

Structural Engineers report for a Proposed Concrete Raft Foundation

The Paddocks
Duffryn Road
Neath Abbey
SA10 7BQ

Document Control

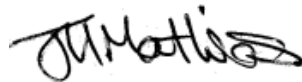
Project	Proposed PV installation to the existing roof at 2-16 Canaston Place, Cardiff CF5 5LN
Client	Mr & Mrs A S Follant
Vale Consultancy Ref:	23324

Document Checking:

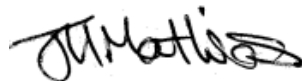
Prepared By: Richard Dean BSc (Hons) CEng
 MIStructE FGS
 Structural Engineer

Signed: 

Checked By: Jon Mathias BEng (Hons)
 Director

Signed: 

Verified By: Jon Mathias BEng (Hons)
 Director

Signed: 

Issue	Date	Status
1	12 December, 2025	First issue

Our Ref: 23324/RD/JM

12 December, 2025

Mr & Mrs A S Follant

Issued by Email to Mr & Mrs Follant: maipaddock@gmail.com

23324; Neath Abbey – Proposed Concrete Raft Foundation

Thank you for your instruction to attend the above site and to review the provided information in order to provide an structural engineering design solution to satisfy The Regulators and to comply with the requirements of both The Coal Authority and current British Standards and Design Codes.

We confirm that we undertook a visual inspection of the site with yourselves on Wednesday, 19 November, 2025 when the weather was cloudy and raining.

This report shall be for the confidential use of our client, and the report shall not be reproduced in whole or in part, or relied upon by a third party without the express written authorisation of Vale Consultancy.

1.0 Introduction

The Client has undertaken to partial demolition of the existing property with the intention to re-use the existing raft foundation to support the new loadings from a modern timber -framed house.

The current site is reflected by the photograph included as Appendix A

2.0 Planning

The requirements of the Planning Authority were conditioned upon the receipt and acceptance of a Coal Mining investigation and interpretive report to inform and to provide recommendations regarding the existing/proposed raft foundations at the site.

A Geo-technical report was produced by Consultants Spencer Quantum and is included as Appendix B.

The conclusions and recommendations of this report were that the raft foundation has to be capable of spanning an internal 3.0m diameter void or be able to cantilever 1.5m at a corner.

3.0 Structural Investigation and Analysis

The structural composition of the existing concrete raft foundation was investigated by removal of the concrete at a corner.

The raft foundation which was originally constructed in the 1930's and was found to be of the

order of 450mm thick and re-inforced with 100mm thick 'I' beams at 900mm lateral centres. No transverse reinforcement was proven.

Upon this basis, it is proposed to construct a new 150mm thick topping slab over the existing raft slab, suitably re-inforced with H12 bars at 125mm centres as required by the detail design calculations.

4.0 Conclusions and Recommendations

The structural calculations for the design of the new raft foundation are included as Appendix D/ pages 1.0 -3.4.

The detailed loading calculations were provided by the proposed timber frame manufacturer at page 2.0.

The proposed raft slab construction is detailed upon drawing SK01 included as Appendix E.

If you have any queries with the contents of this report, please do not hesitate to contact me.

Yours sincerely,



Richard Dean BSc (Hons) CEng MStructE FGS
Structural Engineer at Vale Consultancy

Appendix A: Photographs

Figure 1: Existing raft slab



Appendix B: Spencer Quantum Geotechnical Report Ref ; Q1813-TN1 dated 22 October, 2025

 SPENCER QUANTUM			
The Paddock – Additional Mining Related Information Ref: Q1813-TN1 October 2025			
Project:	The Paddock, DYFFRYN, NEATH, SA10 7BQ		
Document	Additional Information in Relation to Mining Constraints P2025/0689		
Client:	Andrew Follant and Sam Toone		
Ref.:	Q1813-TN1	Date:	22 nd October, 2025

Background

As part of the Planning Process to seek approval for the development at the site at The Paddock, Dyffryn, Neath, the Coal Authority / Mining Remediation Authority, as a consultee, has expressed concerns over the risk to the development from the effects of shallow mineworkings. Our Client requires additional supporting information in relation to the risk to the site from shallow mine workings to the building foundations.

This Technical Note presents a discussion of these potential constraints in order to provide supporting information to demonstrate that the mitigation proposed is suitable and adequate.

This additional technical note should be read in conjunction with Spencer Quantum's previous reports on the site, namely:

1. Q1813/GIR THE PADDOCK, DYFFRYN, NEATH, SA10 7BQ Phase II: Ground Investigation Interpretative Report Report No. Q1813/GIR September 2025
2. BC/MB/04.22.02 Coal Mining Risk Assessment: Land at The Paddocks, Bryn Coch, Neath, Bladford Consulting Ltd, April 2022

Shallow Mine Workings

The borehole logs and site plan presented in Report Q1813/GIR are included in Appendix I.

Evidence of solid coal and workings believed to be associated with the Mynyddislwyn Big Rider coal seam has been proven in the seam encountered in all three boreholes. Soft drill, interpreted to be collapsed mine workings / ground was encountered from 21.5m to 24.0m and from 21.0 to 23.0m in Boreholes BH01 and BH03 respectively, whilst coal was encountered between 22.0 and 23.0mbgl in Borehole BH02. No open voids have been proven.

Within Borehole BH01 10m of Superficial deposits were proven, with 11.5m of solid roof cover proven above the collapsed mine workings / ground. Within Borehole BH03 9.5m of Superficial deposits were proven, again with 11.5m of solid roof cover proven above the collapsed mine workings / ground.

The worst-case scenario with the shallow workings identified in BH01 with 2.5m thick collapsed mine workings / ground with 11.5m solid roof cover of Mudstone with Sandstone bands.

The risk to the development is defined by the likelihood of the workings present migrating towards the surface to an extent that affects the dwellings. The migration of any collapsed workings, or voids if present, is dependent upon the actual depth of any workings, the thickness of the workings, the nature of any infilling and the quality/strength of the overlying solid strata.

A relationship which defines the maximum height to which a void can migrate above workings was developed by Price, Malkin and Knill¹ (over page).

This is given by: $H = h / (B - 1)$ where H = Height to which a void can migrate (m)

¹ Price, D, Malkin, A, & Knill, J, 1969, Foundations of multi-storey blocks on the Coal Measures with special reference to old mine workings. QJEG Vol.1. 1969



h = Thickness of seam (m)
B = Measurement of the bulking effect of the rock.

The ratio B will vary according to the rock type present. For sandstone it may be 1.5, whereas for a mudstone it would be 1.2. Using this method of calculation, for the shallowest workings / thickest workings / collapsed ground, 1.98m extraction thickness at a depth of 11.5m, conservatively assuming a roof comprising only of Mudstone, then theoretical collapse of any roof material into an open void might migrate some 9.9m before arresting.

Healy and Head² presented data that the extent to which a void may migrate from the roof of any workings is governed by the structure and form of deformation and disintegration of the roof material. The geometry of collapse, they proposed, could be due to *rectangular* collapse, *wedge* collapse or *conical* collapse. The maximum height of potential collapse is then related to the bulking factor, as discussed, for the three differing modes of failure. From the data presented in CIRIA SP32 for the lowest bulking factor and the worst-case failure type (Conical), the height to which a void may migrate may reach 10 times the seam or workings thickness. At the maximum thickness of workings found of 2.5m, then collapse may migrate 25m above such a void before becoming arrested. This would suggest migration of voids reaching the surface.

The revision of CIRIA SP32, CIRIA C758³, does refer to the 10 times roof cover 'rule of thumb' but clearly qualifies this as being the exceptional case, stating that 2 times to 5 times rock cover is a better figure to adopt. Taking the higher bound figure then workings could theoretically migrate c.9.9m above a collapse, based on the recorded extraction thickness of 1.98m.

C758 also refers to the work undertaken by Statham & Scott⁴, that in their study specifically of numerous South Wales drill and grout contracts quantified that there was no 'significant tendency for a migration of more than 2 times' the coal/workings thickness. They found that migration of voids to any significant distance above the level of the original workings was hardly ever encountered. Furthermore, they noted that subsidence incidents in South Wales are rare and most relate to mine entries or outcrop workings and their effects are generally limited to a small area. Adopting a figure of 10 times is therefore overly conservative in the South Wales Coalfield.

It is important to note that the boreholes have proved the collapsed ground has only extended to a thickness of 2.5m in BH1, in comparison to a recorded extraction height of 1.98m and a proven coal seam thickness of 1m in BH2. This mirrors the findings of Statham & Scott. This provides strong evidence that any collapse has arrested. No broken or fractured ground was noted in the boreholes in the roof cover that could allow further propagation of any void migration. In addition to no voids have been proved.

It is also apparent that the rule of thumb presented in CIRIA C758 of migration of voids to 10-times the seam thickness is extremely conservative when reviewing the evidence of this site. The two-times cover suggested by Statham and Scott, from study of drill & grout schemes in South Wales, is in fact conservative in comparison to the evidence on this site.

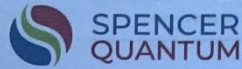
Taking an overall risk-based approach to the theory and findings of the site under review, we consider that the following are reasonable conclusions to draw.

- Given the length of time since any workings may have taken place (last worked in 1936) it is considered likely that ground movements associated with such workings will have ceased.
- No voids have been proved; only solid coal or collapsed ground.
- While the most conservative theory of the ten-times cover rule of thumb does indicate that any voids may migrate to the surface, the evidence in the boreholes contradicts this with any collapse being arrested less than one seam thickness above the workings.

² Healy & Head, 1982, Construction Over Abandoned Mineworkings, CIRIA SP32.

³ Parry & Chiverrell (Eds.), 2020, Abandoned mine workings manual, CIRIA C758D.

⁴ Statham & Scott, 1992, An Analysis of Drill and Grout Contracts in the South Wales Coalfield, Proc. Grouting in the Ground Conference, ICE.



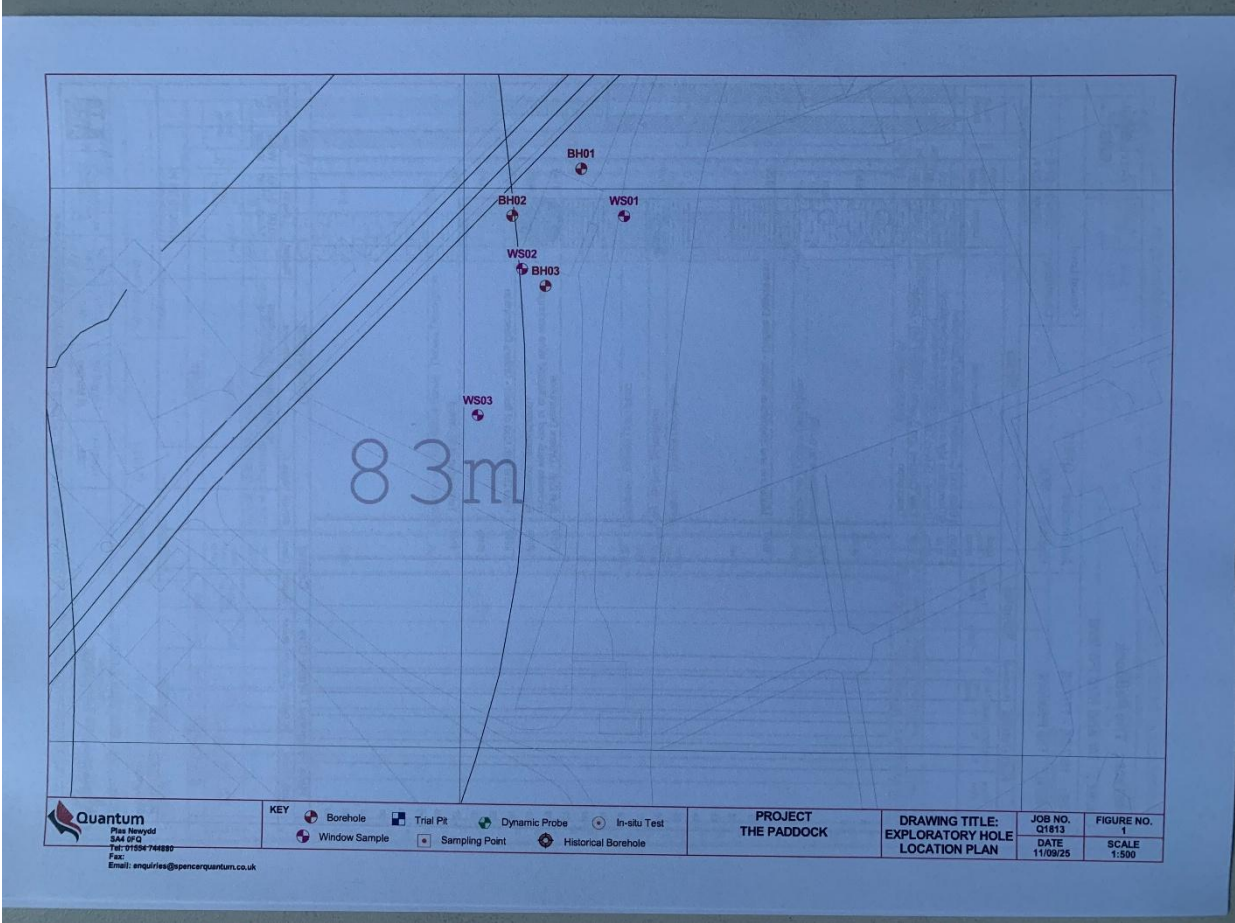
The Paddock – Additional Mining Related Information
Ref: Q1813-TN1
October 2025

- By Price, Malkin & Knill, the thickness of the mixed Sandstone/Mudstone roof cover above the coal seam and workings, indicates that there is at worst only a marginal risk of development of collapse to an extent that will affect the surface of the site.

On the balance of this evidence, the use of fully reinforced foundations for the structures, designed to span a 3m void, therefore provides adequate mitigation of the risk.

For and on behalf of
SPENCER QUANTUM LTD

P. Darby
Principal / Chartered Engineering Geologist





Contract : The Paddock												Borehole No. BH01									
Client : Mr and Mrs Pollard																					
Dates : 16/7/25 - 17/7/25						Job Number : Q1813				Ground Level :											
Location : The Paddock						Engineer : SQL				Coordinates: 273720.97 E 199903.52 N <i>Coordinates to National Grid</i>											
S m B.G.L.	Run Details				Samples		Strata				Depth (Thick- ness)	Water	Install/ Backfill								
	Core Run	TCR %	SCR %	ROD %	Fractures Min Avg Max	FI	Depth	Type No. Blows	Description	Legend											
1									(0.20)	Brick and Concrete Fill - Drillers Description	(0.20)										
2									0.20	(Demolition Material - Engineers Interpretation)	0.20										
3									(0.30)	Topsoil - Drillers Description	(0.30)										
4									0.50	Clay, Cobbles and Boulders (Glacial Till) - Driller Description	0.50										
5									(9.50)		(9.50)										
6																					
7																					
8																					
9																					
10									10.00	Mudstone and Sandstone Bands - Drillers Description	10.00										
11																					
12																					
13																					
14																					
15									(11.50)		(11.50)										
16																					
17																					
18																					
19																					
20																					
21									21.50	Soft Drill - Drillers Description	21.50										
22									(2.50)	(Possible weak coal or collapsed mine workings - Engineers Interpretation)	(2.50)										
23																					
24									24.00	Hard Drill with loss of flush - Drillers Description	24.00										
25																					
26									(3.00)		(3.00)										
27									27.00	Terminated at 27.0mbgl	27.00										
28																					
29																					
30																					
31																					
32																					
33																					
34																					
Hole Progress / Water Obs														Casing		Groundwater				Flush	
Date / Time		H. Depth / C. Depth		Water	Depth	Diam.	Struck	Rose To	Behaviour		Sealed	Depth	Type	Returns							
									No Groundwater Encountered			0.00 - 24.00 24.00 - 27.00	Water Water	100 0							
Plant: Commachio 450																					
Remarks:																					
		Plus Henry Ltd 54487Q Tel: 01554 748839 email: enquiries@quantumgeosols.co.uk						Operator: SQL		Logged By: R Morgan		Sheet No. 1 Of 1		m Per Page 35		All measurements in metres unless otherwise stated					



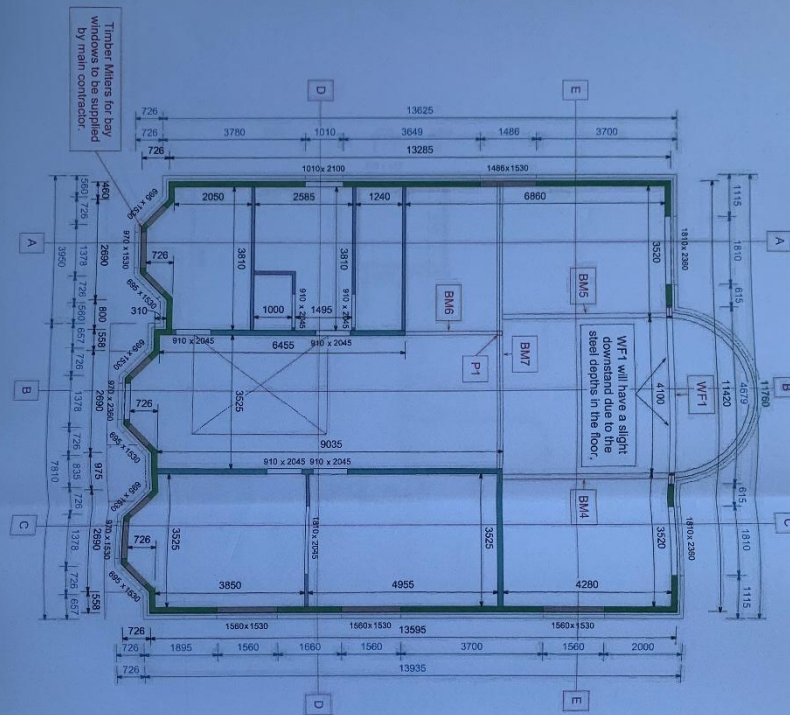
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Location : The Paddock					Engineer : SQL					Coordinates: 273708.66 E 199895.26 N Co-ordinates to National Grid			
m B.O.L.	Run Details				Fractures	Samples		Strata					
	Core Run	TCR %	SCR %	RCD %	FI	Depth	Type No. Blows	Depth (Thickness)	Description	Legend	Depth (Thickness)	Water	Install/ Backfill
0								(0.40)	Topsoil - Drillers Description		(0.40)		
1								0.40	Clay and Boulders (Glacial Till) - Drillers Description		0.40		
2													
3													
4													
5													
6								(9.60)			(9.60)		
7													
8													
9													
10								10.00	Mudstone and Sandstone Bands - Drillers Description		10.00		
11													
12													
13													
14													
15								(10.50)			(10.50)		
16													
17													
18													
19													
20								20.50	Mudstone - Drillers Description		20.50		
21								(1.50)			(1.50)		
22								22.00	Coal - Drillers Description		22.00		
23								(1.00)			(1.00)		
24								23.00	Mudstone - Drillers Description		23.00		
25													
26								(6.75)			(6.75)		
27													
28													
29													
30								29.75	Sandstone - Drillers Description		29.75		
31								(0.25)			(0.25)		
32								30.00	Terminated at 30.0mbgl		30.00		
33													
34													
Hole Progress / Water Obs													
Casing													
Groundwater													
Flush													
Date / Time	H. Depth	C. Depth	Water	Depth	Diam.	Struck	Rose To	Behaviour	Sealed	Depth	Type	Returns	
								No Groundwater Encountered		0.00 - 30.00	Water	100	
Plant: Commachio 450													
Remarks:													
Quantum Geotata Plot Newydd S44 8EQ Tel: 01554 744880 email: enquiries@spencerquantum.co.uk													
Operator: SQL Logged By: P Darby Sheet No. 1 Of 1 m Per Page 35 All measurements in metres unless otherwise stated													
AGS													



Contract : The Paddock										Borehole No. BH03																	
Client : Mr and Mrs Pollard					Job Number : Q1813					Ground Level :																	
Dates : 15/7/25 - 16/7/25					Engineer : SQL					Coordinates: 273714.64 E 199882.94 N <i>(Co-ordinates to National Grid)</i>																	
Location : The Paddock																											
m B.G.L.	Run Details				Fracture	Samples		Strata			Depth (Thickness)	Water	Install/Backfill														
	Core Run	TCR %	SCR %	RQD %	Min Avg Max	FI	Depth	Type No. Blows	Description	Legend																	
1									(0.40) Topsoil - Drillers Description		(0.40)																
2									0.40 Clay and Boulders (Glacial Till) - Drillers Description		0.40																
3																											
4																											
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Hole Progress / Water Obs														Casing		Groundwater				Flush							
Date / Time		H. Depth		C. Depth		Water		Depth		Diam.		Struck		Rose To		Behaviour		Sealed		Depth		Type		Returns			
																No Groundwater Encountered				0.00 - 21.00		Water		100			
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Plant: Commachio 450																											
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Appendix C

Proposed Architectural Drawings



No.	Proj. Number	Issued For
Description		
Date		

THIS DRAWING IS A PRELIMINARY DESIGN. THE MAIN CONSTRUCTION WORK SHOULD BE CARRIED OUT BY PERSONS WITH APPROPRIATE KNOWLEDGE AND EXPERIENCE. EXHAUSTIVE DETAILS IS NOT REQUIRED.

ADDITIONAL STUDIES CONCERNING WIND NEED TO BE DONE UNDER SPECIAL CIRCUMSTANCES.

(STUDY USED TO CONTINUE TO BE REQUIRED)

WORK DESIGN WORK MAY BE REQUIRED WHEN EFFECTIVE DUE TO THE NATURE OF WORKING WITH WOOD.

TEMPORARY BUILDINGS MUST REMAIN IN PLACE UNTIL INTERNAL ROOFING IS COMPLETED.

THIS DRAWING IS TO BE USED IN CONNECTION WITH RELEVANT ASPECTS OF THE CONSTRUCTION DESIGN.

THIS INFORMATION IS PROVIDED IN GOOD FAITH AND WITHOUT LIABILITY. THE USER ASSUMES FULL RESPONSIBILITY TO ENSURE THE STABILITY OF THE OVERALL STRUCTURE.

Timber Frame Manufacturers

Tel: 01646 600 722

Customer: Andrew Follant

GF Panels

Job no: FH3482

Sheet 3

Designer: Ethan Morgan

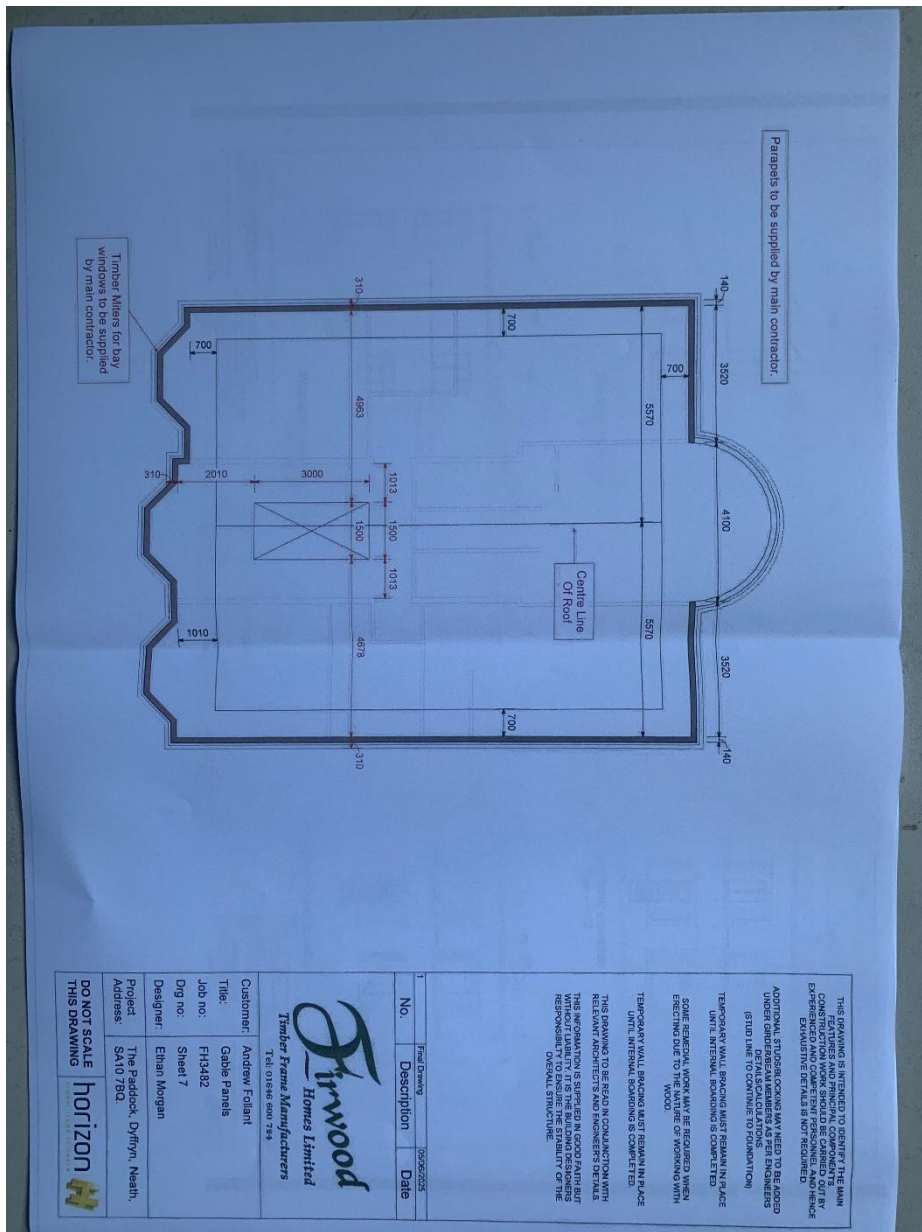
Project	The Paddock
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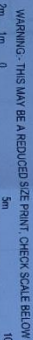
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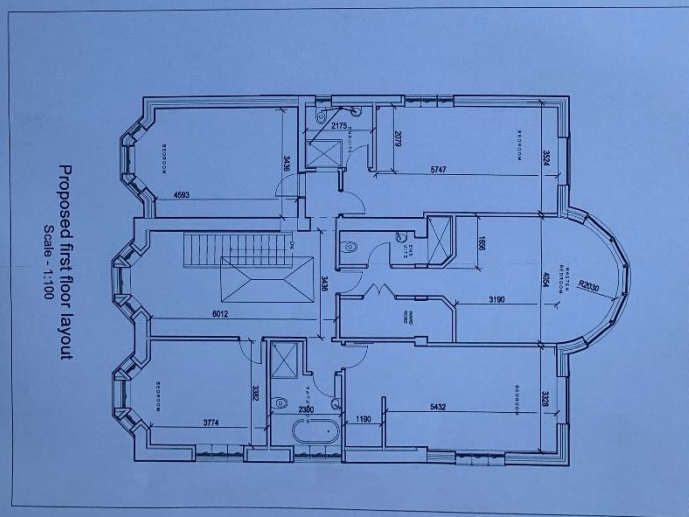
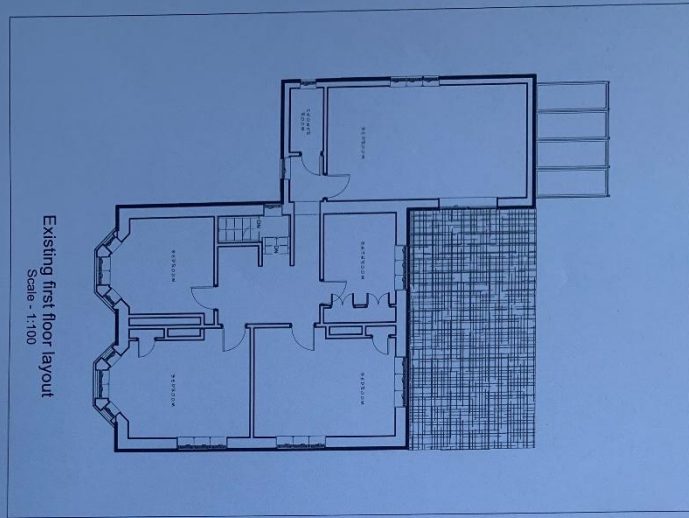
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Appendix D

Vale Consultancy Structural Calculations 23324 / 1.0, 2.0, 3.0 – 3.4

Vale Consultancy
CONSULTING CIVIL & STRUCTURAL ENGINEERS

Project No. 23324
Sheet No. 1.0

Prepared by R.D.
Checked by

GENERAL LOADINGS

NEW 150mm RAFT SLAB

	KN/m^2
G_u Slab = 0.15×24.0	= 3.60
Q_u Residential	= 1.50

EXTERNAL BRICK LEAF

G_u Brickwork = 0.10×21.0	= 2.10
--------------------------------------	--------

Proposed Loadings

	G_u	Q_u
INTERNAL - worst case	15	13
Slab	$\frac{3.6}{18.6}$	$\frac{1.5}{14.5}$
	<u>18.6</u>	<u>14.5</u>

- page 2.0
= w_i

EXTERNAL - worst case

Brick leaf = 2.1×6.0	$\frac{12.6}{21.6}$	$\frac{6.0}{6.0}$
	<u>21.6</u>	<u>6.0</u>

- page 2.0
= w_e

For Design of Reinforced concrete retaining wall - raft must be able to span 3.0m void between and 1.5m corner cantilever at:

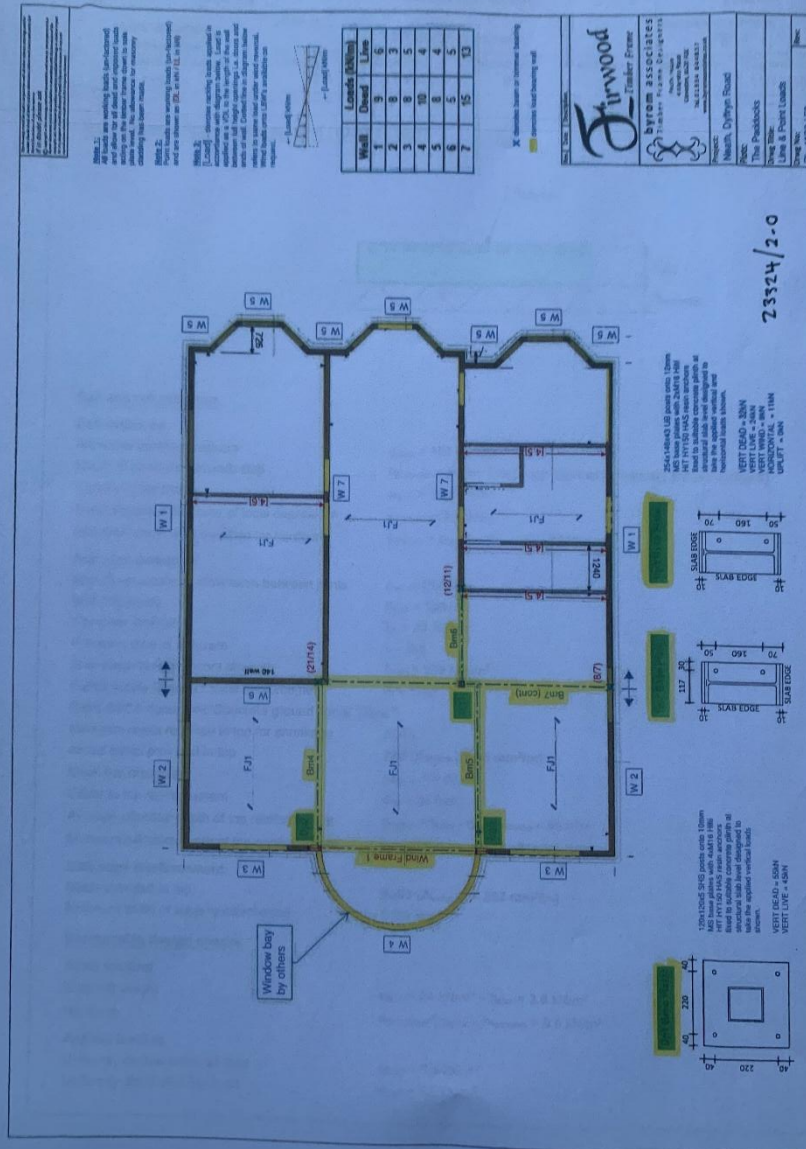
w_i

3.0m

w_e

1.5m

Refer to pages 3.1 - 3.4

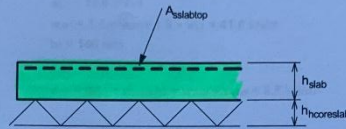




 Vale Consultancy 29 Bocam Park Old Field Road, Pencoeed CF35 5LJ	Project The Paddock, Duffryn		Job no. 23324	
	Calcs for Flat Raft Foundation		Start page no./Revision 3/1	
	Calcs by LM	Calcs date 11/12/2025	Checked by RD	Checked date 11/12/2025
			Approved by RD	Approved date 11/12/2025

RAFT FOUNDATION DESIGN (BS8110 : PART 1 : 1997)

Tedds calculation version 1.0.13



Soil and raft definition

Soil definition

Allowable bearing pressure	$q_{allow} = 100.0 \text{ kN/m}^2$
Depth of hardcore beneath slab	$h_{core\text{slab}} = 0 \text{ mm}$ (Dispersion allowed for bearing pressure check)
Density of hardcore	$\gamma_{hcore} = 20.0 \text{ kN/m}^3$
Basic assumed diameter of local depression	$\phi_{depbasic} = 3000 \text{ mm}$
Diameter under slab modified for hardcore	$\phi_{depslab} = \phi_{depbasic} - h_{core\text{slab}} = 3000 \text{ mm}$

Raft slab definition

Max dimension/max dimension between joints	$l_{max} = 10.000 \text{ m}$
Slab thickness	$h_{slab} = 150 \text{ mm}$
Concrete strength	$f_{cu} = 35 \text{ N/mm}^2$
Poissons ratio of concrete	$\nu = 0.2$
Slab mesh reinforcement strength	$f_{y\text{slab}} = 500 \text{ N/mm}^2$
Partial safety factor for steel reinforcement	$\gamma_s = 1.15$
From C&CA document 'Concrete ground floors' Table 5	A142
Minimum mesh required in top for shrinkage	A98 ($A_{s\text{slabtop}} = 98 \text{ mm}^2/\text{m}$)
Actual mesh provided in top	$\phi_{slabtop} = 5 \text{ mm}$
Mesh bar diameter	$c_{top} = 50 \text{ mm}$
Cover to top reinforcement	$d_{slabav} = h_{slab} - c_{top} - \phi_{slabtop} = 95 \text{ mm}$
Average effective depth of top reinforcement	$d_{slabmin} = d_{slabav} - \phi_{slabtop}/2 = 92 \text{ mm}$
Minimum effective depth of top reinforcement	

Slab edge reinforcement

Mesh provided in top	A393 ($A_{s\text{edgetop}} = 393 \text{ mm}^2/\text{m}$)
Minimum width of edge reinforcement	2169 mm

Internal slab design checks

Basic loading

Slab self weight	$W_{slab} = 24 \text{ kN/m}^3 \times h_{slab} = 3.6 \text{ kN/m}^2$
Hardcore	$W_{hcore\text{slab}} = \gamma_{hcore} \times h_{hcore\text{slab}} = 0.0 \text{ kN/m}^2$

Applied loading

Uniformly distributed dead load	$W_{Dudl} = 3.6 \text{ kN/m}^2$
Uniformly distributed live load	$W_{Ludl} = 1.5 \text{ kN/m}^2$



 Vale Consultancy 29 Bocam Park Old Field Road, Pencoed CF35 5LJ	Project The Paddock, Duffryn				Job no. 23324	
	Calcs for Flat Raft Foundation				Start page no./Revision 3/2	
	Calcs by LM	Calcs date 11/12/2025	Checked by RD	Checked date 11/12/2025	Approved by RD	Approved date 11/12/2025

Slab load number 1	
Load type	Line load
Dead load	$W_{D1} = 15.0 \text{ kN/m}$
Live load	$W_{L1} = 13.0 \text{ kN/m}$
Ultimate load	$W_{ult1} = 1.4 \times W_{D1} + 1.6 \times W_{L1} = 41.8 \text{ kN/m}$
Line load width	$b_1 = 140 \text{ mm}$
Internal slab bearing pressure check	
Total uniform load at formation level	$W_{ult} = W_{slab} + W_{holedslab} + W_{ud1} + W_{Lud} = 8.7 \text{ kN/m}^2$
Bearing pressure beneath load number 1	
Effective loaded width	$X_1 = b_1 + 2 \times h_{slab} = 440 \text{ mm}$
Bearing pressure at formation level	$q_1 = (W_{D1} + W_{L1})/X_1 + W_{ult} = 72.3 \text{ kN/m}^2$
	PASS - $q \leq q_{allow}$ - Applied bearing pressure is less than allowable
Internal slab bending and shear check	
Applied bending moments	
Effective span of slab	$l_{slab} = (\phi_{deplslab} + d_{slabtop})/2 = 1548 \text{ mm}$
Ultimate self weight udl	$W_{swult} = 1.4 \times W_{slab} = 5.0 \text{ kN/m}^2$
Approximate self weight cantilever moment at edge	$M_{sw} = (W_{swult} \times \pi \times l_{slab}^2) \times (l_{slab}/3) / (2 \times \pi \times l_{slab}) = 2.0 \text{ kNm/m}$
Self weight shear force at edge	$V_{sw} = W_{swult} \times l_{slab} / 2 = 3.9 \text{ kN/m}$
Moments due to applied uniformly distributed loads	
Ultimate applied udl	$W_{udult} = 1.4 \times W_{ud1} + 1.6 \times W_{Lud} = 7.4 \text{ kN/m}^2$
Approximate cantilever moment at edge	$M_{ud} = (W_{udult} \times \pi \times l_{slab}^2) \times (l_{slab}/3) / (2 \times \pi \times l_{slab}) = 3.0 \text{ kNm/m}$
Shear force at edge	$V_{ud} = W_{udult} \times l_{slab} / 2 = 5.8 \text{ kN/m}$
Moment due to load number 1	
Approximate equivalent udl	$W_{ud1} = 1.5 \times W_{D1} / (2 \times 0.3 \times l_{slab}) = 67.5 \text{ kN/m}^2$
Approximate cantilever moment at edge	$M_{D1} = (W_{ud1} \times \pi \times l_{slab}^2) \times (l_{slab}/3) / (2 \times \pi \times l_{slab}) = 27.0 \text{ kNm/m}$
Shear force at edge	$V_1 = W_{ud1} \times l_{slab} / 2 = 52.3 \text{ kN/m}$
Resultant moments and shears	
Total moment at edge	$M_{ca} = 31.9 \text{ kNm/m}$
Total shear force	$V_c = 61.9 \text{ kN/m}$
Reinforcement required in top	
K factor	$K_{slabtop} = M_{ca} / (f_{cu} \times d_{slabtop}^2) = 0.101$
Lever arm	$Z_{slabtop} = d_{slabtop} \times \min(0.95, 0.5 + \sqrt{(0.25 - K_{slabtop}/0.9)}) = 82.7 \text{ mm}$
Area of steel required for bending	$A_{slabtopbend} = M_{ca} / ((1.0/f_y) \times f_{y,slab} \times Z_{slabtop}) = 888 \text{ mm}^2/\text{m}$
Minimum area of steel required	$A_{slabmin} = 0.0013 \times h_{slab} = 195 \text{ mm}^2/\text{m}$
Area of steel required	$A_{slabtopreq} = \max(A_{slabtopbend}, A_{slabmin}) = 888 \text{ mm}^2/\text{m}$
FAIL - $A_{slabtopreq} > A_{slabtop}$ - Area of reinforcement provided in top to span local depressions is inadequate	
Shear check	
Applied shear stress	$v = V_c / d_{slabmin} = 0.669 \text{ N/mm}^2$
Tension steel ratio	$\rho = 100 \times A_{slabtop} / d_{slabmin} = 0.106$
From BS8110-1:1997 - Table 3.8	
Design concrete shear strength	$v_c = 0.482 \text{ N/mm}^2$
	FAIL - $v > v_c$ - Shear capacity of slab is exceeded
Internal slab deflection check	
Basic allowable span to depth ratio	$Ratio_{basic} = 7.0$

PRVIDE 712 bars @ 125mm spacing (905mm²/m) OR 81131 mesh (1151mm²/m)



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Moment factor	$M_{factor} = M_{Ed}/d_{slab}^2 = 3.538 \text{ N/mm}^2$
Steel service stress	$f_s = 2/3 \times f_{yk,slab} \times A_{s,edgetop/bend}/A_{s,edgetop} = 3018.997 \text{ N/mm}^2$
Modification factor	$MF_{slab} = \min(2.0, 0.55 + [(477 \text{ N/mm}^2 - f_s)/(120 \times (0.9 \text{ N/mm}^2 + M_{factor}))])$
	$MF_{slab} = -4.223$
Modified allowable span to depth ratio	$Ratio_{allow} = Ratio_{basic} \times MF_{slab} = -29.560$
Actual span to depth ratio	$Ratio_{actual} = l_{slab}/d_{slab} = 16.289$
	FAIL - $Ratio_{actual} > Ratio_{allow}$ - Slab span to depth ratio is inadequate

Slab edge design checks

Basic loading

Hardcore $W_{hcoreslab} = \gamma_{hcore} \times h_{hcoreslab} = 0.0 \text{ kN/m}^2$

Slab self weight $W_{slab} = 24 \text{ kN/m}^3 \times h_{slab} = 3.6 \text{ kN/m}^2$

Slab edge bearing pressure check

Total uniform load at formation level $W_{tiedge} = W_{Dud} + W_{Lud} + W_{slab} + W_{hcoreslab} = 8.7 \text{ kN/m}^2$

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

Slab edge bending check

Considering a 1.0m width of slab

Applied bending moments

Cantilever span of slab $l_{edge} = (d_{slab} + d_{slabmin})/2 = 1546 \text{ mm}$

Ultimate self weight udl $W_{edgeudl} = 1.4 \times W_{slab} = 5.0 \text{ kN/m}^2$

Self weight bending moment $M_{edgeudl} = W_{edgeudl} \times l_{edge}^2/2 = 6.0 \text{ kNm/m}$

Self weight shear force $V_{edgeudl} = W_{edgeudl} \times l_{edge} = 7.8 \text{ kN/m}$

Moments due to applied uniformly distributed loads

Ultimate udl $W_{edgeudl} = W_{tiedge} = 7.4 \text{ kN/m}^2$

Bending moment $M_{edgeudl} = W_{edgeudl} \times l_{edge}^2/2 = 8.9 \text{ kNm/m}$

Shear force $V_{edgeudl} = W_{edgeudl} \times l_{edge} = 11.5 \text{ kN/m}$

Resultant moments and shears

Total moment (hogging and sagging) $M_{ledge} = 14.9 \text{ kNm/m}$

Maximum shear force $V_{ledge} = 19.3 \text{ kN/m}$

Reinforcement required in top

K factor $K_{edgetop} = M_{ledge}/(f_{cu} \times d_{slabmin}^2) = 0.050$

Lever arm $Z_{edgetop} = d_{slabmin} \times \min(0.95, 0.5 + \sqrt{(0.25 - K_{edgetop}/0.9)}) = 87 \text{ mm}$

Area of steel required for bending $A_{s,edgetop/bend} = M_{ledge}/((1.0/y_s) \times f_{yk,slab} \times Z_{edgetop}) = 394 \text{ mm}^2/\text{m}$

Area of steel required $A_{s,edgetopreq} = \max(A_{s,edgetop/bend}, A_{s,slabmin}) = 394 \text{ mm}^2/\text{m}$

FAIL - $A_{s,edgetopreq} > A_{s,edgetop}$ - Area of reinforcement provided in top of slab is inadequate

Applied shear stress $V_{edge} = V_{ledge} \times 1.0\text{m}/(1000\text{mm} \times d_{slabmin}) = 0.209 \text{ N/mm}^2$

Tension steel ratio $\rho_{edge} = 100 \times A_{s,edgetop} \times 1.0\text{m}/(1000\text{mm} \times d_{slabmin}) = 0.425$

From BS8110-1:1997 - Table 3.8

Design concrete shear strength $V_{c,edge} = 0.766 \text{ N/mm}^2$

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

Slab edge deflection check

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

Moment factor $M_{factoredge} = M_{ledge}/d_{slabmin}^2 = 1.744 \text{ N/mm}^2$

Steel service stress $f_{s,edge} = 2/3 \times f_{yk,slab} \times A_{s,edgetop/bend}/A_{s,edgetop} = 334.303 \text{ N/mm}^2$

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



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				Approved by	RD	Approved date		11/12/2025

Modified allowable span to depth ratio
Actual span to depth ratio

MF_{edge} = 1.000
Ratio_{allowedge} = Ratio_{basicedge} × MF_{edge} = 6.999
Ratio_{actualledge} = l_{edge}/d_{slabmin} = 16.716
FAIL - Ratio_{actualledge} > Ratio_{allowedge} - Slab span to depth ratio is inadequate

Corner design checks

Basic loading

Corner bearing pressure check
Total uniform load at formation level

W_{udcorner} = W_{udsl} + W_{Ludl} + W_{slab} + W_{cornerstab} = 8.7 kN/m²
PASS - W_{udcorner} ≤ q_{allow} - Applied bearing pressure is less than allowable

Slab corner bending check
Cantilever span of slab at corner

l_{corner} = φ_{depslab}/√(2) + d_{slabav}/2 = 2169 mm

Moment and shear due to self weight
Considering triangular loading
Maximum ultimate self weight udl
Self weight bending moment
Self weight shear force

W_{swult} = 1.4 × W_{slab} × φ_{depslab}/√(2) = 10.7 kN/m
M_{corneruw} = W_{swult} × l_{corner}²/(6 × φ_{depslab}/√(2)) = 4.0 kNm/m
V_{corneruw} = W_{swult} × l_{corner}/(2 × φ_{depslab}/√(2)) = 5.5 kN/m

Moment and shear due to udl
Maximum ultimate udl
Bending moment
Shear force

W_{cornerudl} = ((1.4 × W_{udsl}) + (1.6 × W_{Ludl})) × φ_{depslab}/√(2) = 15.8 kN/m
M_{cornerudl} = W_{cornerudl} × l_{corner}²/(6 × φ_{depslab}/√(2)) = 5.8 kNm/m
V_{cornerudl} = W_{cornerudl} × l_{corner}/(2 × φ_{depslab}/√(2)) = 8.1 kN/m

Resultant moments and shears
Total design moment
Total design shear force

M_{zcorner} = M_{corneruw} + M_{cornerudl} = 9.8 kNm/m
V_{zcorner} = V_{corneruw} + V_{cornerudl} = 13.5 kN/m

Reinforcement required in top of slab at corners
K factor
Lever arm
Area of steel required for bending
Area of steel required

K_{corner} = M_{zcorner}/(f_{cu} × d_{slabmin}²) = 0.033
Z_{corner} = d_{slabmin} × min(0.95, 0.5 + √(0.25 - K_{corner}/0.9)) = 88 mm
A_{cornerbend} = M_{zcorner}/(1.0f_{yk} × f_{yslab} × Z_{corner}) = 256 mm²/m
A_{corner} = max(A_{cornerbend}, A_{slabmin}) = 256 mm²/m
PASS - A_{corner} ≤ A_{sadgetop} - Area of reinforcement provided in top of slab at corners is adequate

Applied shear stress
Tension steel ratio
From BS8110-1:1997 - Table 3.8
Design concrete shear strength

V_{corner} = V_{zcorner}/d_{slabmin} = 0.146 N/mm²
ρ_{corner} = 100 × A_{adgetop}/d_{slabmin} = 0.425
V_{corner} = 0.766 N/mm²
PASS - V_{corner} ≤ V_{corner} - Shear capacity of the slab is not exceeded

Slab corner deflection check
Basic allowable span to depth ratio
Moment factor
Steel service stress
Modification factor

Ratio_{basicecorner} = 7.0
M_{factorcorner} = M_{zcorner}/d_{slabmin}² = 1.143 N/mm²
f_{scorner} = 2/3 × f_{yslab} × A_{cornerbend}/A_{sadgetop} = 217.200 N/mm²
MF_{corner} = min(2.0, 0.55 + ((477 N/mm² - f_{scorner})/(120 × (0.9 N/mm² + M_{factorcorner}))))
MF_{corner} = 1.609

Modified allowable span to depth ratio
Actual span to depth ratio

Ratio_{allowcorner} = Ratio_{basicecorner} × MF_{corner} = 11.266
Ratio_{actualcorner} = l_{corner}/d_{slabmin} = 23.447
FAIL - Ratio_{actualcorner} > Ratio_{allowcorner} - Slab span to depth ratio is inadequate

Appendix E – Vale Consultancy Drawing SK01 – Proposed Raft Slab

