
Design concrete shear strength $v_{cedge} = \mathbf{0.476}$ N/mm^2

$v_{edge} <= v_{cedge}$ + 0.4N/mm^2 - Therefore minimum links required

Link area to spacing ratio required $A_{sv\_upon\_s_{vreqedge}} = 0.4\text{N/mm}^2 \times b_w/((1.0/\gamma_s) \times f_{ys}) = \mathbf{0.690}$ mm
Link area to spacing ratio provided $A_{sv\_upon\_s_{vprovedge}} = N_{edgelink} \times \pi \times \phi_{edgelink}^2/(4 \times s_{vedge}) = \mathbf{0.698}$ mm

PASS - $A_{sv\_upon\_s_{vreqedge}} <= A_{sv\_upon\_s_{vprovedge}}$ - Shear reinforcement provided in edge beams is adequate

Boot design check

Effective cantilever span $l_{boot} = b_{boot} + d_{boot}/2 = \mathbf{418}$ mm
Approximate ultimate bearing pressure $q_{ult} = 1.55 \times q_{allow} = \mathbf{116.3}$ kN/m^2
Cantilever moment $M_{boot} = q_{ult} \times l_{boot}^2/2 = \mathbf{10.1}$ kNm/m
Shear force $V_{boot} = q_{ult} \times l_{boot} = \mathbf{48.5}$ kN/m
K factor $K_{boot} = M_{boot}/(f_{cu} \times d_{boot}^2) = \mathbf{0.003}$
Lever arm $z_{boot} = d_{boot} \times \min(0.95, 0.5 + \sqrt{(0.25 - K_{boot}/0.9)}) = \mathbf{318}$ mm
Area of reinforcement required $A_{sbootreq} = M_{boot}/((1.0/\gamma_s) \times f_{yboot} \times z_{boot}) = \mathbf{73}$ mm^2/m

PASS - $A_{sbootreq} <= A_{sboot}$ - Area of reinforcement provided in boot is adequate for bending

Applied shear stress $v_{boot} = V_{boot}/d_{boot} = \mathbf{0.145}$ N/mm^2
Tension steel ratio $\rho_{boot} = 100 \times A_{sboot}/d_{boot} = \mathbf{0.104}$
From BS8110-1:1997 - Table 3.8
Design concrete shear strength $v_{cboot} = \mathbf{0.348}$ N/mm^2

PASS - $v_{boot} <= v_{cboot}$ - Shear capacity of the boot is adequate

<u>Corner design checks</u>

Basic loading

Corner load number 1

Load type **Line load in x direction**
Dead load $w_{Dcorner1} = \mathbf{12.2}$ kN/m
Live load $w_{Lcorner1} = \mathbf{5.9}$ kN/m
Ultimate load $w_{ultcorner1} = 1.4 \times w_{Dcorner1} + 1.6 \times w_{Lcorner1} = \mathbf{26.5}$ kN/m
Centroid of load from outside face of raft $y_{corner1} = \mathbf{350}$ mm

Corner load number 2

Load type **Line load in y direction**
Dead load $w_{Dcorner2} = \mathbf{12.2}$ kN/m
Live load $w_{Lcorner2} = \mathbf{5.9}$ kN/m
Ultimate load $w_{ultcorner2} = 1.4 \times w_{Dcorner2} + 1.6 \times w_{Lcorner2} = \mathbf{26.5}$ kN/m
Centroid of load from outside face of raft $x_{corner2} = \mathbf{350}$ mm

Corner load number 3

Load type **Line load in x direction**
Dead load $w_{Dcorner3} = \mathbf{7.6}$ kN/m
Live load $w_{Lcorner3} = \mathbf{0.0}$ kN/m
Ultimate load $w_{ultcorner3} = 1.4 \times w_{Dcorner3} + 1.6 \times w_{Lcorner3} = \mathbf{10.6}$ kN/m
Centroid of load from outside face of raft $y_{corner3} = \mathbf{148}$ mm

Corner load number 4

Load type **Line load in y direction**
Dead load $w_{Dcorner4} = \mathbf{7.6}$ kN/m
Live load $w_{Lcorner4} = \mathbf{0.0}$ kN/m
Ultimate load $w_{ultcorner4} = 1.4 \times w_{Dcorner4} + 1.6 \times w_{Lcorner4} = \mathbf{10.6}$ kN/m
Centroid of load from outside face of raft $x_{corner4} = \mathbf{148}$ mm

Corner bearing pressure check

Total uniform load at formation level $w_{udlcorner} = w_{Dudl} + w_{Ludl} + w_{edge}/b_{bearing} + w_{hcorethick} = \mathbf{20.4}$ kN/m^2
Net bearing press avail to resist line/point loads $q_{netcorner} = q_{allow} - w_{udlcorner} = \mathbf{54.6}$ kN/m^2

Total line/point loads

Total unfactored line load in x direction — $w_{\Sigma linex}$ = **25.7** kN/m

Total ultimate line load in x direction — $w_{\Sigma ultlinex}$ =**37.2** kN/m

Total unfactored line load in y direction — $w_{\Sigma liney}$ = **25.7** kN/m

Total ultimate line load in y direction — $w_{\Sigma ultliney}$ = **37.2** kN/m

Total unfactored point load — $w_{\Sigma point}$ = **0.0** kN

Total ultimate point load — $w_{\Sigma ultpoint}$ = **0.0** kN

Length of side of sq reqd to resist line/point loads — $p_{corner} = [w_{\Sigma linex}+w_{\Sigma liney}+\sqrt{((w_{\Sigma linex}+w_{\Sigma liney})^2+4\times q_{netcorner}\times w_{\Sigma point})}]/(2\times q_{netcorner})$

p_{corner} = **942** mm

Bending moment about x-axis due to load/reaction eccentricity

Moment due to load 1 (x line) — $M_{x1} = \max(0\text{ kNm}, w_{ultcorner1} \times p_{corner} \times (p_{corner}/2 - y_{corner1}))$ = **3.0** kNm

Moment due to load 3 (x line) — $M_{x3} = \max(0\text{ kNm}, w_{ultcorner3} \times p_{corner} \times (p_{corner}/2 - y_{corner3}))$ = **3.2** kNm

Total moment about x axis — $M_{\Sigma x}$ = **6.3** kNm

Bending moment about y-axis due to load/reaction eccentricity

Moment due to load 2 (y line) — $M_{y2} = \max(0\text{ kNm}, w_{ultcorner2} \times p_{corner} \times (p_{corner}/2 - x_{corner2}))$ = **3.0** kNm

Moment due to load 4 (y line) — $M_{y4} = \max(0\text{ kNm}, w_{ultcorner4} \times p_{corner} \times (p_{corner}/2 - x_{corner4}))$ = **3.2** kNm

Total moment about y axis — $M_{\Sigma y}$ = **6.3** kNm

Check top reinforcement in edge beams for load/reaction eccentric moment

Max moment due to load/reaction eccentricity — $M_{\Sigma} = \max(M_{\Sigma x}, M_{\Sigma y})$ = **6.3** kNm

Assume all of this moment is resisted by edge beam

From edge beam design checks away from corners

Moment due to edge beam spanning depression — $M_{\Sigma edge}$ = **118.4** kNm

Total moment to be resisted — $M_{\Sigma cornerbp} = M_{\Sigma} + M_{\Sigma edge}$ = **124.7** kNm

Width of section in compression zone — $b_{edgetop} = b_{edge} + b_{boot}$ = **700** mm

K factor — $K_{cornerbp} = M_{\Sigma cornerbp}/(f_{cu} \times b_{edgetop} \times d_{edgetop}^2)$ = **0.017**

Lever arm — $z_{cornerbp} = d_{edgetop} \times \min(0.95, 0.5 + \sqrt{(0.25 - K_{cornerbp}/0.9)})$ = **523** mm

Total area of top steel required — $A_{scornerbp} = M_{\Sigma cornerbp} /((1.0/\gamma_s) \times f_y \times z_{cornerbp})$ = **549** mm^2

PASS - $A_{scornerbp}$ <= $A_{sedgetop}$ - Area of reinforcement provided to resist eccentric moment is adequate

The allowable bearing pressure at the corner will not be exceeded

Corner beam bending check

Cantilever span of edge beam — $l_{corner} = \phi_{depthick}/\sqrt{(2)} + d_{edgetop}/2$ = **2644** mm

Moment and shear due to self weight

Ultimate self weight udl — $w_{edgeult} = 1.4 \times w_{edge}$ = **17.6** kN/m

Average ultimate slab udl (approx) — $w_{cornerslab} = \max(0\text{ kN/m}, 1.4\times w_{slab}\times(\phi_{depthick}/(\sqrt{(2)}\times 2)-(b_{edge}+(h_{edge}-h_{slab})/\tan(\alpha_{edge}))))$

$w_{cornerslab}$ = **2.2** kN/m

Self weight and slab bending moment — $M_{cornersw} = (w_{edgeult} + w_{cornerslab}) \times l_{corner}^2/2$ = **69.4** kNm

Self weight and slab shear force — $V_{cornersw} = (w_{edgeult} + w_{cornerslab}) \times l_{corner}$ = **52.5** kN

Moment and shear due to udls

Maximum ultimate udl — $w_{cornerudl} = ((1.4\times w_{Dudl})+(1.6\times w_{Ludl})) \times \phi_{depthick}/\sqrt{(2)}$ = **21.8** kN/m

Bending moment — $M_{cornerudl} = w_{cornerudl} \times l_{corner}^2/6$ = **25.4** kNm

Shear force — $V_{cornerudl} = w_{cornerudl} \times l_{corner}/2$ = **28.8** kN

Moment and shear due to line loads in x direction

Bending moment — $M_{cornerlinex} = w_{\Sigma ultlinex} \times l_{corner}^2/2$ = **129.9** kNm

Shear force — $V_{cornerlinex} = w_{\Sigma ultlinex} \times l_{corner}$ = **98.2** kN

Moment and shear due to line loads in y direction

Bending moment — $M_{cornerliney} = w_{\Sigma ultliney} \times l_{corner}^2/2$ = **129.9** kNm

Shear force — $V_{cornerliney} = w_{\Sigma ultliney} \times l_{corner}$ = **98.2** kN

Total moments and shears due to point loads

Bending moment about x axis — $M_{cornerpointx}$ = **0.0** kNm

Bending moment about y axis — $M_{cornerpointy}$ = **0.0** kNm

Shear force — $V_{cornerpoint}$ = **0.0** kN

Resultant moments and shears

Total moment about x axis — $M_{\Sigma cornerx} = M_{cornersw}+ M_{cornerudl}+ M_{cornerliney}+ M_{cornerpointx}$ = **224.6** kNm

Total shear force about x axis — $V_{\Sigma cornerx} = V_{cornersw}+ V_{cornerudl}+ V_{cornerliney} + V_{cornerpoint}$ = **179.5** kN

Total moment about y axis $M_{\Sigma cornery} = M_{cornersw} + M_{cornerudl} + M_{cornerlinex} + M_{cornerpointy} = \mathbf{224.6}$ kNm
Total shear force about y axis $V_{\Sigma cornery} = V_{cornersw} + V_{cornerudl} + V_{cornerlinex} + V_{cornerpoint} = \mathbf{179.5}$ kN
Deflection of both edge beams at corner will be the same therefore design for average of these moments and shears
Design bending moment $M_{\Sigma corner} = (M_{\Sigma cornerx} + M_{\Sigma cornery})/2 = \mathbf{224.6}$ kNm
Design shear force $V_{\Sigma corner} = (V_{\Sigma cornerx} + V_{\Sigma cornery})/2 = \mathbf{179.5}$ kN

Reinforcement required in top of edge beam

K factor $K_{corner} = M_{\Sigma corner}/(f_{cu} \times b_{edgetop} \times d_{edgetop}^2) = \mathbf{0.030}$
Lever arm $z_{corner} = d_{edgetop} \times \min(0.95, 0.5 + \sqrt{(0.25 - K_{corner}/0.9)}) = \mathbf{523}$ mm
Area of steel required for bending $A_{scornerbend} = M_{\Sigma corner}/((1.0/\gamma_s) \times f_y \times z_{corner}) = \mathbf{989}$ mm^2
Minimum area of steel required $A_{scornermin} = A_{sedgetopmin} = \mathbf{585}$ mm^2
Area of steel required $A_{scorner} = \max(A_{scornerbend}, A_{scornermin}) = \mathbf{989}$ mm^2

PASS - $A_{scorner} <= A_{sedgetop}$ - Area of reinforcement provided in top of edge beams at corners is adequate

Corner beam shear check

Average web width $b_w = b_{edge} + (h_{edge}/\tan(\alpha_{edge}))/2 = \mathbf{750}$ mm
Applied shear stress $v_{corner} = V_{\Sigma corner}/(b_w \times d_{edgetop}) = \mathbf{0.435}$ N/mm^2
Tension steel ratio $\rho_{corner} = 100 \times A_{sedgetop}/(b_w \times d_{edgetop}) = \mathbf{0.305}$
From BS8110-1:1997 - Table 3.8
Design concrete shear strength $v_{ccorner} = \mathbf{0.439}$ N/mm^2

$v_{corner} <= v_{ccorner}$ + 0.4N/mm^2 - Therefore minimum links required

Link area to spacing ratio required $A_{sv\_upon\_Svreqcorner} = 0.4\text{N/mm}^2 \times b_w/((1.0/\gamma_s) \times f_{ys}) = \mathbf{0.690}$ mm
Link area to spacing ratio provided $A_{sv\_upon\_Svprovedge} = N_{edgelink} \times \pi \times \phi_{edgelink}^2/(4 \times S_{vedge}) = \mathbf{0.698}$ mm

PASS - $A_{sv\_upon\_Svreqcorner} <= A_{sv\_upon\_Svprovedge}$ - Shear reinforcement provided in edge beams at corners is adequate

Corner beam deflection check

Basic allowable span to depth ratio $Ratio_{basiccorner} = \mathbf{7.0}$
Moment factor $M_{factorcorner} = M_{\Sigma corner}/(b_{edgetop} \times d_{edgetop}^2) = \mathbf{1.061}$ N/mm^2
Steel service stress $f_{scorner} = 2/3 \times f_y \times A_{scornerbend}/A_{sedgetop} = \mathbf{262.268}$ N/mm^2
Modification factor $MF_{corner}=\min(2.0,0.55+[(477\text{N/mm}^2-f_{scorner})/(120\times(0.9\text{N/mm}^2+M_{factorcorner}))])$
$MF_{corner} = \mathbf{1.463}$
Modified allowable span to depth ratio $Ratio_{allowcorner} = Ratio_{basiccorner} \times MF_{corner} = \mathbf{10.238}$
Actual span to depth ratio $Ratio_{actualcorner} = l_{corner}/ d_{edgetop} = \mathbf{4.807}$

PASS - $Ratio_{actualcorner} <= Ratio_{allowcorner}$ - Edge beam span to depth ratio is adequate

<u>Internal beam design checks</u>

Basic loading

Hardcore $w_{hcorethick} = \gamma_{hcore} \times h_{hcorethick} = \mathbf{3.0}$ kN/m^2
Internal beam self weight $w_{int} = 24$ kN/m$^3 \times h_{int} \times b_{int} = \mathbf{6.5}$ kN/m

Internal beam load number 1

Load type **Longitudinal line load**
Dead load $w_{Dint1} = \mathbf{13.2}$ kN/m
Live load $w_{Lint1} = \mathbf{0.0}$ kN/m
Ultimate load $w_{ultint1} = 1.4 \times w_{Dint1} + 1.6 \times w_{Lint1} = \mathbf{18.5}$ kN/m
Longitudinal line load width $b_{int1} = \mathbf{300}$ mm
Centroid of load from centreline of beam $x_{int1} = \mathbf{150}$ mm

Internal beam bearing pressure check

Total uniform load at formation level $w_{udlint} = w_{Dudl}+w_{Ludl}+w_{hcorethick}+24\text{kN/m}^3\times h_{int} = \mathbf{23.4}$ kN/m^2
Sum of factored longitud'l and eff'tive line loads $\Sigma UDL_{int} = \mathbf{18.5}$ kN/m
Sum of unfactored longitud'l and eff'tive line loads $\Sigma UDLsls_{int} = \mathbf{13.2}$ kN/m

Centroid of loads from centreline of internal beam

Load x distance for internal load 1 $Moment_{int1} = w_{ultint1} \times x_{int1} = \mathbf{2.8}$ kN
Sum of load x distances $\Sigma Moment_{int} = \mathbf{2.8}$ kN
Centroid of loads $x_{barint} = \Sigma Moment_{int}/\Sigma UDL_{int} = \mathbf{150.0}$ mm
Moment due to eccentricity to be resisted by slab $M_{eccint} = \Sigma UDL_{int} \times \text{abs}(x_{barint}) = \mathbf{2.8}$ kNm/m
No slab bottom reinforcement therefore moment due to eccentricity is resisted by top steel of slab on one side only
From slab bending check
Moment due to depression under slab (hogging) $M_{\Sigma e} = \mathbf{15.0}$ kNm/m

Total moment to be resisted by slab top steel	$M_{slabtopint} = M_{eccint} + M_{\Sigma e} = \mathbf{17.8}$ kNm/m
K factor	$K_{slabtopint} = M_{slabtopint}/(f_{cu} \times d_{tslabmin}^2) = \mathbf{0.019}$
Lever arm	$z_{slabtopint} = d_{tslabmin} \times \min(0.95, 0.5 + \sqrt{(0.25 - K_{slabtopint}/0.9)}) = \mathbf{157}$ mm
Area of steel required	$A_{sslabtopintreq} = M_{slabtopint}/((1.0/\gamma_S) \times f_{yslab} \times z_{slabtopint}) = \mathbf{261}$ mm^2/m

PASS - $A_{sslabtopintreq}$ <= $A_{sslabtop}$ - Area of reinforcement in top of slab is adequate to transfer moment into slab

Bearing pressure

Allowable bearing width	$b_{bearint} = b_{int} = \mathbf{450}$ mm
Bearing pressure due to line/point loads	$q_{linepointint} = \Sigma UDLsls_{int}/b_{bearint} = \mathbf{29.3}$ kN/m^2
Total applied bearing pressure	$q_{int} = q_{linepointint} + w_{udlint} = \mathbf{52.7}$ kN/m^2

PASS - q_{int} <= q_{allow} - Allowable bearing pressure is not exceeded

Internal beam bending check

Divider for moments due to udl's	$\beta_{udl} = \mathbf{10.0}$

Applied bending moments

Span of internal beam	$l_{int} = \phi_{depthick} + d_{inttop} = \mathbf{3902}$ mm
Ultimate self weight udl	$w_{intult} = 1.4 \times w_{int} = \mathbf{9.1}$ kN/m
Ultimate slab udl (approx)	$w_{intslab} = \max(0$ kN/m$, 1.4 \times w_{slab} \times ((\phi_{depthick} \times 3/4) - b_{int})) = \mathbf{13.9}$ kN/m
Self weight and slab bending moment	$M_{intsw} = (w_{intult} + w_{intslab}) \times l_{int}^2/\beta_{udl} = \mathbf{34.9}$ kNm
Self weight shear force	$V_{intsw} = (w_{intult} + w_{intslab}) \times l_{int}/2 = \mathbf{44.7}$ kN

Moments due to applied uniformly distributed loads

Ultimate udl (approx)	$w_{intudl} = w_{udlult} \times \phi_{depthick} \times 3/4 = \mathbf{23.1}$ kN/m
Bending moment	$M_{intudl} = w_{intudl} \times l_{int}^2/\beta_{udl} = \mathbf{35.2}$ kNm
Shear force	$V_{intudl} = w_{intudl} \times l_{int}/2 = \mathbf{45.1}$ kN

Moment and shear due to load number 1

Bending moment	$M_{int1} = w_{ultint1} \times l_{int}^2/\beta_{udl} = \mathbf{28.1}$ kNm
Shear force	$V_{int1} = w_{ultint1} \times l_{int}/2 = \mathbf{36.1}$ kN

Resultant moments and shears

Total moment (hogging and sagging)	$M_{\Sigma int} = \mathbf{98.2}$ kNm
Maximum shear force	$V_{\Sigma int} = \mathbf{125.9}$ kN

Reinforcement required in top

Width of section in compression zone	$b_{inttop} = b_{int} = \mathbf{450}$ mm
Average web width	$b_{wint} = b_{int} = \mathbf{450}$ mm
K factor	$K_{inttop} = M_{\Sigma int}/(f_{cu} \times b_{inttop} \times d_{inttop}^2) = \mathbf{0.020}$
Lever arm	$z_{inttop} = d_{inttop} \times \min(0.95, 0.5 + \sqrt{(0.25 - K_{inttop}/0.9)}) = \mathbf{524}$ mm
Area of steel required for bending	$A_{sinttopbend} = M_{\Sigma int}/((1.0/\gamma_S) \times f_y \times z_{inttop}) = \mathbf{431}$ mm^2
Minimum area of steel	$A_{sinttopmin} = 0.0013 \times b_{wint} \times h_{int} = \mathbf{351}$ mm^2
Area of steel required	$A_{sinttopreq} = \max(A_{sinttopbend}, A_{sinttopmin}) = \mathbf{431}$ mm^2

PASS - $A_{sinttopreq}$ <= $A_{sinttop}$ - Area of reinforcement provided in top of internal beams is adequate

Reinforcement required in bottom

Width of section in compression zone	$b_{intbtm} = b_{int} + 0.2 \times l_{int} = \mathbf{1230}$ mm
K factor	$K_{intbtm} = M_{\Sigma int}/(f_{cu} \times b_{intbtm} \times d_{intbtm}^2) = \mathbf{0.008}$
Lever arm	$z_{intbtm} = d_{intbtm} \times \min(0.95, 0.5 + \sqrt{(0.25 - K_{intbtm}/0.9)}) = \mathbf{520}$ mm
Area of steel required for bending	$A_{sintbtmbend} = M_{\Sigma int}/((1.0/\gamma_S) \times f_y \times z_{intbtm}) = \mathbf{435}$ mm^2
Minimum area of steel required	$A_{sintbtmmin} = 0.0018 \times 1.0 \times b_{wint} \times h_{int} = \mathbf{486}$ mm^2
Area of steel required	$A_{sintbtmreq} = \max(A_{sintbtmbend}, A_{sintbtmmin}) = \mathbf{486}$ mm^2

PASS - $A_{sintbtmreq}$ <= $A_{sintbtm}$ - Area of reinforcement provided in bottom of internal beams is adequate

Internal beam shear check

Applied shear stress	$v_{int} = V_{\Sigma int}/(b_{wint} \times d_{inttop}) = \mathbf{0.507}$ N/mm^2
Tension steel ratio	$\rho_{int} = 100 \times A_{sinttop}/(b_{wint} \times d_{inttop}) = \mathbf{0.243}$
From BS8110-1:1997 - Table 3.8	
Design concrete shear strength	$v_{cint} = \mathbf{0.407}$ N/mm^2

v_{int} <= v_{cint} + 0.4N/mm^2 - Therefore minimum links required

Link area to spacing ratio required	$A_{sv_upon_s_{vreqint}} = 0.4N/mm^2 \times b_{wint}/((1.0/\gamma_S) \times f_{ys}) = \mathbf{0.414}$ mm
Link area to spacing ratio provided	$A_{sv\_upon\_s_{vprovint}} = N_{intlink} \times \pi \times \phi_{intlink}^2/(4 \times s_{vint}) = \mathbf{0.628}$ mm

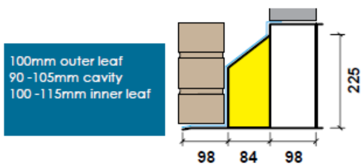
PASS - $A_{sv\_upon\_s_{vreqint}}$ <= $A_{sv\_upon\_s_{vprovint}}$ - Shear reinforcement provided in internal beams is adequate

LINTELS

Lintel L1

Dead udl from	outer leaf	=	0.40	x	2.20	=	0.88	kN/m
Dead udl from	inner leaf	=	0.40	x	2.20	=	0.88	kN/m
Dead udl from	pitched roof	=	6.10	x	0.56	=	3.41	kN/m
Dead udl from	ceiling	=	6.10	x	0.35	=	2.13	kN/m
Imposed udl from	pitched roof	=	6.10	x	0.60	=	3.66	kN/m
Imposed udl from	ceiling	=	6.10	x	0.25	=	1.52	kN/m
	Total Load	=	12.49	x	2.79	=	34.78	kN

SL90 XHD 225



Suitable to support precast concrete floors, attic trusses, and point loads.

STANDARD LENGTHS (mm) Lintels are available in increments of 150mm	600 2100	2250 2700	2850 3300	3450 3900	4050 4200
Nominal Height "h" (mm)	225	225	225	225	225
Weights (kg/m)	27.8	27.8	27.8	27.8	27.8
SWL 5:1 (kN)	79	60	49	41	40
SWL 19:1 (kN)	66	51	41	34	33
RM (kNm)	20.6	20.6	20.6	20.6	20.6

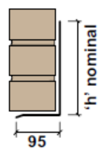
Lintel L2

Dead udl from	outer leaf	=	0.45	x	2.20	=	0.99	kN/m
	Total Load	=	0.99	x	2.79	=	2.76	kN

STANDARD LENGTHS (mm) Lintels are available in increments of 150mm	600 1500	1650 1800	1950 2400	2550 3000	3150 3900
Nominal Height "h" (mm)	115	115	165	225	225
Weights (kg/m)	3.3	5.0	6.2	7.6	7.6
SWL (kN)	8	12	12	17	15
RM (kNm)	1.4	1.4	3.6	5.8	5.8

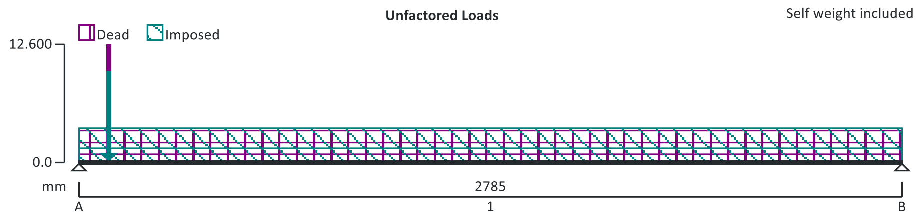
See additional installation requirements on page 42.

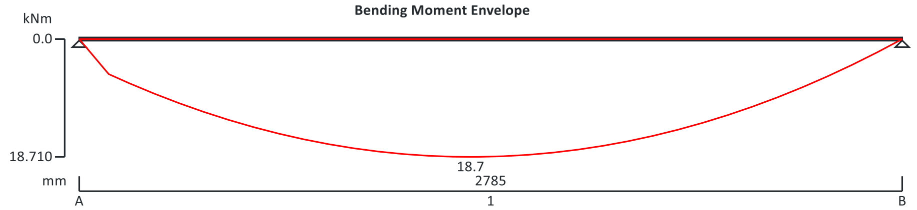
SL100 TR

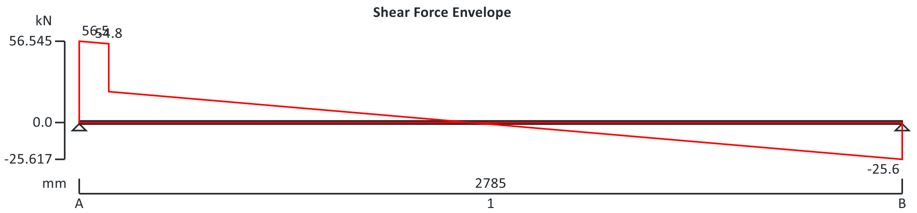


BEAM

Dead udl from	inner leaf	=	0.40	x	2.20	=	0.88	kN/m	
Dead udl from	pitched roof	=	6.10	x	0.56	=	3.41	kN/m	
Dead udl from	ceiling	=	6.10	x	0.35	=	2.13	kN/m	
Imposed udl from	pitched roof	=	6.10	x	0.60	=	3.66	kN/m	
Imposed udl from	ceiling	=	6.10	x	0.25	=	1.52	kN/m	
Dead PL from	GT HT04g					=	12.60	kN	100mm
Imposed PL from	GT HT04g					=	9.80	kN	100mm







Support conditions

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

Analysis results

Maximum moment	M_{max} = **18.7** kNm	M_{min} = **0** kNm
Maximum shear	V_{max} = **56.5** kN	V_{min} = **-25.6** kN

Deflection	δ_{max} = **3.7** mm	δ_{min} = **0** mm
Maximum reaction at support A	R_{A_max} = **56.5** kN	R_{A_min} = **56.5** kN
Unfactored dead load reaction at support A	R_{A_Dead} = **21.3** kN	
Unfactored imposed load reaction at support A	$R_{A\_Imposed}$ = **16.7** kN	
Maximum reaction at support B	R_{B_max} = **25.6** kN	R_{B_min} = **25.6** kN
Unfactored dead load reaction at support B	R_{B_Dead} = **9.7** kN	
Unfactored imposed load reaction at support B	$R_{B\_Imposed}$ = **7.6** kN	

Section details

Section type — **UB 178x102x19 (British Steel Section Range 2022 (BS4-1))**

Steel grade — **S355**

From table 9: Design strength p_y

Thickness of element — max(T, t) = **7.9** mm

Design strength — p_y = **355** N/mm^2

Modulus of elasticity — E = **205000** 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 — $K_{LT.A}$ = **1.40** + 2 × D

$K_{LT.B}$ = **1.40** + 2 × D

Classification of cross sections - Section 3.5

$\varepsilon = \sqrt{[275 \text{ N/mm}^2 / p_y]}$ = **0.88**

Internal compression parts - Table 11

Depth of section — d = **146.8** mm

d / t = 34.7 × ε <= 80 × ε — Class 1 plastic

Outstand flanges - Table 11

Width of section — b = B / 2 = **50.6** mm

b / T = 7.3 × ε <= 9 × ε — Class 1 plastic

Section is class 1 plastic

Shear capacity - Section 4.2.3

Design shear force — $F_v = \max(\text{abs}(V_{max}), \text{abs}(V_{min}))$ = **56.5** kN

d / t < 70 × ε

Web does not need to be checked for shear buckling

Shear area — A_v = t × D = **853** mm^2

Design shear resistance — $P_v = 0.6 \times p_y \times A_v$ = **181.8** kN

PASS - Design shear resistance exceeds design shear force

Moment capacity - Section 4.2.5

Design bending moment — $M = \max(\text{abs}(M_{s1\_max}), \text{abs}(M_{s1\_min}))$ = **18.7** kNm

Moment capacity low shear - cl.4.2.5.2 — $M_c = \min(p_y \times S_{xx}, 1.2 \times p_y \times Z_{xx})$ = **60.8** kNm

Effective length for lateral-torsional buckling - Section 4.3.5

Effective length for lateral torsional buckling — $L_E = 1.4 \times L_{s1} + 2 \times D$ = **4255** mm

Slenderness ratio — $\lambda = L_E / r_{yy}$ = **179.235**

Equivalent slenderness - Section 4.3.6.7

Buckling parameter — u = **0.888**

Torsional index — x = **22.560**

Slenderness factor — $v = 1 / [1 + 0.05 \times (\lambda / x)^2]^{0.25}$ = **0.700**

Ratio - cl.4.3.6.9 — β_W = **1.000**

Equivalent slenderness - cl.4.3.6.7 — $\lambda_{LT} = u \times v \times \lambda \times \sqrt{[\beta_W]}$ = **111.429**

Limiting slenderness - Annex B.2.2 — $\lambda_{L0} = 0.4 \times (\pi^2 \times E / p_y)^{0.5}$ = **30.198**

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

Bending strength - Section 4.3.6.5

Robertson constant — α_{LT} = **7.0**

Perry factor — $\eta_{LT} = \max(\alpha_{LT} \times (\lambda_{LT} - \lambda_{L0}) / 1000, 0)$ = **0.569**

Euler stress — $p_E = \pi^2 \times E / \lambda_{LT}^2$ = **163** N/mm^2

$\phi_{LT} = (p_y + (\eta_{LT} + 1) \times p_E) / 2 =$ **305.3** N/mm^2

Bending strength - Annex B.2.1 — $p_b = p_E \times p_y / (\phi_{LT} + (\phi_{LT}^2 - p_E \times p_y)^{0.5}) =$ **117.3** N/mm^2

Equivalent uniform moment factor - Section 4.3.6.6

Equivalent uniform moment factor for LTB — $m_{LT} =$ **1.000**

Buckling resistance moment - Section 4.3.6.4

Buckling resistance moment — $M_b = p_b \times S_{xx} =$ **20.1** kNm

$M_b / m_{LT} =$ **20.1** kNm

PASS - Buckling resistance moment exceeds design bending moment

Check vertical deflection - Section 2.5.2

Consider deflection due to dead and imposed loads

Limiting deflection — $\delta_{lim} = L_{s1} / 250 =$ **11.14** mm

Maximum deflection span 1 — $\delta = \max(\mathrm{abs}(\delta_{max}), \mathrm{abs}(\delta_{min})) =$ **3.712** mm

PASS - Maximum deflection does not exceed deflection limit

BEARINGS

Bearing A

Masonry details			
Masonry type	**Aggregate concrete blocks (25% or less formed voids)**		
Compressive strength	p_{unit} = **3.6** N/mm^2	Mortar designation	**iii**
Least horiz dim of units	l_{unit} = **100** mm	Height of units	h_{unit} = **215** mm
Masonry units	**Category II**	Construction control	**Normal**
Partial safety factor	γ_m = **3.5**	Characteristic strength	f_k = **3.5** N/mm^2
Leaf thickness	t = **100** mm	Effective wall thickness	t_{ef} = **100** mm
Wall height	h = **2400** mm	Effective height of wall	h_{ef} = **2400** mm
Bearing details			
Beam spanning in plane of wall			
Width of bearing	B = **100** mm	Length of bearing	l_b = **350** mm
Loading details			
Concentrated dead load	G_k = **20** kN	Concentrated imposed load	Q_k = **15** kN
Design concentrated load	F = **52.0** kN		
Distributed dead load	g_k = **0.0** kN/m	Distributed imposed load	q_k = **0.0** kN/m
Design distributed load	f = **0.0** kN/m		
Masonry bearing type			
Bearing type	**Not applicable**	Bearing safety factor	γ_{bear} = **1.00**
Check design bearing without a spreader			
Design bearing stress	f_{ca} = **1.486** N/mm^2	Allowable bearing stress	f_{cp} = **1.000** N/mm^2

FAIL - Design bearing stress exceeds allowable bearing stress, use a spreader

Spreader details			
Length of spreader	l_s = **600** mm	Depth of spreader	h_s = **215** mm
Edge distance	s_{edge} = **0** mm		
Spreader bearing type			
Bearing type	**Type 3**	Bearing safety factor	γ_{bear} = **2.00**
Check design bearing with a spreader			
Loading acts eccentrically outside middle third – triangular stress distribution			
Design bearing stress	f_{ca} = **1.981** N/mm^2	Allowable bearing stress	f_{cp} = **2.000** N/mm^2

PASS - Allowable bearing stress exceeds design bearing stress

Check design bearing at 0.4 × h below the bearing level			
Design bearing stress	f_{ca} = **0.397** N/mm^2	Allowable bearing stress	f_{cp} = **0.605** N/mm^2

PASS - Allowable bearing stress at 0.4 × h below bearing level exceeds design bearing stress

Bearing B

Masonry details

Masonry type	**Aggregate concrete blocks (25% or less formed voids)**		
Compressive strength	p_{unit} = **3.6** N/mm^2	Mortar designation	**iii**
Least horiz dim of units	l_{unit} = **100** mm	Height of units	h_{unit} = **215** mm
Masonry units	**Category II**	Construction control	**Normal**
Partial safety factor	γ_m = **3.5**	Characteristic strength	f_k = **3.5** N/mm^2
Leaf thickness	t = **100** mm	Effective wall thickness	t_{ef} = **100** mm
Wall height	h = **2400** mm	Effective height of wall	h_{ef} = **2400** mm
Bearing details			
Beam spanning in plane of wall			
Width of bearing	B = **100** mm	Length of bearing	l_b = **200** mm
Loading details			
Concentrated dead load	G_k = **10** kN	Concentrated imposed load	Q_k = **8** kN
Design concentrated load	F = **25.7** kN		
Distributed dead load	g_k = **0.0** kN/m	Distributed imposed load	q_k = **0.0** kN/m
Design distributed load	f = **0.0** kN/m		
Masonry bearing type			
Bearing type	**Type 2**	Bearing safety factor	γ_{bear} = **1.50**
Check design bearing without a spreader			
Design bearing stress	f_{ca} = **1.287** N/mm^2	Allowable bearing stress	f_{cp} = **1.500** N/mm^2

PASS - Allowable bearing stress exceeds design bearing stress

Check design bearing at 0.4 × h below the bearing level

Design bearing stress	f_{ca} = **0.222** N/mm^2	Allowable bearing stress	f_{cp} = **0.605** N/mm^2

PASS - Allowable bearing stress at 0.4 × h below bearing level exceeds design bearing stress