Hudds Design
Structural & Architectural Engineering
DESIGN CONSULTANTS

PLANNING AND BUILDING CONTROL; CALCULATIONS & DRAWINGS

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STRUCTURAL CALCULATIONS

FOR

MR C GADDES
@

PLOT 1,2&5 WINDMILL HILL QUARRY, BRIERLEY ROAD, BARNSLEY, S72 7AJ.

Design Summary: (See Sheet A1)

Notes:
- All dimensions and Beam Lengths to be confirmed and checked by builder on site prior to works commencing and materials being ordered
- All Beam Heights and Levels to be agreed with client on site.
- All steel to be minimum grade S275 (UNO)
- All Timber / Timber Packing to be minimum grade C16 (UNO)
- Minimum end bearing for steel to be 100mm (UNO)
- Minimum end bearing for timber to be 100mm
- Minimum end bearing for lintels to be 150mm
- All welded connections for steel to be minimum 6mm fillet welds all round connecting member (UNO)
- All Beams to have minimum fire protection of 30 minutes or as required by building regulations
- All new masonry blockwork to be minimum 7.3 N/mm^2 strength
- All returns and loadbearing walls as shown on Architects drawings and mark ups
- All concrete to be minimum grade C30
- All ground conditions to approval of local authority / building inspector
- All temporary support works to builder/contractor's specification
- All party wall notices and approvals are client/contractor's responsibility.
- Hudds Design has not been appointed as CDM coordinator and is the responsibility of the builder.

ISSUED FOR BUILDING CONTROL APPROVAL:	**9th October 2021**
HUDDS DESIGN	**Ref: HD-S21-1009-Rev.1**

SHEET A1

Refer to sketch mark ups for references
Refer to Architects drawings for dimensions

DESIGN SUMMARY

'Floor FFJ'	**First Floor Joists** 75x200mm Deep Timber Joists @400mm Ctrs Min Grade C24
'Trimmer T1'	Double Up Floor Joists
'Lintel L1'	IG L1/HD 100 Cavity Wall Lintel
'Lintel L2'	Naylor R6 100x140mm Deep Concrete Lintel per 100mm Wall
'Lintel L3'	**Boot Lintel 203x203x71 UC with 10mm plate fully welded to bottom** **Min 6mm fillet welds** 400x100x215mm Deep Concrete Padstones Min 200mm End Bearing
'Lintel L4'	**Boot Lintel 152x152x37 UC with 10mm plate fully welded to bottom** **Min 6mm fillet welds** 200x100x215mm Deep Concrete Padstones
'Windpost WP'	100x100x8 SHS with fixings top and bottom Fixed top and Bottom using 10mm cleats fully welded with 2 No. M12 grade 8.8 bolts. Fixings to wall plate, floor joists, steel beams as required. Ties at 225mm Ctrs
'Raft Foundation'	**Refer to Sketch A9** 200mm Thick Reinforced Concrete Raft Slab **2 Layers** A393 Mesh Top **and** Bottom (2 Top & 2 Bottom) Provide Edge & Internal Beam Thickenings 450x450mm with 3 No. T20 Bars (Top & Bottom) With 10mm Shear Links @ 200mm Ctrs Min Grade C30 Concrete Min 40mm Cover

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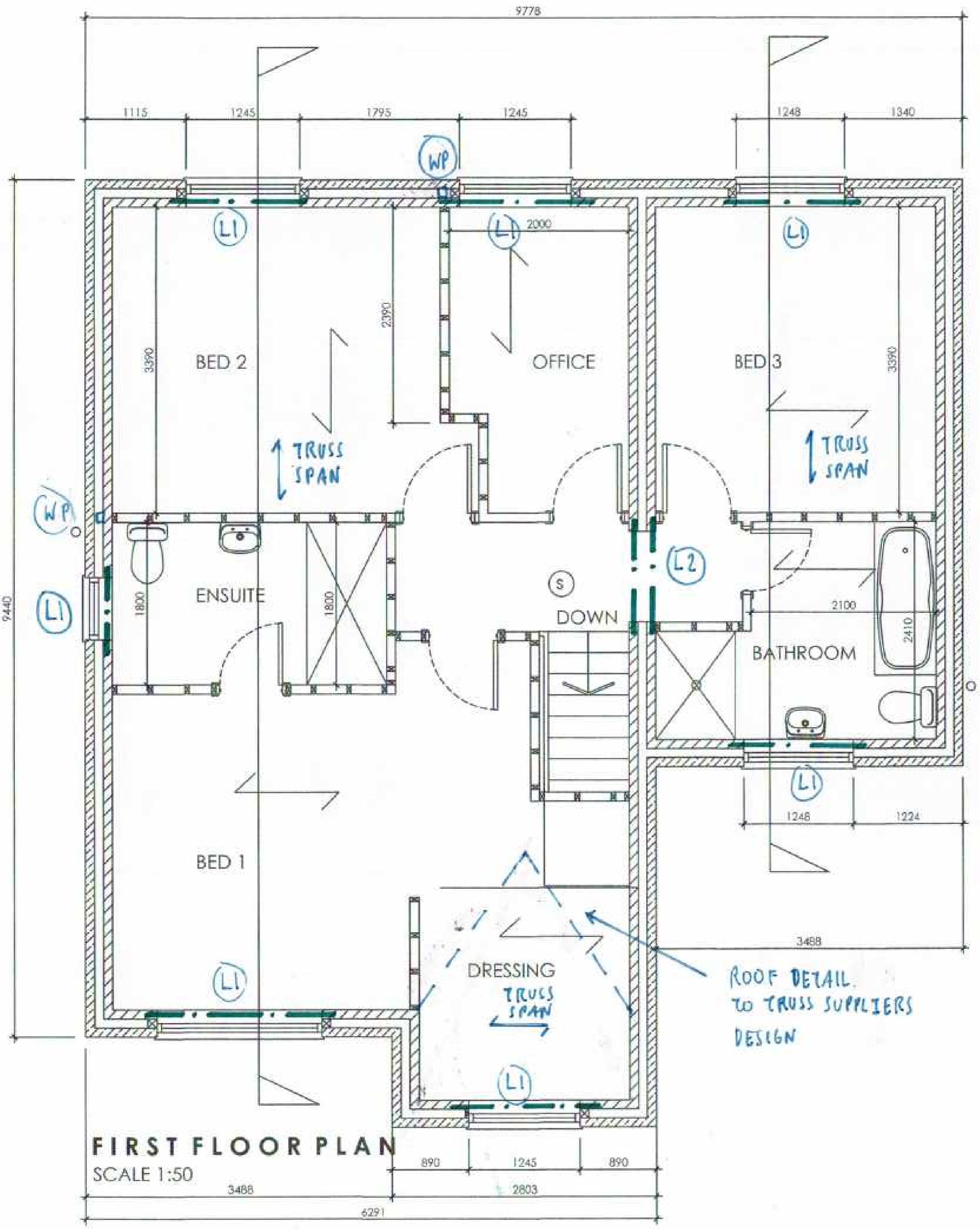
HD-S21-1009 (A2)
PROPOSED SITE PLAN (NTS)

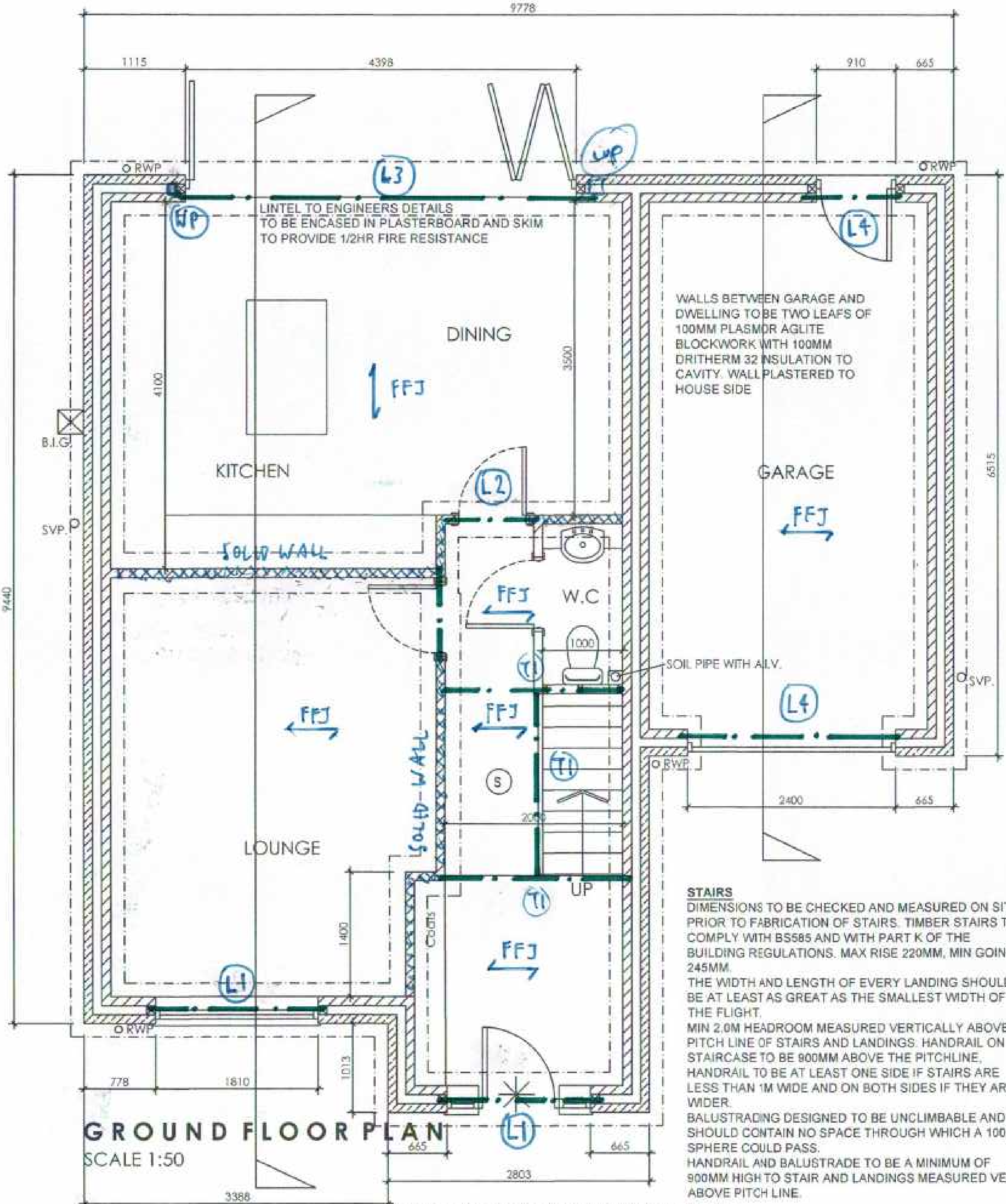
NOTE:
- ADOPT TYPICAL DESIGN TO PLOTS 1, 2, & 5

HD-521-1004 (A3)
PROPOSED FIRST FLOOR PLAN
(NTS)



FIRST FLOOR PLAN
SCALE 1:50

HD-521-1009 (A4)
PROPOSED GROUND FLOOR PLAN
(NTS)

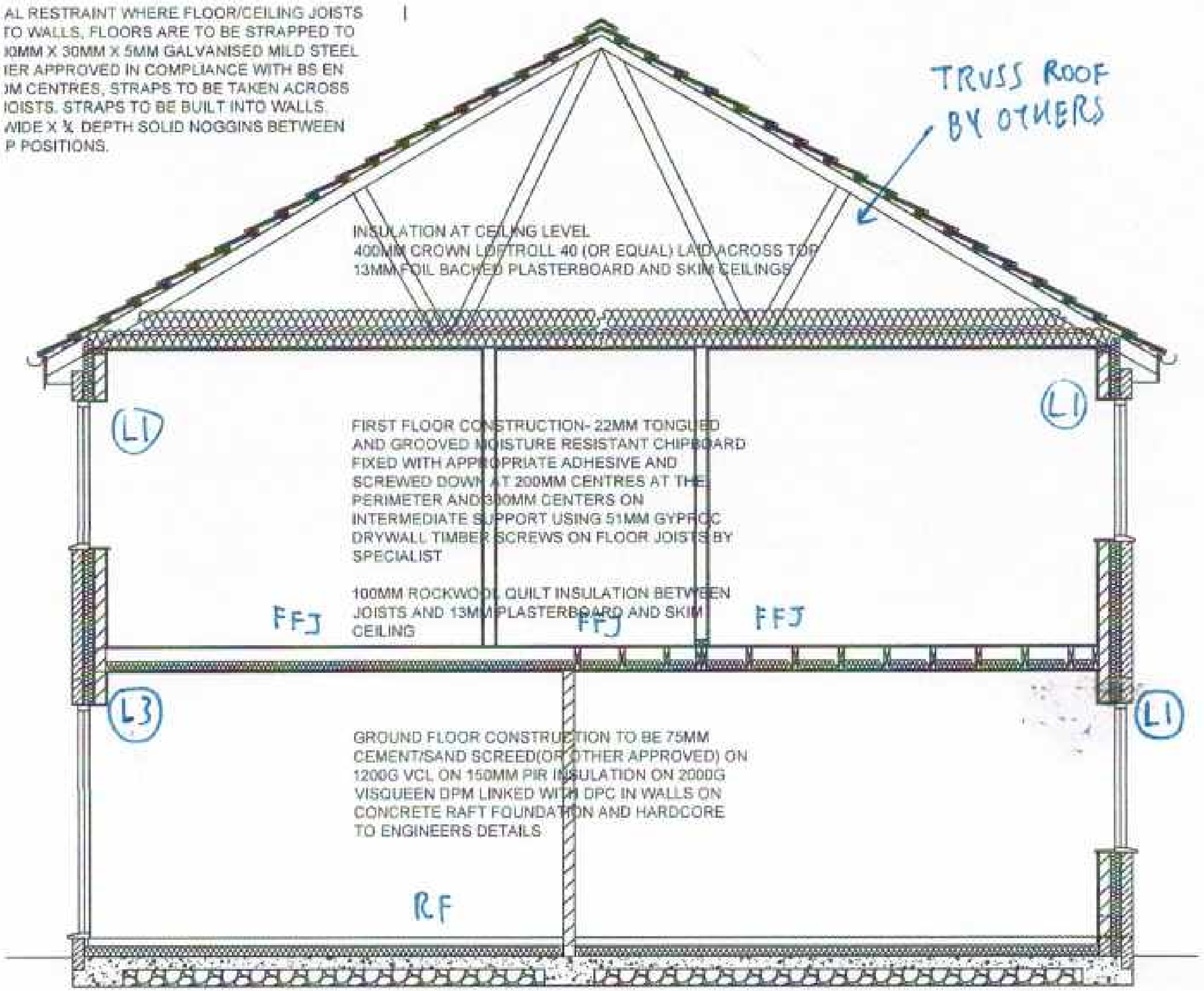


GROUND FLOOR PLAN
SCALE 1:50

STAIRS
DIMENSIONS TO BE CHECKED AND MEASURED ON SITE PRIOR TO FABRICATION OF STAIRS. TIMBER STAIRS TO COMPLY WITH BS585 AND WITH PART K OF THE BUILDING REGULATIONS. MAX RISE 220MM, MIN GOING 245MM.
THE WIDTH AND LENGTH OF EVERY LANDING SHOULD BE AT LEAST AS GREAT AS THE SMALLEST WIDTH OF THE FLIGHT.
MIN 2.0M HEADROOM MEASURED VERTICALLY ABOVE PITCH LINE OF STAIRS AND LANDINGS. HANDRAIL ON STAIRCASE TO BE 900MM ABOVE THE PITCHLINE, HANDRAIL TO BE AT LEAST ONE SIDE IF STAIRS ARE LESS THAN 1M WIDE AND ON BOTH SIDES IF THEY ARE WIDER.
BALUSTRADING DESIGNED TO BE UNCLIMBABLE AND SHOULD CONTAIN NO SPACE THROUGH WHICH A 100MM SPHERE COULD PASS.
HANDRAIL AND BALUSTRADE TO BE A MINIMUM OF 900MM HIGH TO STAIR AND LANDINGS MEASURED VERT. ABOVE PITCH LINE.

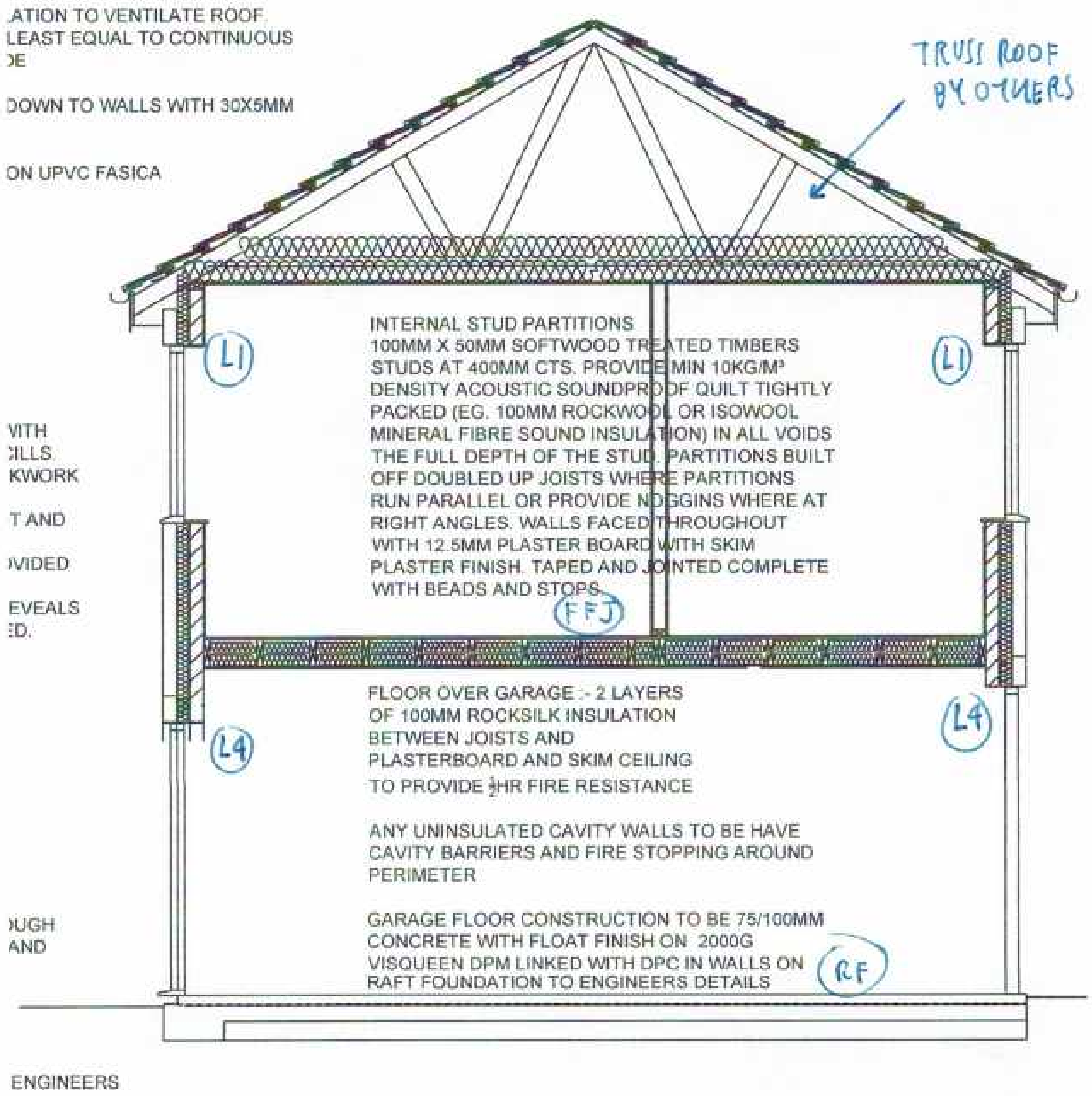
PROVIDE A LEVEL APPROACH TO THE PRINCIPAL ENTRANCE DOOR NO STEEPER THAN 1:20 AND AT LEAST 900MM WIDE, WITH CROSS FALLS NO GREATER THAN 1:40. APPROACH SURFACE MATERIAL TO BE FIRM AND NON-SLIP, CAPABLE OF SUPPORTING THE WEIGHT OF A WHEEL CHAIR AND ITS USER (LOOSE MATERIAL SUCH AS GRAVEL AND SHINGLE WOULD NOT BE SUITABLE).

HD-521-1009 (A5)
PROPOSED SECTION (NTS)



SECTION A-A

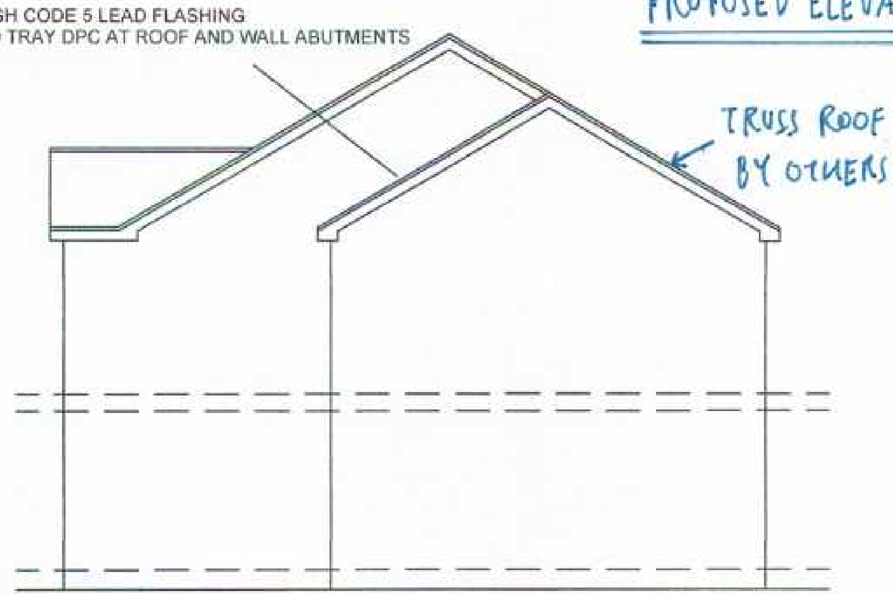
HD - 521 - 1004 (A6)
PROPOSED SECTION (NTS)



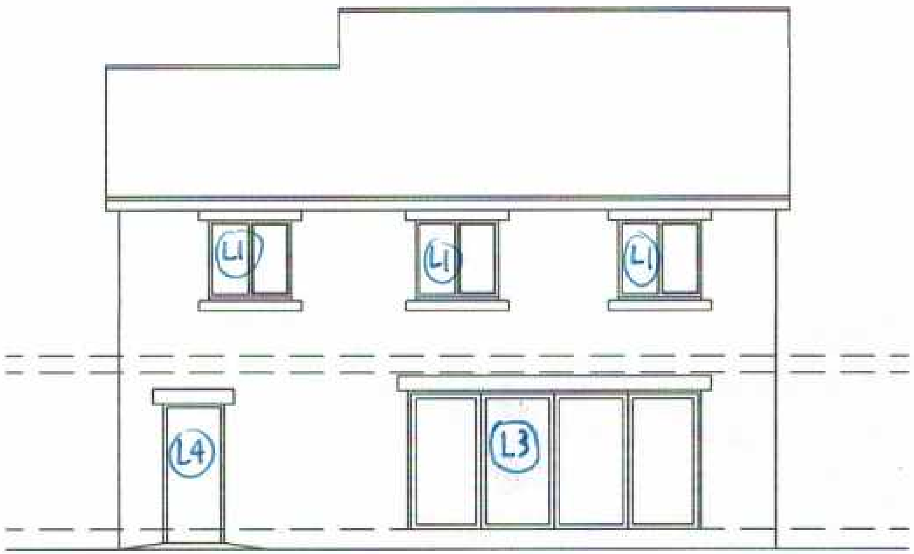
SECTION B-B

HD-521-1009 (A7)
PROPOSED ELEVATIONS (NTS)

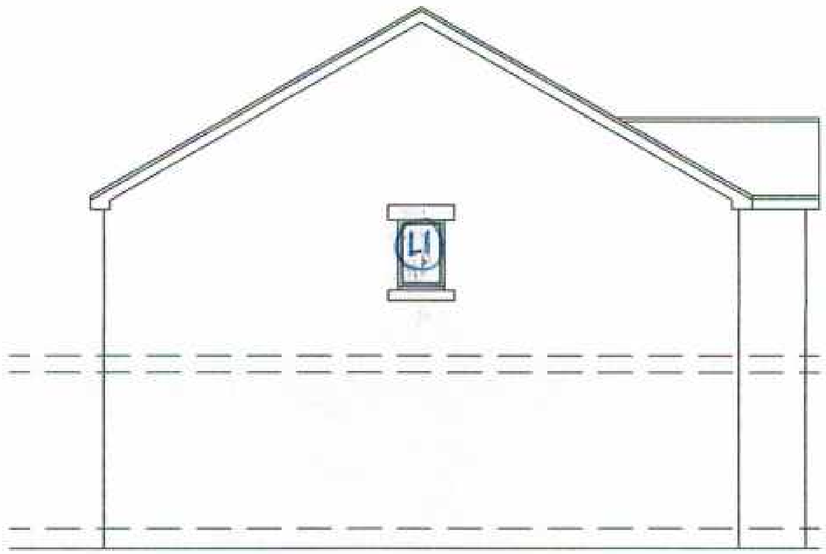
FRONT ELEVATION



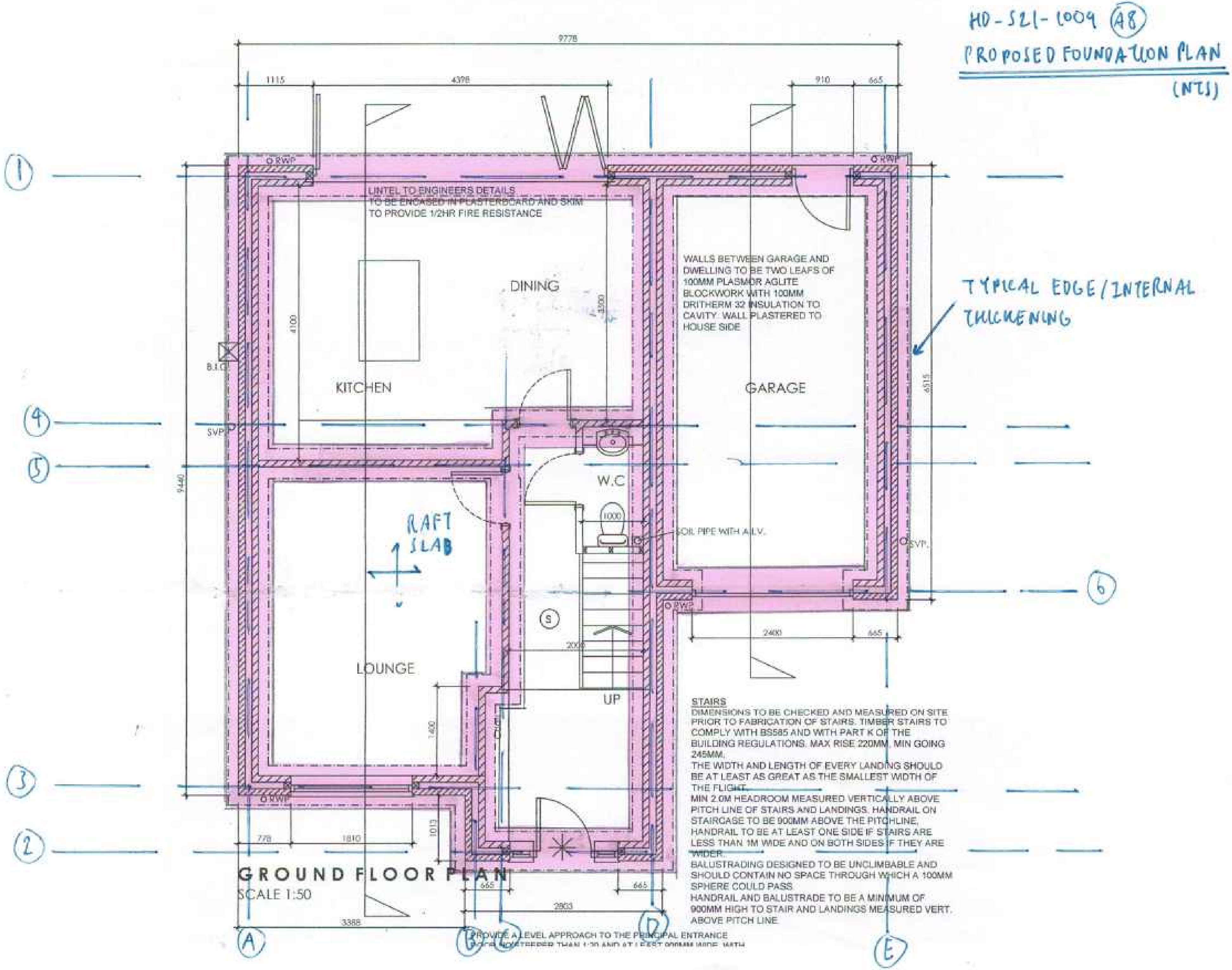
SIDE ELEVATIO N



REAR ELEVATION



SIDE ELEVATION


HD-S21-1009 (A8)
PROPOSED FOUNDATION PLAN
(NTS)
9778
1115
4378
910
665
RWP
LINTEL TO ENGINEERS DETAILS
TO BE ENCASED IN PLASTERBOARD AND SKIM
TO PROVIDE 1/2HR FIRE RESISTANCE
DINING
KITCHEN
4100
3900
B.I.C
SVP
WALLS BETWEEN GARAGE AND
DWELLING TO BE TWO LEAFS OF
100MM PLASMOR AGLITE
BLOCKWORK WITH 100MM
DRITHERM 32 INSULATION TO
CAVITY. WALL PLASTERED TO
HOUSE SIDE
GARAGE
6515
TYPICAL EDGE / INTERNAL
THICKENING
W.C
1000
SOIL PIPE WITH A.I.V.
SVP.
RAFT
SLAB
9440
LOUNGE
2000
UP
1400
1013
778
1810
2400
665
2803
3388
GROUND FLOOR PLAN
SCALE 1:50
STAIRS
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ABOVE PITCH LINE.
PROVIDE A LEVEL APPROACH TO THE PRINCIPAL ENTRANCE
1
2
3
4
5
6
A
B
C
D
E

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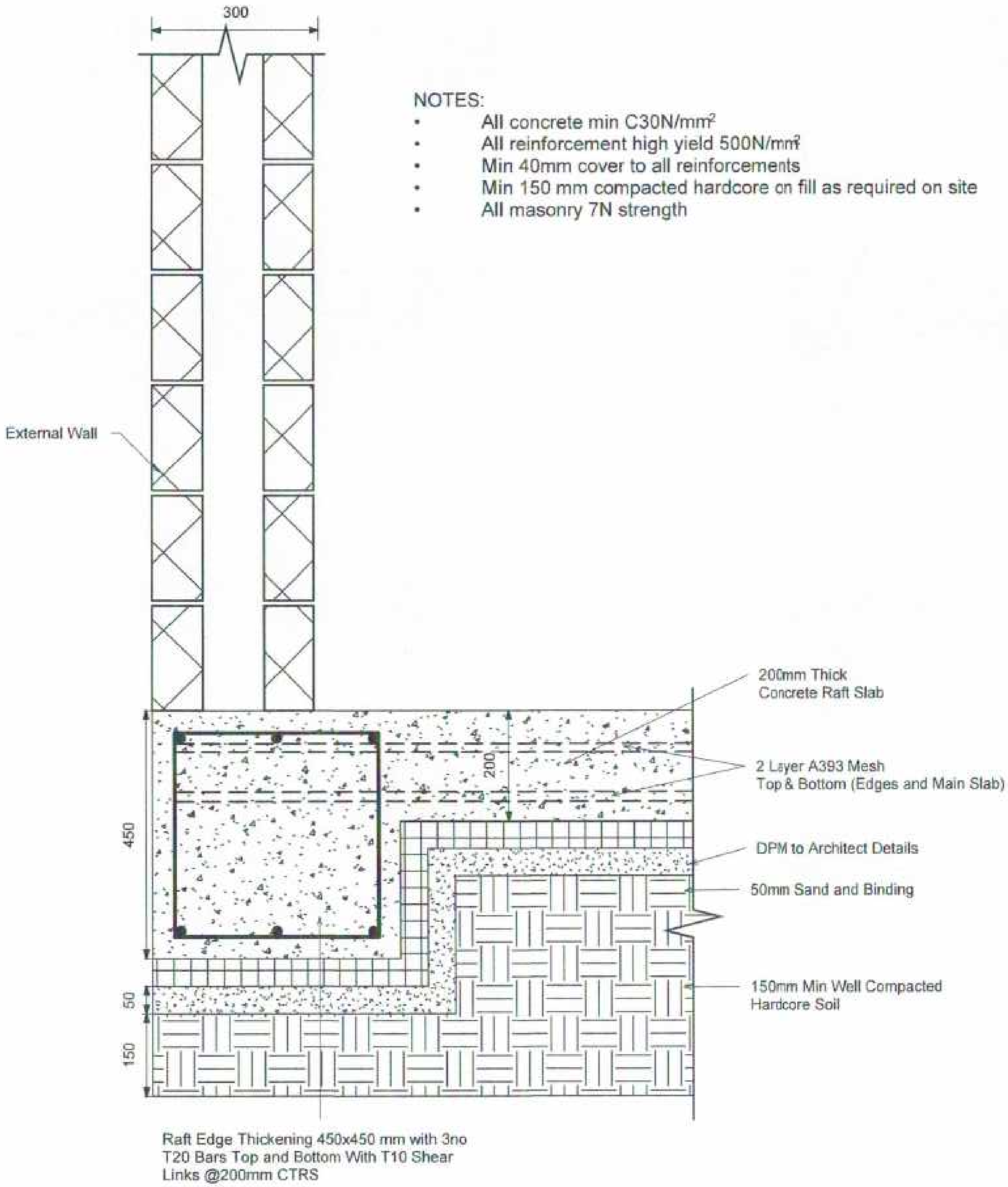
Project: Plot 1,2&5 Windmill Hill Quarry, Brierley Road, Barnsley, S72 7AJ				Job No:	HD-S21-1009
Calcs For: Raft Foundation RF				Start Page: A9	Rev:
Calcs By: JT	Calcs Date: 15/10/21	Checked: JA	Checked Date:	Approved:	Scale: 1 : 10@A4

NOTES:

- All concrete min C30N/mm^2
- All reinforcement high yield 500N/mm^2
- Min 40mm cover to all reinforcements
- Min 150 mm compacted hardcore on fill as required on site
- All masonry 7N strength



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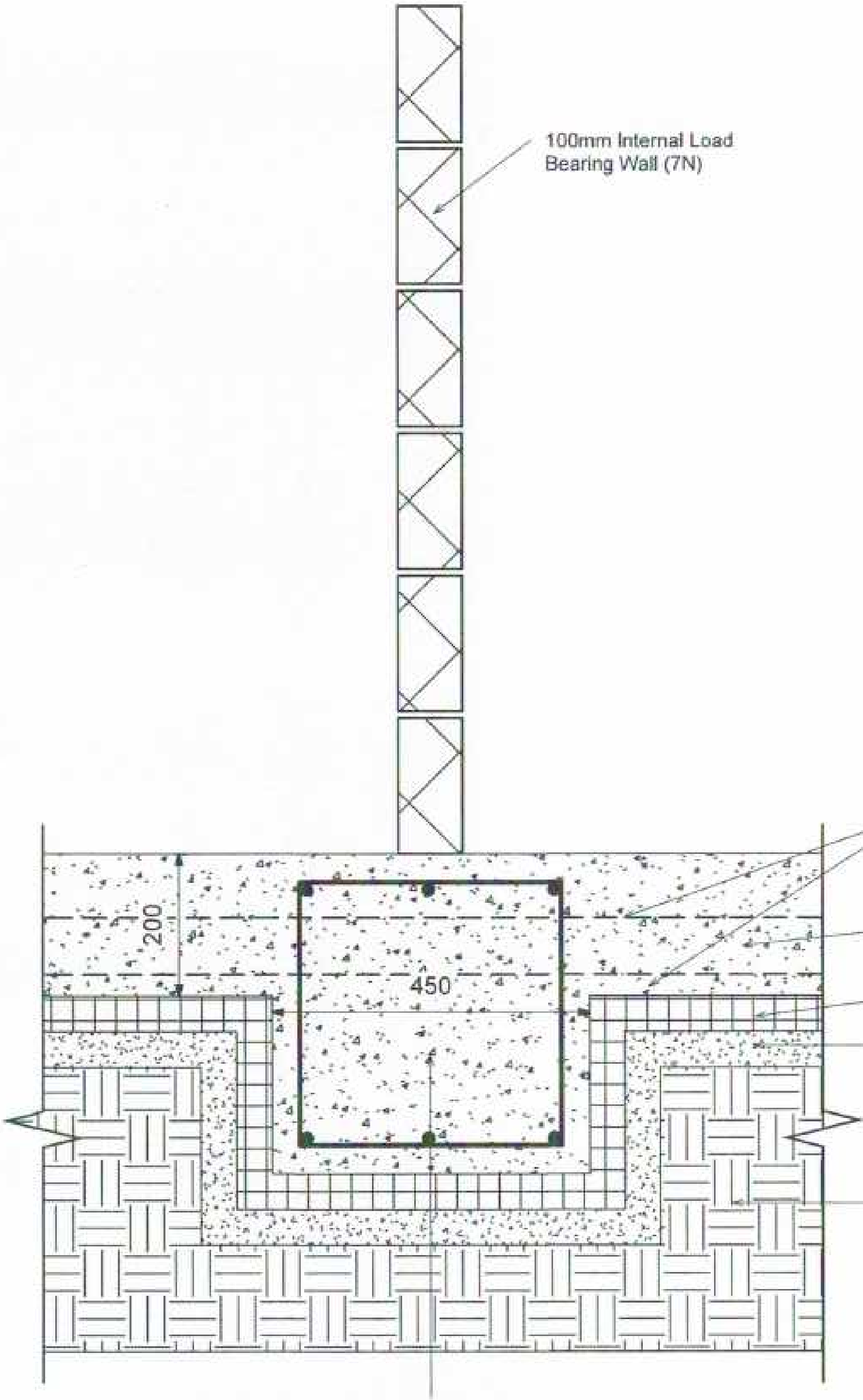
Project:	Plot 1,2&5 Windmill Hill Quarry, Brierley Road, Barnsley, S72 7AJ				Job No:	HD-S21-1009
Calcs For:	Raft Foundation				Start Page: A10	Rev:
Calcs By: JT	Calcs Date:	15/10/21	Checked: JA	Checked Date:	Approved:	Scale: 1 : 10@A4



NOTES:

- All concrete min C30N/mm²
- All reinforcement high yield 500N/mm²
- Min 40mm cover to all reinforcements
- Min 150 mm compacted hardcore on fill as required on site
- All masonry 7N strength

STRUCTURAL CALCULATIONS

Address: **Plot 1,2&5 Windmill Hill Quarry, Brierley Road, Barnsley, S72 7AJ.**

Calculations for:

New Build.

DESIGN LOADINGS

Walls

Inner Skin → 0.1 x 20 kN/m^3 = 2.0 kN/m^2 – (**Block**)

Outer Skin → 0.1 x 24 kN/m^3 = 2.4 kN/m^2 – (**Stone**)

Total **= 4.4 kN/m^2**

Roof – (Tiles)

Dead Loads = 1.3 kN/m^2

Imposed Loads = 0.6 kN/m^2

Total **= 1.9 kN/m^2**

Floor – (Timber)

Dead Loads = 1.0 kN/m^2 – (Allow for Stud Partition Loads)

Imposed Loads = 1.5 kN/m^2

Total **= 2.5 kN/m^2**

Floor – (Raft Slab)

Dead Loads = 1.2 kN/m^2 (Additional Self-Weight Included in Calculation)

Imposed Loads = 2.5 kN/m^2 (Garage)

Total **= 3.7 kN/m^2**

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TIMBER JOIST DESIGN (BS5268-2:2002)

Tedds calculation version 1.1.04

Joist details

Joist breadth $b = \mathbf{75}$ mm — Joist depth $h = \mathbf{200}$ mm

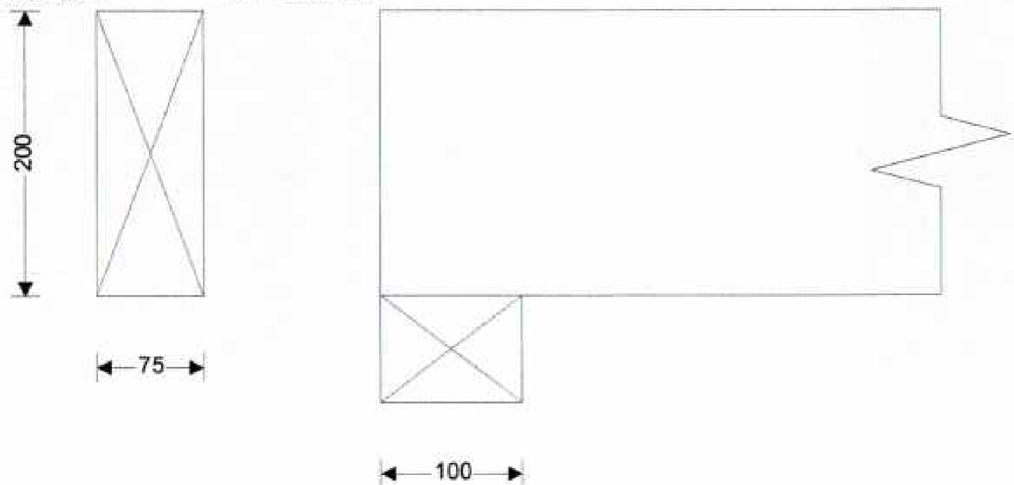
Joist spacing $s = \mathbf{400}$ mm — Service class of timber **1**

Timber strength class **C24**

Span details

Number of spans $N_{span} = \mathbf{1}$ — Length of bearing $L_b = \mathbf{100}$ mm

Clear length of span $L_{s1} = \mathbf{4100}$ mm



Section properties

Second moment of area $I = \mathbf{50000000}$ mm^4 — Section modulus $Z = \mathbf{500000}$ mm^3

Loading details

Joist self weight $F_{swt} = \mathbf{0.05}$ kN/m — Dead load $F_{d\_udl} = \mathbf{1.00}$ kN/m^2

Imposed UDL(Long term) $F_{i\_udl} = \mathbf{1.50}$ kN/m^2

Imposed point load (Medium) $F_{i\_pt} = \mathbf{1.40}$ kN

Consider long term loads

Design bending moment $M = \mathbf{2.209}$ kNm — Design shear force $V = \mathbf{2.156}$ kN

Design support reaction $R = \mathbf{2.156}$ kN — Design deflection $\delta = \mathbf{7.426}$ mm

Check bending stress

Permissible bending stress $\sigma_{m\_adm} = \mathbf{8.626}$ N/mm^2 — Applied bending stress $\sigma_{m\_max} = \mathbf{4.419}$ N/mm^2

PASS - Applied bending stress within permissible limits

Check shear stress

Permissible shear stress τ_{adm} = **0.781** N/mm^2 | Applied shear stress τ_{max} = **0.216** N/mm^2

PASS - Applied shear stress within permissible limits

Check bearing stress

Permissible bearing stress σ_{c_adm} = **2.640** N/mm^2 | Applied bearing stress σ_{c_max} = **0.287** N/mm^2

PASS - Applied bearing stress within permissible limits

Check deflection

Permissible deflection δ_{adm} = **12.300** mm | Actual deflection δ = **7.426** mm

PASS - Actual deflection within permissible limits

Consider medium term loads

Design bending moment M = **2.384** kNm | Design shear force V = **2.326** kN

Design support reaction R = **2.326** kN | Design deflection δ = **7.081** mm

Check bending stress

Permissible bending stress σ_{m_adm} = **10.783** N/mm^2 | Applied bending stress σ_{m_max} = **4.767** N/mm^2

PASS - Applied bending stress within permissible limits

Check shear stress

Permissible shear stress τ_{adm} = **0.976** N/mm^2 | Applied shear stress τ_{max} = **0.233** N/mm^2

PASS - Applied shear stress within permissible limits

Check bearing stress

Permissible bearing stress σ_{c_adm} = **3.300** N/mm^2 | Applied bearing stress σ_{c_max} = **0.310** N/mm^2

PASS - Applied bearing stress within permissible limits

Check deflection

Permissible deflection δ_{adm} = **12.300** mm | Actual deflection δ = **7.081** mm

PASS - Actual deflection within permissible limits

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Project: PLOT 1,2 KS WINDMILL HILL QUARY, S72 7AJ				Job No. HD-S21-1004	
Calcs for: LINTEL L1				Start Page: L1-1	Rev:
Calcs by: JT	Calcs Date: 12/10/2021	Checked:	Checked Date:	Approved:	Approved Date:

LINTEL L1

MAX CLEAR SPAN = 1900mm

SUPPORTING WALL HEIGHT = 0.5m
+ ROOF LENGTH - 4.5m (TRUSS)

LOADS ON LINTEL L1

TOTAL WALL LOAD = 0.5m × 4.4kN/m² = 2.2kN/m

TOTAL ROOF LOAD = 4.5m × 1.9kN/m² = 8.0kN/m

TOTAL UDL (UNFACTORED) = 10.2kN/m

ADOPT IG L1/HD 100 CAVITY WALL LINTEL

SWL = 35.0kN > 10.2kN/m × 1.9m = 19.4kN

HD-S21-L009
L1-2

If lintels are required to carry loads not indicated on the load tables, please contact IG's Technical Department.

LINTEL HOTLINE
01633 486486

Fax Back Enquiry Forms are also available for download.
www.iglintels.com/technical

IG Fastrack CAD Database is accessible from iglintels.com

Cavity Wall

Cavity widths from **50mm to 165mm**

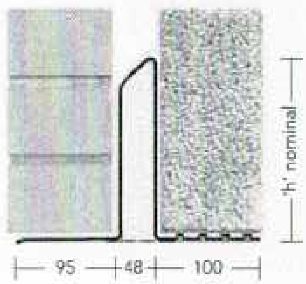
OUTER LEAF	INNER LEAF
102mm	100mm

Heavy Duty Load

L1/HD 50 — For cavity widths 50-65mm

Manufactured length 150mm increments	600-1350	1500	1650	1800-2100	2250-3000	3150-3600	3750-4000
Height 'h'	105	121	121	171	209	209	209
Thickness	3.2	3.2	3.2	3.2	3.2	3.2	3.2
Total UDL kN 3:1	21	27	27	32	37	34	30
Total UDL kN 19:1	18	22	22	24	33	31	27

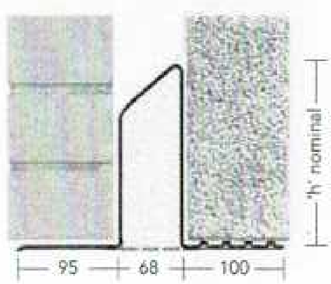
50-65mm cavity



L1/HD 75 — For cavity widths 70-85mm

Manufactured length 150mm increments	600-1200	1350-1500	1650-2100	2250-2550	2700-3000	3150-3600	3750-4200
Height 'h'	103	117	167	205	205	205	205
Thickness	2.9	2.9	2.9	2.9	3.2	3.2	3.2
Total UDL kN 3:1	30	30	40	40	40	35	33
Total UDL kN 19:1	22	22	35	35	35	32	28

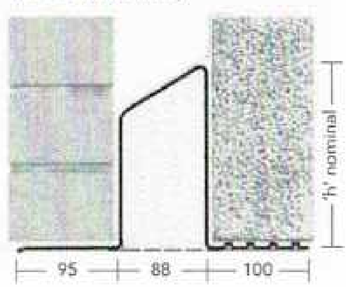
70-85mm cavity



L1/HD 100 — For cavity widths 90-105mm

Manufactured length 150mm increments	600-1200	1350-1500	1650-2100	2250-2550	2700-3000	3150-3600	3750-4200
Height 'h'	110	135	163	203	203	203	203
Thickness	2.9	2.9	2.9	2.9	3.2	3.2	3.2
Total UDL kN 3:1	30	30	40	40	40	35	33
Total UDL kN 19:1	22	22	35	35	35	32	28

90-105mm cavity



DAMP PROOFING
Provide a damp proof course over all lintels. Please see IG installation details for guidance on page 12.

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Project: PLOT L2 & 5 WINDMILL HILL QUAY, ST2 TAJ				Job No. HD-S21-1009	
Calcs for: LINTEL L3 (BOOT LINTEL)				Start Page: L3-1	Rev:
Calcs by: JT	Calcs Date: 12/10/2021	Checked:	Checked Date:	Approved:	Approved Date:

LINTEL L3 (BOOT LINTEL)

MAX CLEAR SPAN = 4400mm

SUPPORTING WALL HEIGHT = 3.0m
+ ROOF LENGTH = 4.5m
+ FLOOR LENGTH = 2.1m

LOADS ON LINTEL L3 (BOOT LINTEL)

ROOF DEAD LOAD =	4.5m	×	1.3kN/m²	= 5.9 kN/m	} = 8.0 kN/m
FLOOR DEAD LOAD =	2.1m	×	1.0kN/m²	= 2.1 kN/m	
ROOF IMPOSED LOAD =	4.5m	×	0.6kN/m²	= 2.7 kN/m	} = 5.9 kN/m
FLOOR IMPOSED LOAD =	2.1m	×	1.5kN/m²	= 3.2 kN/m	

ADOPT 203 × 203 × 71 UC WITH 10mm PLATE FULLY WELDED TO BOTTOM

MIN 6mm FILLET WELDS (SEE CALC L3)

PADSTONES

R = 89.0kN

$\sigma_c = \frac{89.0 \times 1000}{400 \times 100} = 2.2 N/mm^2 < 2.4 N/mm^2$ (NEW BUILD) SATISFACTORY

ADOPT 400 × 100 × 215mm DEEP CONCRETE PADSTONES

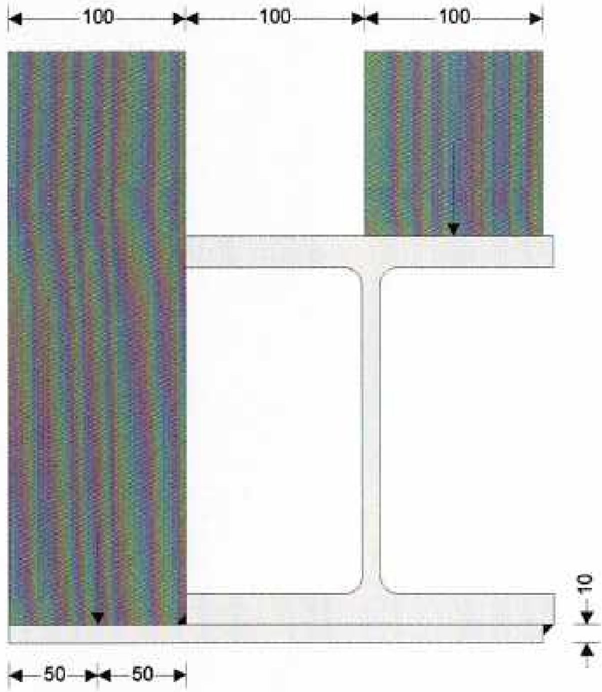
MIN 200mm END BEARING (SEE CALC L3)

Hudds Design Structural Engineering Consultants Hudds Design 103-105 Bradford Rd Huddersfield HD1 6DZ	Project Plot 1,2&5 Windmill Hill Quarry, Brierley Road, Barnsley, S72				Job no. HD-S21-1009	
	Calcs for Lintel L3 (Boot Lintel)				Start page no./Revision L3 - 1	
	Calcs by JT	Calcs date 12/10/2021	Checked by JA	Checked date 05/05/2021	Approved by	Approved date

STEEL MASONRY SUPPORT

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

Tedds calculation version 1.0.04



Steel member details

Torsion beam	UC 203x203x71	Masonry support angle	plate
Steel grade of support angle	User	Design strength support angle	p_{ysb} = **275** N/mm^2
Modulus of elasticity	E = 205000 N/mm^2	Constant	ε = **1.000**
Length of plate beyond beam	l_h = **100** mm	Total length of plate	l_{plate} = **300** mm
Thickness of plate	t_{sb} = **10** mm	Width of main beam	B_{mb} = **206** mm
Area of plate	A_{sbu} = **3000.0** mm^2	Dist weld position to CoG	c_{yysb} = **-50** mm

Supported materials detail

Density mas. main beam	$\rho_{m,mb}$ = **20.0** kN/m^3	Width masonry main beam	b_{mmb} = **100** mm
Height masonry main beam	h_{mmb} = **3000** mm		
Ecc. of main beam material	e_{mb} = **100** mm		
Add dead force main beam	P_{Gaddmb} = **8.0** kN/m	Add live force main beam	P_{Qaddmb} = **5.9** kN/m
Density mas. support beam	$\rho_{m,sb}$ = **24.0** kN/m^3	Width masonry support beam	b_{msb} = **100** mm
Height masonry support beam	h_{msb} = **3000** mm		
Add dead force support beam	P_{Gaddsb} = **0.0** kN/m	Add live force support beam	P_{Qaddsb} = **0.0** kN/m

Geometry

Cavity width	c = **100** mm	Supported width of masonry	d_m = **100** mm

Biaxial stress effects in the plate (SCI-P-110)

Max overall bending moment	M_x = **97.9** kNm	Dist to NA combined section	$y_{e,all}$ = **28** mm
Second moment of area	$I_{xx,all}$ = **10489** cm^4	Elastic section modulus	$Z_{xx,all}$ = **1168.40** cm^3
Section modulus of plate	$Z_{xx,plate}$ = **16.67** cm^3/m	Eccentricity on support beam	e_1 = **50** mm
Force on support plate	P_1 = **10.1** kN/m	Bending at heel	$M_{x,plate}$ = **0.5** kNm/m
Moment capacity of plate	M_c = **5.5** kNm/m		

PASS - Design strength exceeds stress at heel

Long stress overall bending	σ_1 = **83.8** N/mm^2	Von Mises curve constant	c_{fp} = **530.5** N/mm^2
Trans bending stress ratio limit	α_{ls} = **0.941**	Trans bending stress ratio	α_{ts} = **0.092**

PASS - Transverse bending stress ratio less than allowable limit

Deflection at toe

Unfact force on plate	P_{1SLS} = **7.2** kN/m	Distance from weld to load	a_m = **50** mm
Load resultant to edge of plate	b_m = **50** mm	Weld to load pos as ratio	a_l = **0.500**
Effect second mnt of inertia	I_{eff_def} = **83333** mm^4/m	Deflection at toe	δ = **0.04** mm
Deflection limit	δ_{lim} = **2.00** mm		

PASS - Deflection is within specified criteria

Weld details - assume a full length weld and that the plate acts as a propped cantilever with the prop at the weld position and the fixed end at the centre of the torsion beam

Leg length of weld	s_{weld} = **6** mm	Throat size of weld	a_{weld} = **4.2** mm
Shear force at weld position	R_A = **17.4** kN/m	Max possible force in plate	R_p = **842.6** kN
Long shear beam/plate	R_l = **383.0** kN/m	Horizontal shear beam/plate	R_h = **63.0** kN/m
Resultant weld force	R_{weld} = **0.389** kN/mm	Strength of weld (Table 37)	p_{weld} = **220.0** N/mm^2
Capacity of full length weld	$p_{c,weld}$ = **0.933** kN/mm		

$1/\sqrt{(2)} \times s_{weld}$

Torsional loading ULS

Loading support beam	w_{1ULS} = **10.08** kN/m	Loading of main beam	w_{2ULS} = **29.04** kN/m
Self weight of support beam	w_{3ULS} = **0.33** kN/m		

Torsional loading SLS

Loading support beam	w_{1SLS} = **7.20** kN/m	Loading of main beam	w_{2SLS} = **19.90** kN/m
Self weight of support beam	w_{3SLS} = **0.24** kN/m		

Eccentricities

Distance of shear centre	e_{0mb} = **0** mm	Ecc of support beam masonry	e_{1mb} = **153** mm
Ecc of main beam masonry	e_{2mb} = **-47** mm	Ecc of support beam	e_{3mb} = **53** mm

Torsional effects

Applied torque	T_{qULS} = **0.20** kNm/m	Torsional moment (ULS)	T_q = **0.89** kNm
Applied torque (SLS)	T_{qSLS} = **0.18** kNm/m	Torsional moment (SLS)	T_{qu} = **0.81** kNm

STEEL BEAM TORSION DESIGN

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

Tedds calculation version 2.0.02

Section details

Section type	UC 203x203x71	Steel grade	S275
Design stength	$p_{yw} = p_y$ = **265** N/mm^2	Constant	ε = **1.019**

Geometry - Beam unrestrained against lateral-torsional buckling between supports.

Effective span	L = **4400** mm		
Length of segment LTB	L_{LT} = **4400** mm	Effective length for LTB	L_{E_LT} = **4832** mm

Loading - Torsional loading comprises only full-length uniformly distributed load(s)

Internal forces & moments on member under factored loading for uls design

Applied shear force	F_{vy} = **89.0** kN	Maximum bending moment	$M_{LT} = M_x$ = **97.87** kNm
Applied torque	T_q = **0.89** kNm	Minor axis bending moment	M_y = 0 kNm
Compression force	F_c = 0 kN		

Equivalent uniform moment factors

EUM factor (Cl.4.3.6.6 & T18) m_{LT} = **1.000**

Torsional deflection parameters

Beam is torsion fixed and warping free at each end. (as defined in SCI-P-057 section 2.1.6) - Appendix B case 4

Dist for first deriv of twist z_1 = 0 mm | Dist for second deriv of twist z_2 = L / 2 = **2200** mm

First deriv of angle of twist ϕ'_1 = **4.21×10^{-3}** rads/m | Third deriv of angle of twist ϕ'''_1 = **-3.51×10^{-3}** rads/m^3

Angle of twist ϕ_2 = **0.006** rads | Second deriv of angle of twist ϕ''_2 = **-2.65×10^{-3}** rads/m^2

Design parameters

Total angle of twist ϕ = **0.006** rads | First derivative of ϕ ϕ' = **4.21×10^{-3}** rads/m

Second derivative of ϕ ϕ'' = **2.65×10^{-3}** rads/m^2 | Third derivative of ϕ ϕ''' = **3.51×10^{-3}** rads/m^3

Section classification

b / T = **6.0** | d / t = **16.1**

r_{1s} = **0.000** | r_{2s} = **0.000**

Section classification is plastic

Shear capacity (parallel to y-axis)

Design shear force F_{vy} = **89.0** kN | Design shear resist (cl. 4.2.3) P_{vy} = **343.1** kN

Pass - Shear

Moment capacity (x-axis)

Design bending moment M_x = **97.9** kNm | Mnt cap low shear (cl. 4.2.5.1) M_{cx} = **211.7** kNm

Pass - Moment capacity exceeds design bending moment

Lateral torsional buckling

Effective length for LTB L_{E_LT} = **4832** mm

Slenderness ratio λ = **91** | Buckling parameter u = **0.853**

Flange ratio η = 0.5 | Torsional index x = **11.9**

Slenderness factor v = **0.71** | Ratio - cl 4.3.6.9 β_w = **1.000**

Equiv slenderness – cl 4.3.6.7 λ_{LT} = **55** | Limit slendernes – Ann B2.2 λ_{L0} = **35**

Euler stress p_E = **663** N/mm^2 | Perry factor η_{LT} = **0.142**

ϕ_{LT} = **511007700.142** | Bending strength p_b = **219** N/mm^2

Buckling resistance moment M_b = **174.6** kNm | Max mnt gov buckling resist M_{LT} = **97.9** kNm

Equiv uniform mnt factor LTB m_{LT} = **1.00** | M_b / m_{LT} = **174.6** kNm

Pass - lat. tors. buckling

Buckling under combined bending & torsion -SCI-P-057 section 2.3

For simplicity, a conservative check is applied using the maximum stresses due to each of the separate load effects, even though these do not necessarily all occur at the same section along the member.

Span factor L / a = **4.89** | Angle of twist ϕ = **0.006** rads

Second derivative of ϕ ϕ'' = **2.65×10^{-3}** rads/m^2 | Induced minor axis moment M_{yt} = **0.55** kNm

Normal stress flange M_{yt} σ_{byt} = **2** N/mm^2 | Normal stress flange warping σ_w = **6** N/mm^2

Interaction index i_b = **0.60**

Pass - Combined bending and torsion check satisfied

Local capacity under combined bending & torsion

For simplicity, a conservative check is applied using the maximum stresses due to each of the separate load effects, even though these do not necessarily all occur at the same section along the member.

Max. direct stress due to M_x σ_{bx} = M_x / Z_x = **139** N/mm^2

Combined stress - eqn 2.22 $\sigma_{bx} + \sigma_{byt} + \sigma_w$ = **146** N/mm^2 | Design strength p_y = **265** N/mm^2

Pass - Local capacity

Combined shear stresses - SCI-P-057 section 2.3

For simplicity, a conservative check is applied using the maximum shear stresses due to each of the separate load effects, even though these do not necessarily all occur at the same section along the member.

Shear stress bending web	τ_{bw} = **47** N/mm^2	Shear stress bending flange	τ_{bf} = **12** N/mm^2
Shear stresses torsion web	τ_{tw} = **3** N/mm^2	Shear stresses torsion flange	τ_{tf} = **6** N/mm^2
Shear stresses warping flange	τ_{wf} = **0** N/mm^2	Shear stress tors & warp web	τ_{vtw} = **4** N/mm^2
Shear str tors & warp flange	τ_{vtf} = **8** N/mm^2		

Combined shear stresses due to bending, torsion & warping:

Comb shear stresses in web	τ_w = **51** N/mm^2	Comb shear stresses in flange	τ_f = **19** N/mm^2
Shear strength	p_v = **159** N/mm^2		

Pass - Combined shear stresses

Twist check

Total applied torque	T_{qu} = **0.81** kNm		
Max twist under sls loading	ϕ_{sls} = **0.29** deg	Twist limit	ϕ_{lim} = **2.00** deg

Pass - Twist

Deflection

Maximum y-axis deflection	δ_{y_max} = **8.8** mm	Deflection limit - cl. 2.5.2	δ_{lim} = **10.0** mm

Pass - Deflection within specified limit

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Project: PLOT 1, 2 & 5 WINDMILL HILL QUAY, S72 7AJ				Job No. HD-S21-1009	
Calcs for: LINTEL L4 (BOOT LINTEL)				Start Page: L4-1	Rev:
Calcs by: JT	Calcs Date: 12/10/2021	Checked:	Checked Date:	Approved:	Approved Date:

LINTEL L4 (BOOT LINTEL)

MAX CLEAR SPAN = 2400mm

SUPPORTING WALL HEIGHT = 3.3m
+ ROOF LENGTH = 3.0m (TRUSS)

LOADS ON LINTEL L4 (BOOT LINTEL)

ROOF DEAD LOAD = 3.0m × 1.3kN/m² = **3.9 kN/m**

ROOF IMPOSED LOAD = 3.0m × 0.6kN/m² = **1.8 kN/m**

ADOPT 152×152×37 UC WITH 10mm PLATE FULLY WELDED TO BOTTOM
MIN 6mm FILLET WELDS (SEE CALC L4)

PADSTONES

R = 35.4 kN

$\sigma_c = \frac{35.4 \times 1000}{200 \times 100} = 1.8 N/mm^2 < 2.4 N/mm^2$ (NEW BUILD) **SATISFACTORY**

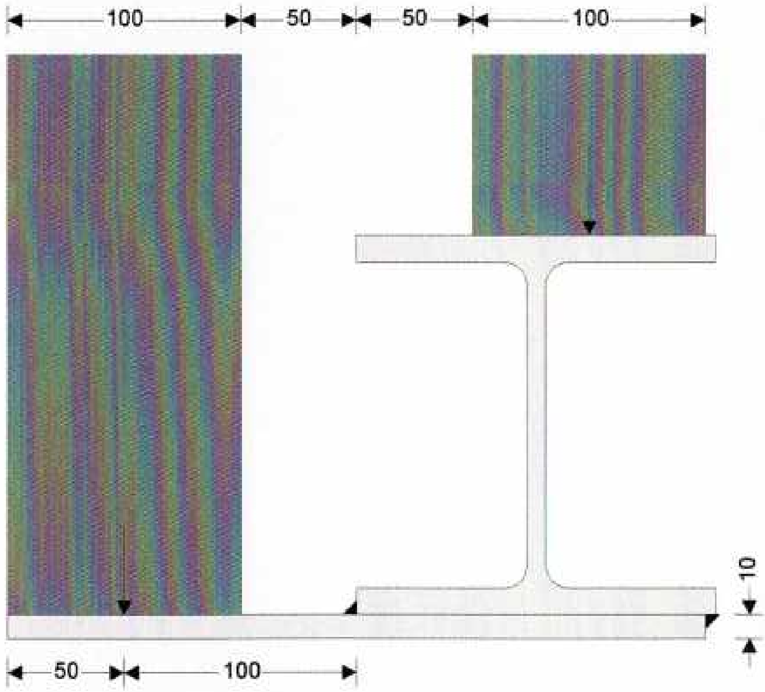
ADOPT 200×100×215mm DEEP CONCRETE PADSTONES

Hudds Design Structural Engineering Consultants Hudds Design 103-105 Bradford Rd Huddersfield HD1 6DZ	Project Plot 1,2&5 Windmill Hill Quarry, Brierley Road, Barnsley, S72				Job no. HD-S21-1009	
	Calcs for Lintel L4 (Boot Lintel)				Start page no./Revision L4 - 1	
	Calcs by JT	Calcs date 12/10/2021	Checked by JA	Checked date 05/05/2021	Approved by	Approved date

STEEL MASONRY SUPPORT

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

Tedds calculation version 1.0.04



Steel member details

Torsion beam	UC 152x152x37	Masonry support angle	plate
Steel grade of support angle	User	Design strength support angle	p_{ysb} = **275** N/mm^2
Modulus of elasticity	E = 205000 N/mm^2	Constant	ε = **1.000**
Length of plate beyond beam	l_h = **150** mm	Total length of plate	l_{plate} = **300** mm
Thickness of plate	t_{sb} = **10** mm	Width of main beam	B_{mb} = **154** mm
Area of plate	A_{sbu} = **3000.0** mm^2	Dist weld position to CoG	c_{yysb} = **0** mm

Supported materials detail

Density mas. main beam	$\rho_{m,mb}$ = **20.0** kN/m^3	Width masonry main beam	b_{mmb} = **100** mm
Height masonry main beam	h_{mmb} = **3300** mm		
Ecc. of main beam material	e_{mb} = **50** mm		
Add dead force main beam	P_{Gaddmb} = **3.9** kN/m	Add live force main beam	P_{Qaddmb} = **1.8** kN/m
Density mas. support beam	$\rho_{m,sb}$ = **24.0** kN/m^3	Width masonry support beam	b_{msb} = **100** mm
Height masonry support beam	h_{msb} = **3300** mm		
Add dead force support beam	P_{Gaddsb} = **0.0** kN/m	Add live force support beam	P_{Qaddsb} = **0.0** kN/m

Geometry

Cavity width	c = **100** mm	Supported width of masonry	d_m = **100** mm

Biaxial stress effects in the plate (SCI-P-110)

Max overall bending moment	M_x = **21.3** kNm	Dist to NA combined section	$y_{e,all}$ = **33** mm
Second moment of area	$I_{xx,all}$ = **3563** cm^4	Elastic section modulus	$Z_{xx,all}$ = **619.84** cm^3
Section modulus of plate	$Z_{xx,plate}$ = **16.67** cm^3/m	Eccentricity on support beam	e_1 = **100** mm
Force on support plate	P_1 = **11.1** kN/m	Bending at heel	$M_{x,plate}$ = **1.1** kNm/m
Moment capacity of plate	M_c = **5.5** kNm/m		

PASS - Design strength exceeds stress at heel

Long stress overall bending σ_1 = **34.3** N/mm^2 — Von Mises curve constant c_{fp} = **546.8** N/mm^2

Trans bending stress ratio limit α_{ts} = **0.990** — Trans bending stress ratio α_{ls} = **0.202**

PASS - Transverse bending stress ratio less than allowable limit

Deflection at toe

Unfact force on plate P_{1SLS} = **7.9** kN/m — Distance from weld to load a_m = **100** mm

Load resultant to edge of plate b_m = **50** mm — Weld to load pos as ratio a_l = **0.667**

Effect second mnt of inertia I_{eff_def} = **83333** mm^4/m — Deflection at toe δ = **0.27** mm

Deflection limit δ_{lim} = **2.00** mm

PASS - Deflection is within specified criteria

Weld details - assume a full length weld and that the plate acts as a propped cantilever with the prop at the weld position and the fixed end at the centre of the torsion beam

Leg length of weld s_{weld} = **6** mm — Throat size of weld a_{weld} = **4.2** mm

Shear force at weld position R_A = **32.6** kN/m — Max possible force in plate R_p = **837.1** kN

Long shear beam/plate R_l = **697.6** kN/m — Horizontal shear beam/plate R_h = **138.6** kN/m

Resultant weld force R_{weld} = **0.712** kN/mm — Strength of weld (Table 37) p_{weld} = **220.0** N/mm^2

Capacity of full length weld $p_{c,weld}$ = **0.933** kN/mm

$1/\sqrt{2} \times s_{weld}$

Torsional loading ULS

Loading support beam w_{1ULS} = **11.09** kN/m — Loading of main beam w_{2ULS} = **17.58** kN/m

Self weight of support beam w_{3ULS} = **0.33** kN/m

Torsional loading SLS

Loading support beam w_{1SLS} = **7.92** kN/m — Loading of main beam w_{2SLS} = **12.30** kN/m

Self weight of support beam w_{3SLS} = **0.24** kN/m

Eccentricities

Distance of shear centre e_{0mb} = **0** mm — Ecc of support beam masonry e_{1mb} = **177** mm

Ecc of main beam masonry e_{2mb} = **-23** mm — Ecc of support beam e_{3mb} = **77** mm

Torsional effects

Applied torque T_{qULS} = **1.59** kNm/m — Torsional moment (ULS) T_q = **3.81** kNm

Applied torque (SLS) T_{qSLS} = **1.14** kNm/m — Torsional moment (SLS) T_{qu} = **2.74** kNm

STEEL BEAM TORSION DESIGN

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

Tedds calculation version 2.0.02

Section details

Section type UC 152x152x37 — Steel grade S275

Design stength $p_{yw} = p_y$ = **275** N/mm^2 — Constant ε = **1.000**

Geometry - Beam unrestrained against lateral-torsional buckling between supports.

Effective span L = **2400** mm

Length of segment LTB L_{LT} = **2400** mm — Effective length for LTB L_{E_LT} = **2724** mm

Loading - Torsional loading comprises only full-length uniformly distributed load(s)

Internal forces & moments on member under factored loading for uls design

Applied shear force F_{vy} = **35.4** kN — Maximum bending moment $M_{LT} = M_x$ = **21.25** kNm

Applied torque T_q = **3.81** kNm — Minor axis bending moment M_y = 0 kNm

Compression force F_c = 0 kN

Equivalent uniform moment factors

EUM factor (Cl.4.3.6.6 & T18) m_{LT} = **1.000**

Torsional deflection parameters

Beam is torsion fixed and warping free at each end. (as defined in SCI-P-057 section 2.1.6) - Appendix B case 4

Dist for first deriv of twist	$z_1 = 0$ mm	Dist for second deriv of twist	$z_2 = L / 2 =$ **1200** mm
First deriv of angle of twist	$\phi'_1 = \mathbf{5.46 \times 10^{-2}}$ rads/m	Third deriv of angle of twist	$\phi'''_1 = \mathbf{-1.32 \times 10^{-1}}$ rads/m³
Angle of twist	$\phi_2 =$ **0.040** rads	Second deriv of angle of twist	$\phi''_2 = \mathbf{-6.55 \times 10^{-2}}$ rads/m²

Design parameters

Total angle of twist	$\phi =$ **0.040** rads	First derivative of ϕ	$\phi' = \mathbf{5.46 \times 10^{-2}}$ rads/m
Second derivative of ϕ	$\phi'' = \mathbf{6.55 \times 10^{-2}}$ rads/m²	Third derivative of ϕ	$\phi''' = \mathbf{1.32 \times 10^{-1}}$ rads/m³

Section classification

b / T = **6.7** | d / t = **15.5**

r_{1s} = **0.000** | r_{2s} = **0.000**

Section classification is plastic

Shear capacity (parallel to y-axis)

Design shear force F_{vy} = **35.4** kN | Design shear resist (cl. 4.2.3) P_{vy} = **213.6** kN

Pass - Shear

Moment capacity (x-axis)

Design bending moment M_x = **21.3** kNm | Mnt cap low shear (cl. 4.2.5.1) M_{cx} = **84.9** kNm

Pass - Moment capacity exceeds design bending moment

Lateral torsional buckling

Effective length for LTB	L_{E_LT} = **2724** mm		
Slenderness ratio	λ = **70**	Buckling parameter	u = **0.848**
Flange ratio	$\eta = 0.5$	Torsional index	x = **13.3**
Slenderness factor	v = **0.80**	Ratio - cl 4.3.6.9	β_w = **1.000**
Equiv slenderness – cl 4.3.6.7	λ_{LT} = **48**	Limit slendernes – Ann B2.2	λ_{L0} = **34**
Euler stress	p_E = **879** N/mm²	Perry factor	η_{LT} = **0.096**
	ϕ_{LT} = **619015412.780**	Bending strength	p_b = **243** N/mm²
Buckling resistance moment	M_b = **75.0** kNm	Max mnt gov buckling resist	M_{LT} = **21.3** kNm
Equiv uniform mnt factor LTB	m_{LT} = **1.00**		M_b / m_{LT} = **75.0** kNm

Pass - lat. tors. buckling

Buckling under combined bending & torsion -SCI-P-057 section 2.3

For simplicity, a conservative check is applied using the maximum stresses due to each of the separate load effects, even though these do not necessarily all occur at the same section along the member.

Span factor	L / a = **3.26**	Angle of twist	ϕ = **0.040** rads
Second derivative of ϕ	$\phi'' = \mathbf{65.5 \times 10^{-3}}$ rads/m²	Induced minor axis moment	M_{yt} = **0.86** kNm
Normal stress flange M_{yt}	σ_{byt} = **9** N/mm²	Normal stress flange warping	σ_w = **78** N/mm²
Interaction index	i_b = **0.65**		

Pass - Combined bending and torsion check satisfied

Local capacity under combined bending & torsion

For simplicity, a conservative check is applied using the maximum stresses due to each of the separate load effects, even though these do not necessarily all occur at the same section along the member.

Max. direct stress due to M_x $\sigma_{bx} = M_x / Z_x$ = **78** N/mm²

Combined stress - eqn 2.22 $\sigma_{bx} + \sigma_{byt} + \sigma_w$ = **165** N/mm² | Design strength p_y = **275** N/mm²

Pass - Local capacity

Combined shear stresses - SCI-P-057 section 2.3

For simplicity, a conservative check is applied using the maximum shear stresses due to each of the separate load effects, even though these do not necessarily all occur at the same section along the member.

Shear stress bending web τ_{bw} = **31** N/mm^2

Shear stress bending flange τ_{bf} = **9** N/mm^2

Shear stresses torsion web τ_{tw} = **34** N/mm^2

Shear stresses torsion flange τ_{tf} = **49** N/mm^2

Shear stresses warping flange τ_{wf} = **6** N/mm^2

Shear stress tors & warp web τ_{vtw} = **39** N/mm^2

Shear str tors & warp flange τ_{vtf} = **63** N/mm^2

Combined shear stresses due to bending, torsion & warping:

Comb shear stresses in web τ_w = **70** N/mm^2

Comb shear stresses in flange τ_f = **72** N/mm^2

Shear strength p_v = **165** N/mm^2

Pass - Combined shear stresses

Twist check

Total applied torque T_{qu} = **2.74** kNm

Max twist under sls loading ϕ_{sls} = **1.66** deg

Twist limit ϕ_{lim} = **2.00** deg

Pass - Twist

Deflection

Maximum y-axis deflection δ_{y_max} = **2.0** mm

Deflection limit - cl. 2.5.2 δ_{lim} = **6.7** mm

Pass - Deflection within specified limit

Hudds Design — Structural Engineering Consultants — www.huddsdesign.com					
Project: PLOT 1, 2, & 5 WINDMILL HILL QUAY ST7AJ				Job No. HD-S21-1004	
Calcs for: FOUNDATION LINE LOADS				Start Page: FL-1	Rev:
Calcs by: JY	Calcs Date: 12/10/2021	Checked:	Checked Date:	Approved:	Approved Date:

FOUNDATION LINE LOADS

GRID 1/2/3 => EXTERNAL LOADS (WORST CASE)

WALL DEAD LOAD = 6.4m × 4.4kN/m² = 28.2kN/m
ROOF DEAD LOAD = 4.5m × 1.3kN/m² = 5.9kN/m
FLOOR DEAD LOAD = 2.1m × 1.0kN/m² = 2.1kN/m
} = 36.2kN/m

ROOF IMPOSED LOAD = 4.5m × 0.6kN/m² = 2.7kN/m
FLOOR IMPOSED LOAD = 2.1m × 1.5kN/m² = 3.2kN/m
} = 5.9kN/m

GRID D => INTERNAL LOADS (WORST CASE)

WALL DEAD LOAD = 8.0m × 4.4kN/m² = 35.2kN/m
FLOOR DEAD LOAD = 2.8m × 1.0kN/m² = 2.8kN/m
} = 38.0kN/m

FLOOR IMPOSED LOAD = 2.8m × 1.5kN/m² = 3.2kN/m

GRID B/C => INTERNAL LOADS

WALL DEAD LOAD = 2.7m × 2.0kN/m² = 5.4kN/m
FLOOR DEAD LOAD = 2.9m × 1.0kN/m² = 2.9kN/m
} = 8.3kN/m

FLOOR IMPOSED LOAD = 2.9m × 1.5kN/m² = 4.4kN/m

RAFT FOUNDATION RF

MAX SPAN = 9800mm

LOADS ON RAFT FOUNDATION => WORST CASE LOADS

GRID D (EDGE LOAD) : DEAD LOAD = 35.2kN/m
IMPOSED LOAD = 5.9 kN/m

GRID B/C (INTERNAL LOAD) : DEAD LOAD = 38.0kN/m
IMPOSED LOAD = 3.2 kN/m

SEE CALC FL-1 FOR DERIVATION OF LOADS ABOVE

ADOPT 200mm DEEP RC RAFT SLAB

WITH 2 LAYERS A393 MESH TOP & BOTTOM

(MIN GRADE C30 => 40mm COVER

PROVIDE EDGE & INTERNAL THICKENING

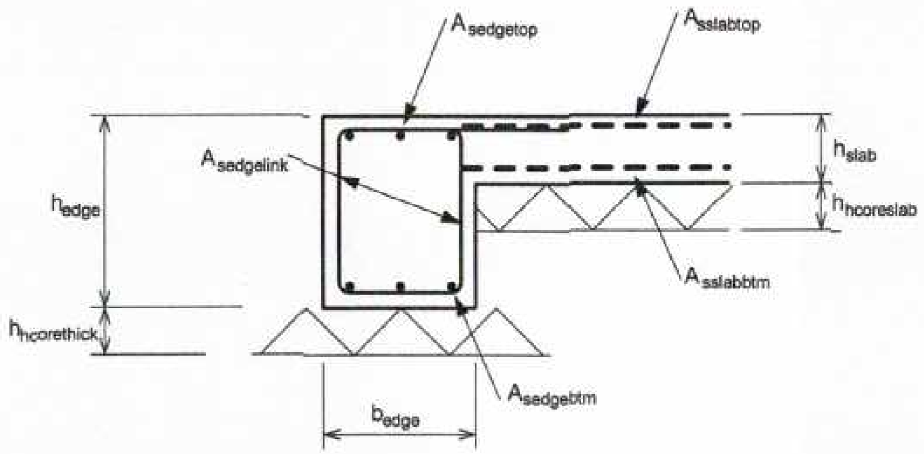
450 x 450mm WITH 3 NO. T20 BARS (TOP & BOTTOM)

WITH 10mm SHEAR LINKS @ 200mm CTRS

(SEE CALC RF)

RAFT FOUNDATION DESIGN (BS8110 : PART 1 : 1997)

Tedds calculation version 1.0.12



Soil and raft definition

Soil definition

Allowable bearing pressure q_{allow} = **25.0** 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 $h_{hcoreslab}$ = **100** 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 $\phi_{depslab} = \phi_{depbasic} - h_{hcoreslab}$ = **3400** 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} = **9.800** m

Slab thickness h_{slab} = **200** mm

Concrete strength f_{cu} = **25** N/mm^2

Poissons ratio of concrete ν = **0.2**

Slab mesh reinforcement strength f_{yslab} = **500** N/mm^2

Partial safety factor for steel reinforcement γ_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 **2 x A393 ($A_{sslabtop}$ = 786 mm^2/m)**

Mesh provided in bottom **2 x A393 ($A_{sslabbtm}$ = 786 mm^2/m)**

Top mesh bar diameter $\phi_{slabtop}$ = **10** mm

Bottom mesh bar diameter $\phi_{slabbtm}$ = **10** mm

Cover to top reinforcement c_{top} = **40** mm

Cover to bottom reinforcement c_{btm} = **40** mm

Average effective depth of top reinforcement $d_{tslabav} = h_{slab} - c_{top} - \phi_{slabtop}$ = **150** mm

Average effective depth of bottom reinforcement $d_{bslabav} = h_{slab} - c_{btm} - \phi_{slabbtm}$ = **150** mm

Overall average effective depth $d_{slabav} = (d_{tslabav} + d_{bslabav})/2$ = **150** mm

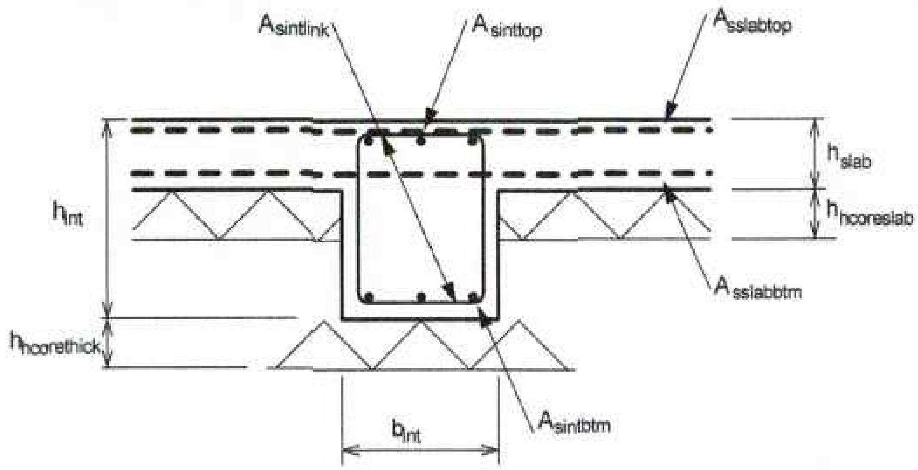
Minimum effective depth of top reinforcement $d_{tslabmin} = d_{tslabav} - \phi_{slabtop}/2 = \mathbf{145}$ mm

Minimum effective depth of bottom reinforcement $d_{bslabmin} = d_{bslabav} - \phi_{slabbtm}/2 = \mathbf{145}$ mm

Edge beam definition

Overall depth	h_{edge} = **450** mm
Width	b_{edge} = **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 H20 bars ($A_{sedgetop}$ = 942 mm^2)**
Reinforcement provided in bottom	**3 H20 bars ($A_{sedgebtm}$ = 942 mm^2)**
Link reinforcement provided	**2 H10 legs at 200 ctrs (A_{sv}/s_v = 0.785 mm)**
Bottom cover to links	c_{beam} = **40** mm
Effective depth of top reinforcement	$d_{edgetop} = h_{edge} - c_{top} - \phi_{slabtop} - \phi_{edgelink} - \phi_{edgetop}/2 = \mathbf{380}$ mm
Effective depth of bottom reinforcement	$d_{edgebtm} = h_{edge} - c_{beam} - \phi_{edgelink} - \phi_{edgebtm}/2 = \mathbf{390}$ mm

Internal beam definition



Overall depth	h_{int} = **450** 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 H20 bars ($A_{sinttop}$ = 942 mm^2)**
Reinforcement provided in bottom	**3 H20 bars ($A_{sintbtm}$ = 942 mm^2)**
Link reinforcement provided	**2 H10 legs at 200 ctrs (A_{sv}/s_v = 0.785 mm)**
Effective depth of top reinforcement	$d_{inttop} = h_{int} - c_{top} - 2 \times \phi_{slabtop} - \phi_{inttop}/2 = \mathbf{380}$ mm
Effective depth of bottom reinforcement	$d_{intbtm} = h_{int} - c_{beam} - \phi_{intlink} - \phi_{intbtm}/2 = \mathbf{390}$ mm

Internal slab design checks

Basic loading

Slab self weight	$w_{slab} = 24$ kN/m$^3 \times h_{slab} = \mathbf{4.8}$ kN/m^2
Hardcore	$w_{hcoreslab} = \gamma_{hcore} \times h_{hcoreslab} = \mathbf{2.0}$ kN/m^2

Applied loading

Uniformly distributed dead load	w_{Dudl} = **1.5** kN/m^2
Uniformly distributed live load	w_{Ludl} = **2.5** kN/m^2

Internal slab bearing pressure check

Total uniform load at formation level — $w_{udl} = w_{slab} + w_{hcoreslab} + w_{Dudl} + w_{Ludl} = \mathbf{10.8}$ kN/m²

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

Internal slab bending and shear check

Applied bending moments

Span of slab — $l_{slab} = \phi_{depslab} + d_{tslabav} = \mathbf{3550}$ mm

Ultimate self weight udl — $w_{swult} = 1.4 \times w_{slab} = \mathbf{6.7}$ kN/m²

Self weight moment at centre — $M_{csw} = w_{swult} \times l_{slab}^2 \times (1 + \nu) / 64 = \mathbf{1.6}$ kNm/m

Self weight moment at edge — $M_{esw} = w_{swult} \times l_{slab}^2 / 32 = \mathbf{2.6}$ kNm/m

Self weight shear force at edge — $V_{sw} = w_{swult} \times l_{slab} / 4 = \mathbf{6.0}$ kN/m

Moments due to applied uniformly distributed loads

Ultimate applied udl — $w_{udlult} = 1.4 \times w_{Dudl} + 1.6 \times w_{Ludl} = \mathbf{6.1}$ kN/m²

Moment at centre — $M_{cudl} = w_{udlult} \times l_{slab}^2 \times (1 + \nu) / 64 = \mathbf{1.4}$ kNm/m

Moment at edge — $M_{eudl} = w_{udlult} \times l_{slab}^2 / 32 = \mathbf{2.4}$ kNm/m

Shear force at edge — $V_{udl} = w_{udlult} \times l_{slab} / 4 = \mathbf{5.4}$ kN/m

Resultant moments and shears

Total moment at edge — $M_{\Sigma e} = \mathbf{5.0}$ kNm/m

Total moment at centre — $M_{\Sigma c} = \mathbf{3.0}$ kNm/m

Total shear force — $V_{\Sigma} = \mathbf{11.4}$ kN/m

Reinforcement required in top

K factor — $K_{slabtop} = M_{\Sigma e}/(f_{cu} \times d_{tslabav}^2) = \mathbf{0.009}$

Lever arm — $z_{slabtop} = d_{tslabav} \times \min(0.95, 0.5 + \sqrt{(0.25 - K_{slabtop}/0.9)}) = \mathbf{142.5}$ mm

Area of steel required for bending — $A_{sslabtopbend} = M_{\Sigma e}/((1.0/\gamma_s) \times f_{yslab} \times z_{slabtop}) = \mathbf{81}$ mm²/m

Minimum area of steel required — $A_{sslabmin} = 0.0013 \times h_{slab} = \mathbf{260}$ mm²/m

Area of steel required — $A_{sslabtopreq} = \max(A_{sslabtopbend}, A_{sslabmin}) = \mathbf{260}$ mm²/m

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

Reinforcement required in bottom

K factor — $K_{slabbtm} = M_{\Sigma c}/(f_{cu} \times d_{bslabav}^2) = \mathbf{0.005}$

Lever arm — $z_{slabbtm} = d_{bslabav} \times \min(0.95, 0.5 + \sqrt{(0.25 - K_{slabbtm}/0.9)}) = \mathbf{142.5}$ mm

Area of steel required for bending — $A_{sslabbtmbend} = M_{\Sigma c}/((1.0/\gamma_s) \times f_{yslab} \times z_{slabbtm}) = \mathbf{49}$ mm²/m

Area of steel required — $A_{sslabbtmreq} = \max(A_{sslabbtmbend}, A_{sslabmin}) = \mathbf{260}$ mm²/m

PASS - $A_{sslabbtmreq} <= A_{sslabbtm}$ - Area of reinforcement provided in bottom to span local depressions is adequate

Shear check

Applied shear stress — $v = V_{\Sigma}/d_{tslabmin} = \mathbf{0.078}$ N/mm²

Tension steel ratio — $\rho = 100 \times A_{sslabtop}/d_{tslabmin} = \mathbf{0.542}$

From BS8110-1:1997 - Table 3.8

Design concrete shear strength — $v_c = \mathbf{0.664}$ N/mm²

PASS - $v <= v_c$ - Shear capacity of the slab is adequate

Internal slab deflection check

Basic allowable span to depth ratio — $Ratio_{basic} = \mathbf{26.0}$

Moment factor — $M_{factor} = M_{\Sigma c}/d_{bslabav}^2 = \mathbf{0.135}$ N/mm²

Steel service stress — $f_s = 2/3 \times f_{yslab} \times A_{sslabbtmbend}/A_{sslabbtm} = \mathbf{20.736}$ N/mm²

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

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

Modified allowable span to depth ratio — $Ratio_{allow} = Ratio_{basic} \times MF_{slab}$ = **52.000**

Actual span to depth ratio — $Ratio_{actual} = l_{slab}/ d_{pslabav}$ = **23.667**

PASS - $Ratio_{actual}$ <= $Ratio_{allow}$ - Slab span to depth ratio is adequate

Edge beam design checks

Basic loading

Hardcore — $w_{hcorethick} = \gamma_{hcore} \times h_{hcorethick}$ = **3.0** kN/m²

Edge beam self weight — $w_{edge} = 24 \text{ kN/m}^3 \times h_{edge} \times b_{edge}$ = **4.9** kN/m

Edge load number 1

Load type — **Longitudinal line load**

Dead load — w_{Dedge1} = **36.2** kN/m

Live load — w_{Ledge1} = **5.9** kN/m

Ultimate load — $w_{ultedge1} = 1.4 \times w_{Dedge1} + 1.6 \times w_{Ledge1}$ = **60.1** kN/m

Longitudinal line load width — b_{edge1} = **300** mm

Centroid of load from outside face of raft — x_{edge1} = **150** mm

Edge beam bearing pressure check

Effective bearing width of edge beam — $b_{bearing} = b_{edge}$ = **450** mm

Total uniform load at formation level — $w_{udledge} = w_{Dudl}+w_{Ludl}+w_{edge}/b_{bearing}+w_{hcorethick}$ = **17.8** kN/m²

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

Load x distance for edge load 1 — $Moment_1 = w_{ultedge1} \times x_{edge1}$ = **9.0** kN

Sum of ultimate longitud'l and equivalent line loads — ΣUDL = **60.1** kN/m

Sum of load x distances — $\Sigma Moment$ = **9.0** kN

Centroid of loads — $x_{bar} = \Sigma Moment/\Sigma UDL$ = **150** mm

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

Sum of unfactored longitud'l and eff'tive line loads — $\Sigma UDLsls$ = **42.1** kN/m

Allowable bearing width — $b_{allow} = 2 \times x_{bar} + 2 \times h_{hcoreslab} \times \tan(30)$ = **415** mm

Bearing pressure due to line/point loads — $q_{linepoint} = \Sigma UDLsls/ b_{allow}$ = **101.3** kN/m²

Total applied bearing pressure — $q_{edge} = q_{linepoint} + w_{udledge}$ = **119.1** kN/m²

$q_{edge} > q_{allow}$ - The slab is required to resist a moment due to eccentricity

Now assume moment due to load/reaction eccentricity is resisted by slab

Bearing width required — $b_{req} = \Sigma UDLsls/(q_{allow} - w_{udledge})$ = **5847** mm

Effective bearing width at u/s of slab — $b_{reqeff} = b_{req} - 2 \times h_{hcoreslab} \times \tan(30)$ = **5732** mm

Load/reaction eccentricity — $e = b_{reqeff}/2 - x_{bar}$ = **2716** mm

Ultimate moment to be resisted by slab — $M_{ecc} = \Sigma UDL \times e$ = **163.3** kNm/m

From slab bending check

Moment due to depression under slab (hogging) — $M_{\Sigma e}$ = **5.0** kNm/m

Total moment to be resisted by slab top steel — $M_{slabtop} = M_{ecc} + M_{\Sigma e}$ = **168.3** kNm/m

K factor — $K_{slab} = M_{slabtop}/(f_{cu} \times d_{slabmin}^2)$ = **0.320**

K > 0.156 - Therefore compression reinforcement is required

The design is outside the scope of this calculation

Edge beam bending check

Divider for moments due to udl's — β_{udl} = **10.0**

Applied bending moments

Span of edge beam — $l_{edge} = \phi_{depthick} + d_{edgetop}$ = **3730** mm

Ultimate self weight udl — $w_{edgeult} = 1.4 \times w_{edge}$ = **6.8** kN/m

Ultimate slab udl (approx) $w_{edgeslab} = \max(0 \text{ kN/m}, 1.4 \times w_{slab} \times ((\phi_{depthick}/2 \times 3/4) - b_{edge}))$ = **5.4** kN/m

Self weight and slab bending moment $M_{edgesw} = (w_{edgeult} + w_{edgeslab}) \times l_{edge}^2/\beta_{udl}$ = **17.0** kNm

Self weight shear force $V_{edgesw} = (w_{edgeult} + w_{edgeslab}) \times l_{edge}/2$ = **22.8** kN

Moments due to applied uniformly distributed loads

Ultimate udl (approx) $w_{edgeudl} = w_{udlult} \times \phi_{depthick}/2 \times 3/4$ = **7.7** kN/m

Bending moment $M_{edgeudl} = w_{edgeudl} \times l_{edge}^2/\beta_{udl}$ = **10.7** kNm

Shear force $V_{edgeudl} = w_{edgeudl} \times l_{edge}/2$ = **14.3** kN

Moment and shear due to load number 1

Bending moment $M_{edge1} = w_{ultedge1} \times l_{edge}^2/\beta_{udl}$ = **83.6** kNm

Shear force $V_{edge1} = w_{ultedge1} \times l_{edge}/2$ = **112.1** kN

Resultant moments and shears

Total moment (hogging and sagging) $M_{\Sigma edge}$ = **111.3** kNm

Maximum shear force $V_{\Sigma edge}$ = **149.2** kN

Reinforcement required in top

Width of section in compression zone $b_{edgetop} = b_{edge}$ = **450** mm

Average web width $b_w = b_{edge}$ = **450** mm

K factor $K_{edgetop} = M_{\Sigma edge}/(f_{cu} \times b_{edgetop} \times d_{edgetop}^2)$ = **0.069**

Lever arm $z_{edgetop} = d_{edgetop} \times \min(0.95, 0.5 + \sqrt{(0.25 - K_{edgetop}/0.9)})$ = **348** mm

Area of steel required for bending $A_{sedgetopbend} = M_{\Sigma edge}/((1.0/\gamma_s) \times f_y \times z_{edgetop})$ = **735** mm²

Minimum area of steel required $A_{sedgetopmin} = 0.0013 \times 1.0 \times b_w \times h_{edge}$ = **263** mm²

Area of steel required $A_{sedgetopreq} = \max(A_{sedgetopbend}, A_{sedgetopmin})$ = **735** mm²

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 $b_{edgebtm} = b_{edge} + 0.1 \times l_{edge}$ = **823** mm

K factor $K_{edgebtm} = M_{\Sigma edge}/(f_{cu} \times b_{edgebtm} \times d_{edgebtm}^2)$ = **0.036**

Lever arm $z_{edgebtm} = d_{edgebtm} \times \min(0.95, 0.5 + \sqrt{(0.25 - K_{edgebtm}/0.9)})$ = **371** mm

Area of steel required for bending $A_{sedgebtmbend} = M_{\Sigma edge}/((1.0/\gamma_s) \times f_y \times z_{edgebtm})$ = **691** mm²

Minimum area of steel required $A_{sedgebtmmin} = 0.0013 \times 1.0 \times b_w \times h_{edge}$ = **263** mm²

Area of steel required $A_{sedgebtmreq} = \max(A_{sedgebtmbend}, A_{sedgebtmmin})$ = **691** mm²

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

Edge beam shear check

Applied shear stress $v_{edge} = V_{\Sigma edge}/(b_w \times d_{edgetop})$ = **0.873** N/mm²

Tension steel ratio $\rho_{edge} = 100 \times A_{sedgetop}/(b_w \times d_{edgetop})$ = **0.551**

From BS8110-1:1997 - Table 3.8

Design concrete shear strength v_{cedge} = **0.525** N/mm²

v_{edge} <= v_{cedge} + 0.4N/mm² - Therefore minimum links required

Link area to spacing ratio required $A_{sv\_}upon\_s_{vreqedge} = 0.4\text{N/mm}^2 \times b_w/((1.0/\gamma_s) \times f_{ys})$ = **0.414** mm

Link area to spacing ratio provided $A_{sv\_}upon\_s_{vprovedge} = N_{edgelink} \times \pi \times \phi_{edgelink}^2/(4 \times s_{vedge})$ = **0.785** mm

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

__Corner design checks__

Basic loading

Corner bearing pressure check

Total uniform load at formation level $w_{udlcorner} = w_{Dudl} + w_{Ludl} + W_{edge}/b_{bearing} + w_{hcorethick}$ = **17.8** kN/m²

PASS - $w_{udlcorner}$ <= q_{allow} - Applied bearing pressure is less than allowable

Corner beam bending check

Cantilever span of edge beam — $l_{corner} = \phi_{depthick}/\sqrt{(2)} + d_{edgetop}/2$ = **2559** mm

Moment and shear due to self weight

Ultimate self weight udl — $w_{edgeult} = 1.4 \times w_{edge}$ = **6.8** kN/m

Average ultimate slab udl (approx) — $w_{cornerslab} = \max(0 \text{ kN/m}, 1.4 \times w_{slab} \times (\phi_{depthick}/(\sqrt{(2)} \times 2) - b_{edge}))$ = **4.9** kN/m

Self weight and slab bending moment — $M_{cornersw} = (w_{edgeult} + w_{cornerslab}) \times l_{corner}^2/2$ = **38.4** kNm

Self weight and slab shear force — $V_{cornersw} = (w_{edgeult} + w_{cornerslab}) \times l_{corner}$ = **30.0** kN

Moment and shear due to udls

Maximum ultimate udl — $w_{cornerudl} = ((1.4 \times w_{Dudl}) + (1.6 \times w_{Ludl})) \times \phi_{depthick}/\sqrt{(2)}$ = **14.4** kN/m

Bending moment — $M_{cornerudl} = w_{cornerudl} \times l_{corner}^2/6$ = **15.8** kNm

Shear force — $V_{cornerudl} = w_{cornerudl} \times l_{corner}/2$ = **18.5** kN

Resultant moments and shears

Total design moment — $M_{\Sigma corner} = M_{cornersw} + M_{cornerudl}$ = **54.2** kNm

Total design shear force — $V_{\Sigma corner} = V_{cornersw} + V_{cornerudl}$ = **48.5** kN

Reinforcement required in top of edge beam

K factor — $K_{corner} = M_{\Sigma corner}/(f_{cu} \times b_{edgetop} \times d_{edgetop}^2)$ = **0.033**

Lever arm — $z_{corner} = d_{edgetop} \times \min(0.95, 0.5 + \sqrt{(0.25 - K_{corner}/0.9)})$ = **361** mm

Area of steel required for bending — $A_{scornerbend} = M_{\Sigma corner}/((1.0/\gamma_s) \times f_y \times z_{corner})$ = **345** mm²

Minimum area of steel required — $A_{scornermin} = A_{sedgetopmin}$ = **263** mm²

Area of steel required — $A_{scorner} = \max(A_{scornerbend}, A_{scornermin})$ = **345** mm²

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 — $b_w = b_{edge}$ = **450** mm

Applied shear stress — $v_{corner} = V_{\Sigma corner}/(b_w \times d_{edgetop})$ = **0.284** N/mm²

Tension steel ratio — $\rho_{corner} = 100 \times A_{sedgetop}/(b_w \times d_{edgetop})$ = **0.551**

From BS8110-1:1997 - Table 3.8

Design concrete shear strength — $v_{ccorner}$ = **0.518** N/mm²

$v_{corner} <= v_{ccorner}$ + 0.4N/mm² - Therefore minimum links required

Link area to spacing ratio required — $A_{sv}\_upon\_s_{vreqcorner} = 0.4\text{N/mm}^2 \times b_w/((1.0/\gamma_s) \times f_{ys})$ = **0.414** mm

Link area to spacing ratio provided — $A_{sv}\_upon\_s_{vprovedge} = N_{edgelink} \times \pi \times \phi_{edgelink}^2/(4 \times s_{vedge})$ = **0.785** mm

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

Corner beam deflection check

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

Moment factor — $M_{factorcorner} = M_{\Sigma corner}/(b_{edgetop} \times d_{edgetop}^2)$ = **0.834** N/mm²

Steel service stress — $f_{scorner} = 2/3 \times f_y \times A_{scornerbend}/A_{sedgetop}$ = **122.130** N/mm²

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} = **2.000**

Modified allowable span to depth ratio — $Ratio_{allowcorner} = Ratio_{basiccorner} \times MF_{corner}$ = **14.000**

Actual span to depth ratio — $Ratio_{actualcorner} = l_{corner}/d_{edgetop}$ = **6.734**

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

Internal beam design checks

Basic loading

Hardcore — $w_{hcorethick} = \gamma_{hcore} \times h_{hcorethick}$ = **3.0** kN/m²

Internal beam self weight	w_{int} = 24 kN/m^3 × h_{int} × b_{int} = **4.9** kN/m
Internal beam load number 1	
Load type	**Longitudinal line load**
Dead load	w_{Dint1} = **38.0** kN/m
Live load	w_{Lint1} = **3.2** kN/m
Ultimate load	$w_{ultint1}$ = 1.4 × w_{Dint1} + 1.6 × w_{Lint1} = **58.3** kN/m
Longitudinal line load width	b_{int1} = **100** mm
Centroid of load from centreline of beam	x_{int1} = **50** mm

Internal beam bearing pressure check

Total uniform load at formation level	w_{udlint} = w_{Dudl}+w_{Ludl}+$w_{hcorethick}$+24kN/m^3×h_{int} = **17.8** kN/m^2
Sum of factored longitud'l and eff'tive line loads	ΣUDL$_{int}$ = **58.3** kN/m
Sum of unfactored longitud'l and eff'tive line loads	ΣUDLsls$_{int}$ = **41.2** kN/m

Centroid of loads from centreline of internal beam

Load x distance for internal load 1	Moment$_{int1}$ = $w_{ultint1}$ × x_{int1} = **2.9** kN
Sum of load x distances	ΣMoment$_{int}$ = **2.9** kN
Centroid of loads	x_{barint} = ΣMoment$_{int}$/ΣUDL$_{int}$ = **50.0** mm
Moment due to eccentricity to be resisted by slab	M_{eccint} = ΣUDL$_{int}$ × abs(x_{barint}) = **2.9** kNm/m

Assume moment due to eccentricity is resisted equally by top steel of slab on one side and bottom steel of slab on other

From slab bending check	
Moment due to depression under slab (hogging)	$M_{\Sigma e}$ = **5.0** kNm/m
Total moment to be resisted by slab top steel	$M_{slabtopint}$ = M_{eccint}/2 + $M_{\Sigma e}$ = **6.5** kNm/m
K factor	$K_{slabtopint}$ = $M_{slabtopint}$/(f_{cu} × $d_{tslabmin}^2$) = **0.012**
Lever arm	$z_{slabtopint}$ = $d_{tslabmin}$ × min(0.95, 0.5 + √(0.25 - $K_{slabtopint}$/0.9)) = **138** mm
Area of steel required	$A_{sslabtopintreq}$ = $M_{slabtopint}$/((1.0/γ_s) × f_{yslab} × $z_{slabtopint}$) = **109** mm^2/m

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

Mt to be resisted by slab btm stl due to load ecc'ty	$M_{slabbtmint}$ = M_{eccint}/2 = **1.5** kNm/m
K factor	$K_{slabbtmint}$ = $M_{slabbtmint}$/(f_{cu} × $d_{bslabmin}^2$) = **0.003**
Lever arm	$z_{slabbtmint}$ = $d_{bslabmin}$ × min(0.95, 0.5 + √(0.25-$K_{slabbtmint}$/0.9)) = **138** mm
Area of steel required in bottom	$A_{sslabbtmintreq}$ = $M_{slabbtmint}$/((1.0/γ_s) × f_{yslab} × $z_{slabbtmint}$) = **24** mm^2/m

PASS - $A_{sslabbtmintreq}$ <= $A_{sslabbtm}$ - Area of reinforcement in bottom of slab is adequate to transfer moment into slab

Bearing pressure

Initially check bearing pressure based on beam soffit/chamfer width only

Allowable bearing width	$b_{bearint}$ = b_{int} = **450** mm
Bearing pressure due to line/point loads	$q_{linepointint}$ = ΣUDLsls$_{int}$/$b_{bearint}$ = **91.6** kN/m^2
Total applied bearing pressure	q_{int} = $q_{linepointint}$ + w_{udlint} = **109.4** kN/m^2

q_{int} > q_{allow} - The slab is required to resist some of the beam loading

Now consider slab to resist some of beam load

Bearing width required	b_{reqint} = ΣUDLsls$_{int}$/(q_{allow} - w_{udlint}) = **5722** mm
Effective bearing width at u/s of beam/slab	$b_{reqeffint}$ = b_{reqint} - 2 × $h_{hcorethick}$ × tan(30) = **5549** mm
Resultant ult upward press acting on u/s of slab	q_{result} = (q_{allow}-w_{udlint}) × b_{reqint}/$b_{reqeffint}$ × ΣUDL$_{int}$/ΣUDLsls$_{int}$ = **10.5** kN/m^2
Slab cantilever moment	$M_{cantint}$ = q_{result} × [(b_{reqint} - b_{int})/2]^2/2 = **36.5** kNm/m
Total moment to be resisted by slab	$M_{slabint}$ = $M_{cantint}$ + M_{eccint}/2 = **38.0** kNm/m
K factor	$K_{slabint}$ = $M_{slabint}$/(f_{cu} × $d_{bslabmin}^2$) = **0.072**
Lever arm	$z_{slabint}$ = $d_{bslabmin}$ × min(0.95, 0.5 + √(0.25 - $K_{slabint}$/0.9)) = **132** mm
Area of steel required	$A_{sslabint}$ = $M_{slabint}$/((1.0/γ_s) × f_{yslab} × $z_{slabint}$) = **661** mm^2/m

PASS - $A_{sslabint}$ <= $A_{sslabbtm}$ - Area of reinforcement provided to distribute the beam load is adequate

The allowable bearing pressure under the internal beam will not be exceeded

Internal beam bending check

Divider for moments due to udl's — β_{udl} = **10.0**

Applied bending moments

Span of internal beam — $l_{int} = \phi_{depthick} + d_{inttop}$ = **3730** mm

Ultimate self weight udl — $w_{intult} = 1.4 \times w_{int}$ = **6.8** kN/m

Ultimate slab udl (approx) — $w_{intslab} = \max(0 \text{ kN/m}, 1.4 \times w_{slab} \times ((\phi_{depthick} \times 3/4) - b_{int}))$ = **13.9** kN/m

Self weight and slab bending moment — $M_{intsw} = (w_{intult} + w_{intslab}) \times l_{int}^2/\beta_{udl}$ = **28.7** kNm

Self weight shear force — $V_{intsw} = (w_{intult} + w_{intslab}) \times l_{int}/2$ = **38.5** kN

Moments due to applied uniformly distributed loads

Ultimate udl (approx) — $w_{intudl} = w_{udlult} \times \phi_{depthick} \times 3/4$ = **15.3** kN/m

Bending moment — $M_{intudl} = w_{intudl} \times l_{int}^2/\beta_{udl}$ = **21.3** kNm

Shear force — $V_{intudl} = w_{intudl} \times l_{int}/2$ = **28.6** kN

Moment and shear due to load number 1

Bending moment — $M_{int1} = w_{ultint1} \times l_{int}^2/\beta_{udl}$ = **81.1** kNm

Shear force — $V_{int1} = w_{ultint1} \times l_{int}/2$ = **108.8** kN

Resultant moments and shears

Total moment (hogging and sagging) — $M_{\Sigma int}$ = **131.2** kNm

Maximum shear force — $V_{\Sigma int}$ = **175.9** kN

Reinforcement required in top

Width of section in compression zone — $b_{inttop} = b_{int}$ = **450** mm

Average web width — $b_{wint} = b_{int}$ = **450** mm

K factor — $K_{inttop} = M_{\Sigma int}/(f_{cu} \times b_{inttop} \times d_{inttop}^2)$ = **0.081**

Lever arm — $z_{inttop} = d_{inttop} \times \min(0.95, 0.5 + \sqrt{(0.25 - K_{inttop}/0.9)})$ = **342** mm

Area of steel required for bending — $A_{sinttopbend} = M_{\Sigma int}/((1.0/\gamma_s) \times f_y \times z_{inttop})$ = **882** mm²

Minimum area of steel — $A_{sinttopmin} = 0.0013 \times b_{wint} \times h_{int}$ = **263** mm²

Area of steel required — $A_{sinttopreq} = \max(A_{sinttopbend}, A_{sinttopmin})$ = **882** mm²

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 — $b_{intbtm} = b_{int} + 0.2 \times l_{int}$ = **1196** mm

K factor — $K_{intbtm} = M_{\Sigma int}/(f_{cu} \times b_{intbtm} \times d_{intbtm}^2)$ = **0.029**

Lever arm — $z_{intbtm} = d_{intbtm} \times \min(0.95, 0.5 + \sqrt{(0.25 - K_{intbtm}/0.9)})$ = **371** mm

Area of steel required for bending — $A_{sintbtmbend} = M_{\Sigma int}/((1.0/\gamma_s) \times f_y \times z_{intbtm})$ = **815** mm²

Minimum area of steel required — $A_{sintbtmmin} = 0.0018 \times 1.0 \times b_{wint} \times h_{int}$ = **365** mm²

Area of steel required — $A_{sintbtmreq} = \max(A_{sintbtmbend}, A_{sintbtmmin})$ = **815** mm²

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

Internal beam shear check

Applied shear stress — $v_{int} = V_{\Sigma int}/(b_{wint} \times d_{inttop})$ = **1.029** N/mm²

Tension steel ratio — $\rho_{int} = 100 \times A_{sinttop}/(b_{wint} \times d_{inttop})$ = **0.551**

From BS8110-1:1997 - Table 3.8

Design concrete shear strength — v_{cint} = **0.518** N/mm²

v_{int} > v_{cint} + 0.4N/mm² - Therefore designed links required

Link area to spacing ratio required — $A_{sv}\_upon\_s_{vreqint} = (v_{int} - v_{cint}) \times b_{wint}/((1.0/\gamma_s) \times f_{ys})$ = **0.528** mm

Link area to spacing ratio provided $A_{sv}\_upon\_s_{vprovint} = N_{intlink} \times \pi \times \phi_{intlink}^2/(4 \times s_{vint})$ = **0.785** mm

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