

PLAN IT JUPITER

STRUCTURAL
DESIGN & DETAIL

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Hillside, Hillside Road, Sutton Veny BA12 7AT
Stable Block

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Sept. 2026

Structural Calculations.

Any Architectural drawings and setting out information provided by others.

The dimensions used within these calculations are for design purposes only. Detailed measurements are to be taken from site by the building contractor and in accordance with the Architects Drawings. Should any major inconsistencies occur inform the Engineer.

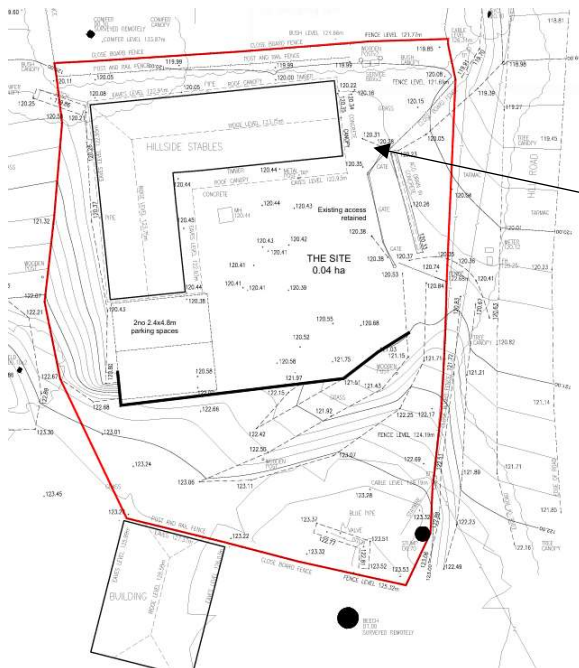
The contractor is to act as the temporary works coordinator and should be competent and experienced enough to carry out the design and installation of temporary works to BS 5975 in order to maintain stability of the building and excavations.

If these calculations are undertaken at "Planning stage" and prior to Architectural Building Regulation approval being available, and as such some detail may need to be clarified with the Architect and building control prior to construction. The designs herein should be adequate for the Architect to develop their proposals, Tender process and for Building Regulation submission related to structure only. It is advised that calculations are checked and signed off by building control prior to commencement on site.

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Design Brief.

This report is to review the structure with a view to the potential for conversion.



Stables:

The stables are a timber framed construction built off a substantial concrete slab, in an "L" shape on plan, with trussed rafter frames supporting roof finishes.



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Summary related for Planning

Here are the relevant policy tests from Policy 48 of the Wiltshire Core Strategy, and the NPPF S5:1.c:

Conversion and re-use of rural buildings

Proposals to convert and re-use rural buildings for employment, tourism, cultural and community uses will be supported where they satisfy the following criteria:

- i. The building(s) is/are structurally sound and capable of conversion without major rebuilding, and with only necessary extension or modification which preserves the character of the original building.

S5: Principle of development outside settlements

1. Only certain forms of development should be approved outside settlements, as set out in the following list. These should be approved, unless the benefits of doing so would be substantially outweighed by any adverse effects, when assessed against the national decision-making policies in this Framework:
 - c. The reuse, extension, alteration or replacement of an existing building, provided that the existing building is of permanent and substantial construction, is lawful in planning terms, and any extension or alteration will not result in a disproportionate increase in size compared to the existing building²⁵. In the case of proposals for a replacement building, it should be for the same use and not disproportionately larger than the one it replaces;

I can confirm that those thresholds will not be breached following the guidance in this structural report.

This development does not require 'substantial reconstruction' in compliance with local plan policy.



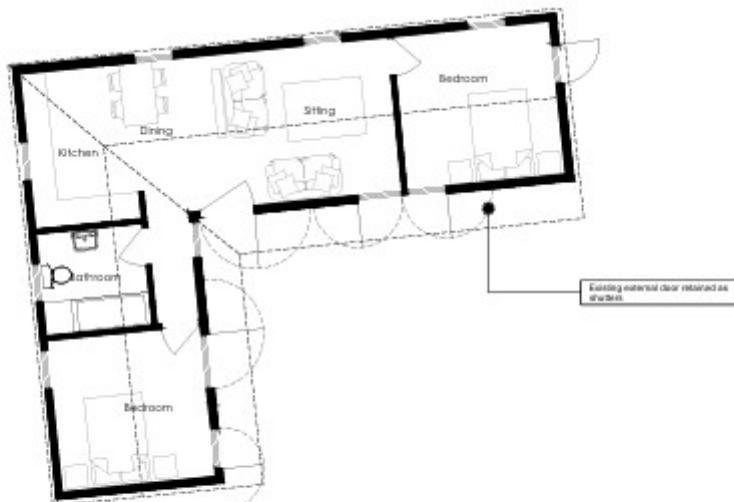
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Floor Plan
Scale 1:100
EXISTING



Floor Plan
Scale 1:100
PROPOSED



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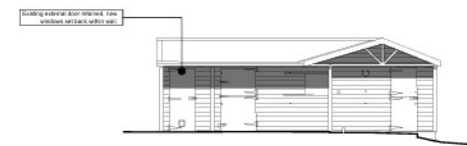
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South Elevation
Scale 1:100



East Elevation
Scale 1:100



North Elevation
Scale 1:100



West Elevation
Scale 1:100

ROOF ARRANGEMENT



Trussed rafters supporting roof sheets via purlins / boarding.



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By inspection, Roof member sizes on site are acceptable and capable of coping with any additional roof loads as anticipated with conversion and the use of light weight insulated roofing sheets help with racking resistance.

In our opinion, the trussed rafters and their spacing is acceptable should any additional ceiling and insulation were required for the conversion.

We suggest that any new ceilings should span clear parallel with the existing trusses.

LOADINGS

Roof Loading.

Dead Load

Insulated sheets;	=	0.25 kN/m²
Top hats and closers;	=	0.05 kN/m²
purlins;	=	0.30 kN/m²
Dead Load on slope;	=	0.60 kN/m²
Dead Load (on plan);	=	0.95 kN/m²
Guttering, services etc;	=	0.15 kN/m²
Total Dead Load (on plan);	=	1.10 kN/m²

Imposed Load

Roof Imposed Load (on plan);	=	0.60 kN/m²
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Wall Loading (assumed, timber clad timber frame).

Dead Load

Cladding and sheathing;	=	0.30 kN/m²
Insulation & studs;	=	0.15 kN/m²
Services and sheathing inner;	=	0.30 kN/m²
Plasterboard & Skim;	=	0.15 kN/m²
Total Dead Load;	=	0.90 kN/m²

WIND LOADING (BS6399)



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OBSERVATIONS



In our opinion the condition of the existing structural elements of the stables are in good condition generally, although some local signs of damage to studs is noted within stable stalls.

These very local areas can be dealt with during the conversion works.
It is considered that making good and general maintenance will be undertaken either prior to conversion or during the conversion phase.

The cladding is providing some racking resistance at roof level, and to the walls.
The cladding will undoubtedly be repaired where needed and augmented as part of the conversion related to insulation, ventilation and vapour control requirements.

Although the stables are utilising internal division stall walls for buttressing to perimeter walls it is considered that the new arrangement of walls for the conversion will also provide sufficient buttressing for the perimeter walls.

The introduction of OSB and plasterboard will also increase such racking resistance.

Any new openings will require appropriate double studs and timber lintels as part of the conversion and the actual sizes will need to be determined related to the opening widths required.

The stud walls are constructed in appropriate panels and these are secured together where they abut.

Typically the trusses span onto perimeter wall plates onto the panels and are considered to be on average approx 1.2m spacings.



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At the corner intersection of the stables the trusses are curtailed and supported from a diagonal truss forming the hip and valley. All acceptable.

The stables currently have a concrete slab floor, in very good condition, with estimated thickness of 200mm and considering previous storage and use, loading onto slab is likely to be less for domestic loading including any additional finishes, insulation and internal partitioning.

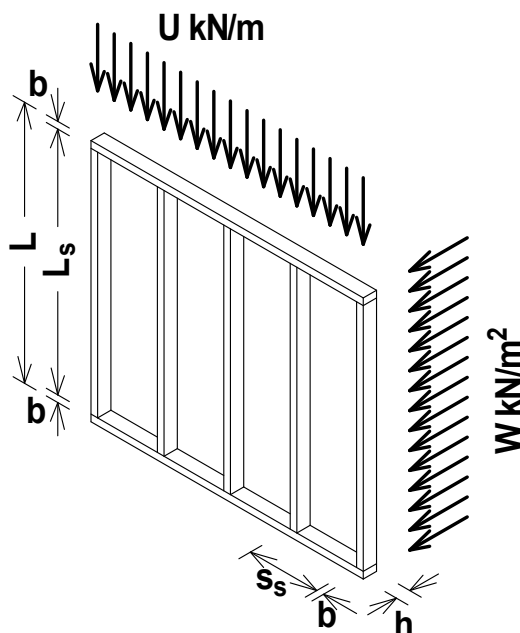
Trial pits reveal the slab depth and prove that the raft slab is built on the anticipated chalk formation related to the review of Geological survey data suggesting Melbury Chalk formation.

It is anticipated that the raft is reinforced with minimum reinforcement related to its current loading related to perimeter and internal loadings.

Structural check calculations.

Check existing Studs for proposed loads.

Timber stud Design (bs5268-2:2002)



Stud details

Stud breadth; $b = 40 \text{ mm}$

Stud depth; $h = 75 \text{ mm}$

Number of studs; $N_s = 1$



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Strength class C24 timber (Table 8 BS5268:Pt 2:2002)

Section properties

Cross sectional area; $A = N_s \times b \times h = 3000 \text{ mm}^2$

Section modulus; $Z = N_s \times b \times h^2 / 6 = 37500 \text{ mm}^3$

Moment of inertia in the major axis; $I_x = N_s \times b \times h^3 / 12 = 1406250 \text{ mm}^4$

Moment of inertia in the minor axis; $I_y = N_s \times h \times b^3 / 12 = 400000 \text{ mm}^4$

Radius of gyration in the major axis; $r_x = \sqrt{I_x / A} = 21.7 \text{ mm}$

Radius of gyration in the minor axis; $r_y = \sqrt{I_y / A} = 11.5 \text{ mm}$

Panel details - Studs restrained by sheathing in the plane of the panel

Panel height; $L = 2400 \text{ mm}$

Stud length; $L_s = L - (2 \times b) = 2320 \text{ mm}$

Standard stud spacing; $s_s = 600 \text{ mm}$

Panel opening; $O = 0 \text{ mm}$

Loaded panel length; $s = \max(s_s, (O + s_s) / 2) = 600 \text{ mm}$

Effective length in the major axis; $L_{ex} = 0.85 \times L_s = 1972 \text{ mm}$

Slenderness ratio; $l = L_{ex} / r_x = 91.08$

Vertical loading details

Dead loads

Imposed loads

Wall UDL; $U_{w,d} = 1.00 \text{ kN/m}$

Roof UDL; $U_{r,d} = 2.20 \text{ kN/m}; U_{r,i} = 1.20 \text{ kN/m}$

Lateral loading details

Wind loading; $W = 0.60 \text{ kN/m}^2$

Wind load duration; **Very short term**

Modification factors

Section depth factor; $K_7 = (300 \text{ mm} / h)^{0.11} = 1.16$

Load sharing factor; $K_8 = 1.10$

Consider combined axial compression and bending under very short term loads

Load duration factor; $K_3 = 1.75$

Vertical loading; $F = (U_{w,d} + U_{r,d} + U_{r,i}) \times s = 2.64 \text{ kN}$

Check bending stress

Bending parallel to grain; $s_m = 7.500 \text{ N/mm}^2$

Permissible bending stress; $s_{m,adm} = s_m \times K_3 \times K_7 \times K_8 = 16.816 \text{ N/mm}^2$

Bending moment; $M_{max} = W \times s \times L^2 / 8 = 0.259 \text{ kNm}$

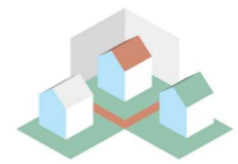
Applied bending stress; $s_{m,max} = M_{max} / Z = 6.912 \text{ N/mm}^2$

PASS - Applied bending stress under very short term loads is within permissible limits

Check compressive stress on stud

Compression member factor; $K_{12} = 0.32$

Compression parallel to grain; $s_c = 7.900 \text{ N/mm}^2$



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Permissible compressive stress; $S_{c_adm} = S_c \times K_3 \times K_8 \times K_{12} = 4.915 \text{ N/mm}^2$

Applied compressive stress; $S_{c_max} = F / (N_s \times b \times h) = 0.880 \text{ N/mm}^2$

PASS - Applied compressive stress under very short term loads is within permissible limits

Check compressive stress on rail

Bearing stress modification factor; $K_4 = 1.33$

Compression perpendicular to grain (no wane); $S_{cp1} = 2.400 \text{ N/mm}^2$

Permissible compressive stress; $S_{cp1_adm} = S_{cp1} \times K_3 \times K_4 = 5.586 \text{ N/mm}^2$

Applied compressive stress; $S_{cp1_max} = F / (N_s \times b \times h) = 0.880 \text{ N/mm}^2$

PASS - Applied compressive stress under very short term loads is within permissible limits

Check combined axial compression and bending

Euler critical stress; $S_e = (p^2 \times E_{min}) / l^2 = 8.566 \text{ N/mm}^2$

Euler coefficient; $K_{eu} = 1 - (1.5 \times S_{c_max} \times K_{12} / S_e) = 0.950$

Combined axial compression and bending value; $K = S_{m_max} / (S_{m_adm} \times K_{eu}) + S_{c_max} / S_{c_adm} = 0.612; < 1$

PASS - Combined compressive and bending stresses under very short term loads are within permissible limits

Check stud deflection

Euler critical stress; $S_e = (p^2 \times E_{min}) / l^2 = 8.566 \text{ N/mm}^2$

Maximum deflection; $d_{adm} = \min(7.0 \text{ mm}, 0.003 \times (L - 2 \times b)) = 6.960 \text{ mm}$

Bending deflection; $d_{max} = 0.005 \times l \times (S_{c_max} + S_{m_max}) / (S_e - S_{c_max}) \times (Z / A) = 5.771 \text{ mm}$

PASS - Deflection due to wind loading is less than permissible limit

Consider axial compression without bending under medium term loads

Load duration factor; $K_3 = 1.25$

Vertical loading; $F = (U_{w_d} + U_{r_d} + U_{r_i}) \times s = 2.64 \text{ kN}$

Check compressive stress on stud

Compression member factor; $K_{12} = 0.40$

Compression parallel to grain; $S_c = 7.900 \text{ N/mm}^2$

Permissible compressive stress; $S_{c_adm} = S_c \times K_3 \times K_8 \times K_{12} = 4.362 \text{ N/mm}^2$

Applied compressive stress; $S_{c_max} = F / (N_s \times b \times h) = 0.880 \text{ N/mm}^2$

PASS - Applied compressive stress under medium term loads is within permissible limits

Check compressive stress on rail

Bearing stress modification factor; $K_4 = 1.33$

Compression perpendicular to grain (no wane); $S_{cp1} = 2.400 \text{ N/mm}^2$

Permissible compressive stress; $S_{cp1_adm} = S_{cp1} \times K_3 \times K_4 = 3.990 \text{ N/mm}^2$

Applied compressive stress; $S_{cp1_max} = F / (N_s \times b \times h) = 0.880 \text{ N/mm}^2$

PASS - Applied compressive stress under medium term loads is within permissible limits

Consider axial compression without bending under long term loads



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Load duration factor; $K_3 = 1.00$

Vertical loading; $F = (U_{w,d} + U_{r,d}) \times s = 1.92 \text{ kN}$

Check compressive stress on stud

Compression member factor; $K_{12} = 0.45$

Compression parallel to grain; $s_c = 7.900 \text{ N/mm}^2$

Permissible compressive stress; $s_{c,adm} = s_c \times K_3 \times K_8 \times K_{12} = 3.925 \text{ N/mm}^2$

Applied compressive stress; $s_{c,max} = F / (N_s \times b \times h) = 0.640 \text{ N/mm}^2$

PASS - Applied compressive stress under long term loads is within permissible limits

Check compressive stress on rail

Bearing stress modification factor; $K_4 = 1.33$

Compression perpendicular to grain (no wane); $s_{cp1} = 2.400 \text{ N/mm}^2$

Permissible compressive stress; $s_{cp1,adm} = s_{cp1} \times K_3 \times K_4 = 3.192 \text{ N/mm}^2$

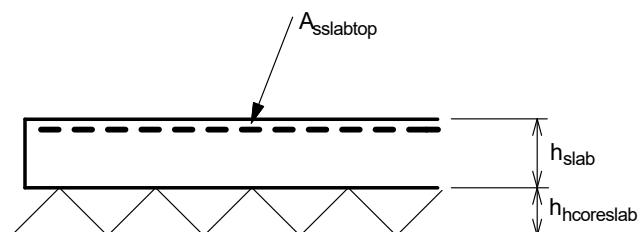
Applied compressive stress; $s_{cp1,max} = F / (N_s \times b \times h) = 0.640 \text{ N/mm}^2$

PASS - Applied compressive stress under long term loads is within permissible limits

We therefore conclude that the existing timber framed studwork is acceptable for conversion, but undoubtably will require some additional framing to suit any new openings and proposed layouts.

Check raft for loads

RAFT FOUNDATION DESIGN (BS8110 : PART 1 : 1997)



Soil and raft definition

Soil definition

Allowable bearing pressure;

$q_{allow} = 100.0 \text{ kN/m}^2$

Number of types of soil forming sub-soil;

One type only

Soil density;

Firm to loose

Depth of hardcore beneath slab;
pressure check)

$h_{hcoreslab} = 150 \text{ mm}$; (Dispersal allowed for bearing



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Density of hardcore; $g_{hcore} = 20.0 \text{ kN/m}^3$
 Basic assumed diameter of local depression; $f_{depbasic} = 2000 \text{ mm}$
 Diameter under slab modified for hardcore; $f_{depslab} = f_{depbasic} - h_{hcoreslab} = 1850 \text{ mm}$

Raft slab definition

Max dimension/max dimension between joints; $l_{max} = 12.000 \text{ m}$
 Slab thickness; $h_{slab} = 200 \text{ mm}$
 Concrete strength; $f_{cu} = 35 \text{ N/mm}^2$
 Poissons ratio of concrete; $n = 0.2$
 Slab mesh reinforcement strength; $f_{yslab} = 500 \text{ N/mm}^2$
 Partial safety factor for steel reinforcement; $g_s = 1.15$
 From C&CA document 'Concrete ground floors' Table 5
 Minimum mesh required in top for shrinkage; **A142;**
 Actual mesh provided in top; **A393 ($A_{sslabtop} = 393 \text{ mm}^2/\text{m}$)**
 Mesh bar diameter; $f_{slabtop} = 10 \text{ mm}$
 Cover to top reinforcement; $C_{top} = 25 \text{ mm}$
 Average effective depth of top reinforcement; $d_{tslabav} = h_{slab} - C_{top} - f_{slabtop} = 165 \text{ mm}$
 Minimum effective depth of top reinforcement; $d_{tslabmin} = d_{tslabav} - f_{slabtop}/2 = 160 \text{ mm}$

Slab edge reinforcement

Mesh provided in top; **A393 ($A_{sedgetop} = 393 \text{ mm}^2/\text{m}$)**
 ;

Internal slab design checks

Basic loading

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

Applied loading

Uniformly distributed dead load; $w_{Dudl} = 2.5 \text{ kN/m}^2$
 Uniformly distributed live load; $w_{Ludl} = 2.5 \text{ kN/m}^2$

Internal slab bearing pressure check

Total uniform load at formation level; $w_{udl} = w_{slab} + w_{hcoreslab} + w_{Dudl} + w_{Ludl} = 12.8 \text{ kN/m}^2$

PASS - $w_{udl} \leq q_{allow}$ - Applied bearing pressure is less than allowable

Internal slab bending and shear check

Applied bending moments

Effective span of slab; $l_{slab} = (f_{depslab} + d_{tslabav})/2 = 1008 \text{ mm}$
 Ultimate self weight udl; $w_{swult} = 1.4 \times w_{slab} = 6.7 \text{ kN/m}^2$
 Approximate self weight cantilever moment at edge; $M_{esw} = (w_{swult} \times p \times l_{slab}^2) \times (l_{slab}/3) / (2 \times p \times l_{slab}) = 1.1 \text{ kNm/m}$
 Self weight shear force at edge; $V_{sw} = w_{swult} \times l_{slab} / 2 = 3.4 \text{ kN/m}$

Moments due to applied uniformly distributed loads

Ultimate applied udl; $w_{udlult} = 1.4 \times w_{Dudl} + 1.6 \times w_{Ludl} = 7.5 \text{ kN/m}^2$



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Approximate cantilever moment at edge;
kNm/m

$$M_{edl} = (w_{udlult} \times p \times l_{slab}^2) \times (l_{slab}/3) / (2 \times p \times l_{slab}) = \mathbf{1.3}$$

Shear force at edge;

$$V_{udl} = w_{udlult} \times l_{slab} / 2 = \mathbf{3.8 \text{ kN/m}}$$

Resultant moments and shears

Total moment at edge;

$$M_{se} = \mathbf{2.4 \text{ kNm/m}}$$

Total shear force;

$$V_s = \mathbf{7.2 \text{ kN/m}}$$

Reinforcement required in top

K factor;

$$K_{slabtop} = M_{se} / (f_{cu} \times d_{slabav}^2) = \mathbf{0.003}$$

Lever arm;

$$Z_{slabtop} = d_{slabav} \times \min(0.95, 0.5 + \sqrt{(0.25 - K_{slabtop}/0.9)}) =$$

156.8 mm

Area of steel required for bending;

$$A_{slabtopbend} = M_{se} / ((1.0/g_s) \times f_{yslab} \times Z_{slabtop}) = \mathbf{35 \text{ mm}^2/\text{m}}$$

Minimum area of steel required;

$$A_{slabmin} = 0.0013 \times h_{slab} = \mathbf{260 \text{ mm}^2/\text{m}}$$

Area of steel required;

$$A_{slabtopreq} = \max(A_{slabtopbend}, A_{slabmin}) = \mathbf{260 \text{ mm}^2/\text{m}}$$

PASS - $A_{slabtopreq} \leq A_{slabtop}$ - Area of reinforcement provided in top to span local depressions is adequate

Shear check

Applied shear stress;

$$v = V_s / d_{slabmin} = \mathbf{0.045 \text{ N/mm}^2}$$

Tension steel ratio;

$$r = 100 \times A_{slabtop} / d_{slabmin} = \mathbf{0.246}$$

From BS8110-1:1997 - Table 3.8;

Design concrete shear strength;

$$v_c = \mathbf{0.557 \text{ N/mm}^2}$$

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

Internal slab deflection check

Basic allowable span to depth ratio;

$$\text{Ratio}_{\text{basic}} = \mathbf{7.0}$$

Moment factor;

$$M_{\text{factor}} = M_{se} / d_{slabav}^2 = \mathbf{0.088 \text{ N/mm}^2}$$

Steel service stress;

$$f_s = 2/3 \times f_{yslab} \times A_{slabtopbend} / A_{slabtop} = \mathbf{29.940 \text{ N/mm}^2}$$

Modification factor;

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

+ $M_{\text{factor}}))])$

$$MF_{\text{slab}} = \mathbf{2.000}$$

Modified allowable span to depth ratio;

$$\text{Ratio}_{\text{allow}} = \text{Ratio}_{\text{basic}} \times MF_{\text{slab}} = \mathbf{14.000}$$

Actual span to depth ratio;

$$\text{Ratio}_{\text{actual}} = l_{slab} / d_{slabav} = \mathbf{6.106}$$

PASS - $\text{Ratio}_{\text{actual}} \leq \text{Ratio}_{\text{allow}}$ - Slab span to depth ratio is adequate

Slab edge design checks

Basic loading

Hardcore;

$$w_{\text{coreslab}} = g_{\text{hcore}} \times h_{\text{coreslab}} = \mathbf{3.0 \text{ kN/m}^2}$$

Slab self weight;

$$w_{\text{slab}} = 24 \text{ kN/m}^3 \times h_{\text{slab}} = \mathbf{4.8 \text{ kN/m}^2}$$

Edge load number 1

Load type;

Longitudinal line load

Dead load;

$$w_{\text{Dedge1}} = \mathbf{5.0 \text{ kN/m}}$$

Live load;

$$w_{\text{Ledge1}} = \mathbf{2.0 \text{ kN/m}}$$

Ultimate load;

$$w_{\text{ultedge1}} = 1.4 \times w_{\text{Dedge1}} + 1.6 \times w_{\text{Ledge1}} = \mathbf{10.2 \text{ kN/m}}$$



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Longitudinal line load width;

$$b_{edge1} = 140 \text{ mm}$$

Centroid of load from outside face of raft;

$$x_{edge1} = 0 \text{ mm}$$

Slab edge bearing pressure check

Total uniform load at formation level;

$$W_{udledge} = W_{Dudl} + W_{Ludl} + W_{slab} + W_{hcoreslab} = 12.8 \text{ kN/m}^2$$

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

Load x distance for edge load 1;

$$\text{Moment}_1 = W_{ultedge1} \times x_{edge1} = 0.0 \text{ kN}$$

Sum of ultimate longitud'l and equivalent line loads;

$$SUDL = 10.2 \text{ kN/m}$$

Sum of load x distances;

$$SMoment = 0.0 \text{ kN}$$

Centroid of loads;

$$x_{bar} = SMoment/SUDL = 0 \text{ mm}$$

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

Sum of unfactored longitud'l and eff'tive line loads;

$$SUDLsls = 7.0 \text{ kN/m}$$

Allowable bearing width;

$$b_{allow} = 2 \times x_{bar} + 2 \times h_{hcoreslab} \times \tan(30) = 173 \text{ mm}$$

Bearing pressure due to line/point loads;

$$q_{linepoint} = SUDLsls / b_{allow} = 40.4 \text{ kN/m}^2$$

Total applied bearing pressure;

$$q_{edge} = q_{linepoint} + W_{udledge} = 53.2 \text{ kN/m}^2$$

PASS - $q_{edge} \leq q_{allow}$ - Allowable bearing pressure is not exceeded

Slab edge bending check

Considering a 1.0m width of slab

Applied bending moments

Cantilever span of slab;

$$l_{edge} = (f_{depslab} + d_{tslabmin})/2 = 1005 \text{ mm}$$

Ultimate self weight udl;

$$W_{edgeult} = 1.4 \times W_{slab} = 6.7 \text{ kN/m}^2$$

Self weight bending moment;

$$M_{edgesw} = W_{edgeult} \times l_{edge}^2/2 = 3.4 \text{ kNm/m}$$

Self weight shear force;

$$V_{edgesw} = W_{edgeult} \times l_{edge} = 6.8 \text{ kN/m}$$

Moments due to applied uniformly distributed loads

Ultimate udl;

$$W_{edgeudl} = W_{udlult} = 7.5 \text{ kN/m}^2$$

Bending moment;

$$M_{edgeudl} = W_{edgeudl} \times l_{edge}^2/2 = 3.8 \text{ kNm/m}$$

Shear force;

$$V_{edgeudl} = W_{edgeudl} \times l_{edge} = 7.5 \text{ kN/m}$$

Moment and shear due to load number 1

Effective slab width;

$$b_{eff1} = \min\{x_{edge1}, l_{edge}/2 + b_{edge1}/2\} + l_{edge}/2 + b_{edge1}/2 =$$

573 mm

Bending moment;

$$M_{edge1} = W_{ultedge1} \times l_{edge}^2/(2 \times b_{eff1}) = 9.0 \text{ kNm/m}$$

Shear force;

$$V_{edge1} = W_{ultedge1} \times l_{edge}/b_{eff1} = 17.9 \text{ kN/m}$$

Resultant moments and shears

Total moment (hogging and sagging);

$$M_{sedge} = 16.2 \text{ kNm/m}$$

Maximum shear force;

$$V_{sedge} = 32.2 \text{ kN/m}$$

Reinforcement required in top

K factor;

$$K_{edgetop} = M_{sedge}/(f_{cu} \times d_{tslabmin}^2) = 0.018$$

Lever arm;

$$Z_{edgetop} = d_{tslabmin} \times \min(0.95, 0.5 + \sqrt{(0.25 - K_{edgetop}/0.9)}) =$$

152 mm

Area of steel required for bending;

$$A_{sedgeopbend} = M_{sedge}/((1.0/g_s) \times f_{yslab} \times Z_{edgetop}) = 245$$

mm²/m



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Area of steel required;

$$A_{sedge\ topreq} = \max(A_{sedge\ topbend}, A_{slabmin}) = 260 \text{ mm}^2/\text{m}$$

PASS - $A_{sedge\ topreq} \leq A_{sedge\ top}$ - Area of reinforcement provided in top of slab is adequate

Applied shear stress;

$$V_{edge} = V_{sedge} \times 1.0\text{m} / (1000\text{mm} \times d_{slabmin}) = 0.201 \text{ N/mm}^2$$

Tension steel ratio;

$$r_{edge} = 100 \times A_{sedge\ top} \times 1.0\text{m} / (1000\text{mm} \times d_{slabmin}) = 0.246$$

From BS8110-1:1997 - Table 3.8

Design concrete shear strength;

$$V_{cedge} = 0.557 \text{ N/mm}^2$$

PASS - $V_{edge} \leq V_{cedge}$ - Shear capacity of the slab is not exceeded

Slab edge deflection check

Basic allowable span to depth ratio;

$$\text{Ratio}_{\text{basicedge}} = 7.0$$

Moment factor;

$$M_{\text{factorededge}} = M_{\text{sedge}} / d_{\text{slabmin}}^2 = 0.632 \text{ N/mm}^2$$

Steel service stress;

$$f_{\text{sedge}} = 2/3 \times f_{\text{yslab}} \times A_{\text{sedge\ topbend}} / A_{\text{sedge\ top}} = 207.644$$

N/mm²

Modification factor;

$$M_{\text{Fedge}} = \min(2.0, 0.55 + [(477 \text{ N/mm}^2 -$$

$$f_{\text{sedge}}) / (120 \times (0.9 \text{ N/mm}^2 + M_{\text{factorededge}})))$$

$$M_{\text{Fedge}} = 2.000$$

Modified allowable span to depth ratio;

$$\text{Ratio}_{\text{allowededge}} = \text{Ratio}_{\text{basicedge}} \times M_{\text{Fedge}} = 14.000$$

Actual span to depth ratio;

$$\text{Ratio}_{\text{actualedge}} = l_{\text{edge}} / d_{\text{slabmin}} = 6.281$$

PASS - $\text{Ratio}_{\text{actualedge}} \leq \text{Ratio}_{\text{allowededge}}$ - Slab span to depth ratio is adequate

Corner design checks

Basic loading

Corner load number 1

Load type;

Line load in x direction

Dead load;

$$W_{\text{Dcorner1}} = 5.0 \text{ kN/m}$$

Live load;

$$W_{\text{Lcorner1}} = 2.0 \text{ kN/m}$$

Ultimate load;

$$W_{\text{ultcorner1}} = 1.4 \times W_{\text{Dcorner1}} + 1.6 \times W_{\text{Lcorner1}} = 10.2 \text{ kN/m}$$

Centroid of load from outside face of raft;

$$y_{\text{corner1}} = 100 \text{ mm}$$

Corner load number 2

Load type;

Line load in y direction

Dead load;

$$W_{\text{Dcorner2}} = 5.0 \text{ kN/m}$$

Live load;

$$W_{\text{Lcorner2}} = 2.0 \text{ kN/m}$$

Ultimate load;

$$W_{\text{ultcorner2}} = 1.4 \times W_{\text{Dcorner2}} + 1.6 \times W_{\text{Lcorner2}} = 10.2 \text{ kN/m}$$

Centroid of load from outside face of raft;

$$x_{\text{corner2}} = 100 \text{ mm}$$

Corner bearing pressure check

Total uniform load at formation level;

$$W_{\text{udlcorner}} = W_{\text{Dudl}} + W_{\text{Ludl}} + W_{\text{slab}} + W_{\text{coreslab}} = 12.8 \text{ kN/m}^2$$

Net bearing press avail to resist line/point loads; $Q_{\text{netcorner}} = Q_{\text{allow}} - W_{\text{udlcorner}} = 87.2 \text{ kN/m}^2$

Total line/point loads

Total unfactored line load in x direction;

$$W_{\text{slinex}} = 7.0 \text{ kN/m}$$

Total ultimate line load in x direction;

$$W_{\text{ultlinex}} = 10.2 \text{ kN/m}$$

Total unfactored line load in y direction;

$$W_{\text{sliney}} = 7.0 \text{ kN/m}$$

Total ultimate line load in y direction;

$$W_{\text{ultliney}} = 10.2 \text{ kN/m}$$



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Total unfactored point load;

$$W_{\text{spoint}} = 0.0 \text{ kN}$$

Total ultimate point load;

$$W_{\text{sultpoint}} = 0.0 \text{ kN}$$

Length of side of sq reqd to resist line/point loads;

 p_{corner}

$$= [W_{\text{slinex}} + W_{\text{sliney}} + \sqrt{(W_{\text{slinex}} + W_{\text{sliney}})^2 + 4 \times Q_{\text{netcorner}} \times W_{\text{spoint}}}] / (2 \times Q_{\text{netcorner}})$$

$$p_{\text{corner}} = 161 \text{ mm}$$

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

Moment due to load 1 (x line);
kNm/m

$$M_{x1} = \max(0 \text{ kNm/m}, W_{\text{ultcorner1}} \times (p_{\text{corner}}/2 - y_{\text{corner1}})) = 0.0$$

Total moment about x axis;

$$M_{sx} = 0.0 \text{ kNm/m}$$

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

Moment due to load 2 (y line);
kNm/m

$$M_{y2} = \max(0 \text{ kNm/m}, W_{\text{ultcorner2}} \times (p_{\text{corner}}/2 - x_{\text{corner2}})) = 0.0$$

Total moment about y axis;

$$M_{sy} = 0.0 \text{ kNm/m}$$

Check top reinforcement in slab for load/reaction eccentric moment

Max moment due to load/reaction eccentricity;
kNm/m

$$M_s = \max(M_{sx}, M_{sy}) = 0.0$$

From slab edge design checks away from corner

Moment due to slab edge spanning depression;

$$M_{\text{sedge}} = 16.2 \text{ kNm/m}$$

Total moment to be resisted;

$$M_{\text{scornerbp}} = M_s + M_{\text{sedge}} = 16.2 \text{ kNm/m}$$

K factor;

$$K_{\text{cornerbp}} = M_{\text{scornerbp}} / (f_{cu} \times d_{\text{slabmin}}^2) = 0.018$$

Lever arm;

$$Z_{\text{cornerbp}} = d_{\text{slabmin}} \times \min(0.95, 0.5 + \sqrt{(0.25 - K_{\text{cornerbp}}/0.9)}) =$$

152 mm

Area of steel required;
mm²/m

$$A_{\text{scornerbp}} = M_{\text{scornerbp}} / ((1.0/g_s) \times f_{yslab} \times Z_{\text{cornerbp}}) = 245$$

PASS - $A_{\text{scornerbp}} \leq A_{\text{sedge}} -$ Area of reinforcement provided to resist eccentric moment is adequate

The allowable bearing pressure at the corner will not be exceeded

Slab corner bending check

Cantilever span of slab at corner;

$$l_{\text{corner}} = f_{\text{depslab}}/\sqrt{2} + d_{\text{slabov}}/2 = 1391 \text{ mm}$$

Moment and shear due to self weight

Considering triangular loading

Maximum ultimate self weight udl;

$$W_{\text{swult}} = 1.4 \times W_{\text{slab}} \times f_{\text{depslab}}/\sqrt{2} = 8.8 \text{ kN/m}$$

Self weight bending moment;

$$M_{\text{cornersw}} = W_{\text{swult}} \times l_{\text{corner}}^2 / (6 \times f_{\text{depslab}}/\sqrt{2}) = 2.2 \text{ kNm/m}$$

Self weight shear force;

$$V_{\text{cornersw}} = W_{\text{swult}} \times l_{\text{corner}} / (2 \times f_{\text{depslab}}/\sqrt{2}) = 4.7 \text{ kN/m}$$

Moment and shear due to udl

Maximum ultimate udl;
kN/m

$$W_{\text{cornerudl}} = ((1.4 \times W_{\text{Dudl}}) + (1.6 \times W_{\text{Ludl}})) \times f_{\text{depslab}}/\sqrt{2} = 9.8$$

Bending moment;

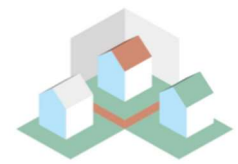
$$M_{\text{cornerudl}} = W_{\text{cornerudl}} \times l_{\text{corner}}^2 / (6 \times f_{\text{depslab}}/\sqrt{2}) = 2.4 \text{ kNm/m}$$

Shear force;

$$V_{\text{cornerudl}} = W_{\text{cornerudl}} \times l_{\text{corner}} / (2 \times f_{\text{depslab}}/\sqrt{2}) = 5.2 \text{ kN/m}$$

Moment and shear due to line loads in x direction

Initially assume this only produces a moment about y axis



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Bending moment;
kNm/m

$$M_{\text{cornerline}} = W_{\text{ultline}} \times l_{\text{corner}}^2 / (2 \times f_{\text{deplab}} / (2 \times \sqrt{(2)})) = 15.1$$

Shear force;

$$V_{\text{cornerline}} = W_{\text{ultline}} \times l_{\text{corner}} / (f_{\text{deplab}} / (2 \times \sqrt{(2)})) = 21.7 \text{ kN/m}$$

Moment and shear due to line loads in y direction

Initially assume this only produces a moment about x axis

Bending moment;
kNm/m

$$M_{\text{cornerliney}} = W_{\text{ultliney}} \times l_{\text{corner}}^2 / (2 \times f_{\text{deplab}} / (2 \times \sqrt{(2)})) = 15.1$$

Shear force;

$$V_{\text{cornerliney}} = W_{\text{ultliney}} \times l_{\text{corner}} / (f_{\text{deplab}} / (2 \times \sqrt{(2)})) = 21.7 \text{ kN/m}$$

Moments and shears due to point loads

Total moments and shears due to point loads

Bending moment about x axis;

$$M_{\text{cornerpointx}} = 0.0 \text{ kNm/m}$$

Bending moment about y axis;

$$M_{\text{cornerpointy}} = 0.0 \text{ kNm/m}$$

Shear force;

$$V_{\text{cornerpoint}} = 0.0 \text{ kN/m}$$

Resultant moments and shears

Total moment about x axis;

$$M_{\text{Scornerx}} = M_{\text{Scornerw}} + M_{\text{Scornerudl}} + M_{\text{Scornerliney}} + M_{\text{Scornerpointx}} =$$

19.7 kNm/m

Total shear force about x axis;

$$V_{\text{Scornerx}} = V_{\text{Scornerw}} + V_{\text{Scornerudl}} + V_{\text{Scornerliney}} + V_{\text{Scornerpoint}} = 31.6$$

kN/m

Total moment about y axis;

$$M_{\text{Scornery}} = M_{\text{Scornerw}} + M_{\text{Scornerudl}} + M_{\text{Scornerlinex}} + M_{\text{Scornerpointy}} =$$

19.7 kNm/m

Total shear force about y axis;

$$V_{\text{Scornery}} = V_{\text{Scornerw}} + V_{\text{Scornerudl}} + V_{\text{Scornerlinex}} + V_{\text{Scornerpoint}} = 31.6$$

kN/m

Deflection of both edge beams at corner will be the same therefore design for average of these moments and shears

Design bending moment;

$$M_{\text{Scorner}} = (M_{\text{Scornerx}} + M_{\text{Scornery}}) / 2 = 19.7 \text{ kNm/m}$$

Design shear force;

$$V_{\text{Scorner}} = (V_{\text{Scornerx}} + V_{\text{Scornery}}) / 2 = 31.6 \text{ kN/m}$$

Reinforcement required in top of slab at corners

K factor;

$$K_{\text{corner}} = M_{\text{Scorner}} / (f_{\text{cu}} \times d_{\text{slabmin}}^2) = 0.022$$

Lever arm;
mm

$$Z_{\text{corner}} = d_{\text{slabmin}} \times \min(0.95, 0.5 + \sqrt{(0.25 - K_{\text{corner}}/0.9)}) = 152$$

Area of steel required for bending;

$$A_{\text{Scornerbend}} = M_{\text{Scorner}} / ((1.0/g_s) \times f_{\text{yslab}} \times Z_{\text{corner}}) = 298 \text{ mm}^2/\text{m}$$

Area of steel required;

$$A_{\text{Scorner}} = \max(A_{\text{Scornerbend}}, A_{\text{slabmin}}) = 298 \text{ mm}^2/\text{m}$$

PASS - $A_{\text{Scorner}} \leq A_{\text{sedge top}}$ - Area of reinforcement provided in top of slab at corners is adequate

Applied shear stress;

$$V_{\text{corner}} = V_{\text{Scorner}} / d_{\text{slabmin}} = 0.197 \text{ N/mm}^2$$

Tension steel ratio;

$$r_{\text{corner}} = 100 \times A_{\text{sedge top}} / d_{\text{slabmin}} = 0.246$$

From BS8110-1:1997 - Table 3.8

Design concrete shear strength;

$$V_{\text{ccorner}} = 0.557 \text{ N/mm}^2$$

Pass - $V_{\text{corner}} \leq V_{\text{ccorner}}$ - Shear capacity of the slab is not exceeded

Slab corner deflection check

Basic allowable span to depth ratio;

$$\text{Ratio}_{\text{basiccorner}} = 7.0$$



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Moment factor;

$$M_{factorcorner} = M_{Scorner} / d_{slabmin}^2 = \mathbf{0.768 \text{ N/mm}^2}$$

Steel service stress;

$$f_{scorner} = 2/3 \times f_{yslab} \times A_{scornerbend} / A_{sedgetop} = \mathbf{252.354 \text{ N/mm}^2}$$

Modification factor;

$$MF_{corner} = \min(2.0, 0.55 + [(477 \text{ N/mm}^2 -$$

$$f_{scorner}) / (120 \times (0.9 \text{ N/mm}^2 + M_{factorcorner}))]$$

$$MF_{corner} = \mathbf{1.672}$$

Modified allowable span to depth ratio;

$$Ratio_{allowcorner} = Ratio_{basiccorner} \times MF_{corner} = \mathbf{11.706}$$

Actual span to depth ratio;

$$Ratio_{actualcorner} = l_{corner} / d_{slabmin} = \mathbf{8.692}$$

PASS - $Ratio_{actualcorner} \leq Ratio_{allowcorner}$ - Slab span to depth ratio is adequate

We therefore conclude that the raft is acceptable for the proposed conversion.