Appendix D

envirolab

Units 7 & 8, Sandpits Business Park
Mottram Road, Hyde, Cheshire, SK14 3AR

Final Test Report

Envirolab Job Number:	17/07997		
Issue Number:	1	Date:	30-Nov-17

Client: Structural Soils Limited (Castleford Lab)
The Potteries
Pottery Street
Castleford
West Yorkshire
UK, WF10 1NJ

Project Manager:	Mark Athorne
Project Name:	Falkland
Project Ref:	782886
Order No:	N/A
Date Samples Received:	22-Nov-17
Date Instructions Received:	24-Nov-17
Date Analysis Completed:	30-Nov-17

Notes - Soil analysis

All results are reported as dry weight (<40°C).

For samples with Matrix Codes 1 - 6 natural stones >10mm are removed or excluded from the sample prior to analysis and reported results corrected to a whole sample basis.

For samples with Matrix Code 7 the whole sample is dried and crushed prior to analysis.

Notes - General

This report shall not be reproduced, except in full, without written approval from Envirolab.

Subscript "A" indicates analysis performed on the sample as received. "D" indicates analysis performed on the dried sample, crushed to pass a 2mm sieve, unless asbestos is found to be present in which case all analysis is performed on the sample as received.

All analysis is performed on the dried and crushed sample for samples with Matrix Code 7 and this supercedes any "A" subscripts.

All analysis is performed on the sample as received for soil samples from outside the European Union and this supercedes any "D" subscripts

Superscript "M" indicates method accredited to MCERTS.

For complex, multi-compound analysis, quality control results do not always fall within chart limits for every compound and we have criteria for reporting in these situations.

If results are in italic font they are associated with such quality control failures and may be unreliable.

A deviating samples report is appended and will indicate if samples or tests have been found to be deviating. Any test results affected may not be an accurate record of the concentration at the time of sampling and, as a result, may be invalid

Predominant Matrix Codes: 1 = SAND, 2 = LOAM, 3 = CLAY, 4 = LOAM/SAND, 5 = SAND/CLAY, 6 = CLAY/LOAM, 7 = OTHER, 8 = Asbestos bulk ID sample

Samples with Matrix Code 7 & 8 are not predominantly a SAND/LOAM/CLAY mix and are not covered by our BSEN 17025 or MCERTS accreditations, with the exception of bulk asbestos which are BSEN 17025 accredited

Secondary Matrix Codes: A = contains stones, B = contains construction rubble, C = contains visible hydrocarbons, D = contains glass/metal, E = contains roots/twigs.

IS indicates Insufficient sample for analysis, NDP indicates No Determination Possible and NAD indicates No Asbestos Detected.

Superscript # indicates method accredited to ISO 17025.

Analytical results reflect the quality of the sample at the time of analysis only. Opinions and interpretations expressed are outside the scope of our accreditation.

Please contact us if you need any further information.

Prepared by:

Richard Wong
Client Manager

Approved by:

Georgia King
Admins & Client Services Supervisor

UKAS TESTING No. 1247

MCERTS THE ENVIRONMENT AGENCY'S MONITORING CERTIFICATION SCHEME envirolab

Landfill WAC analysis must not be used for hazardous waste classification purposes. This analysis is only applicable for landfill acceptance and does not give any indication as to whether a waste may be hazardous or non-hazardous.

Sample Details								Landfill Waste Acceptance Criteria Limits		
Lab Sample ID	**Method**	**ISO17025**	**MCERTS**	**17/07997/3**						
Client Sample Number								**Inert Waste Landfill**	**Stable Non-reactive Hazardous Waste in Non-Hazardous Landfill**	**Hazardous Waste Landfill**
Client Sample ID				WS 02						
Depth to Top				0.8						
Depth to Bottom				0.90						
Date Sampled				15/11/2017						
Sample Type				Soil						
Sample Matrix Code				6A						
Solid Waste Analysis										
pH (pH Units)$_D$	A-T-031	Y	Y					-	>6	-
ANC to pH 4 (mol/kg)$_D$	A-T-ANC	N	N					-	to be evaluated	to be evaluated
ANC to pH 6 (mol/kg)$_D$	A-T-ANC	N	N					-	to be evaluated	to be evaluated
Loss on Ignition (%)$_D$	A-T-030	Y	N					-	-	10
Total Organic Carbon (%)$_D$	A-T-032	Y	Y	**2.54**				3	5	6
PAH Sum of 17 (mg/kg)$_A$	A-T-019	N	N	**<0.08**				100	-	-
Mineral Oil (mg/kg)$_A$	A-T-007	N	N	**<10**				500	-	-
Sum of 7 PCBs (mg/kg)$_D$	A-T-004	N	N	**<0.007**				1	-	-
Sum of BTEX (mg/kg)$_A$	A-T-022	N	N	**<0.01**				6	-	-
Eluate Analysis				**2:1**	**8:1**	**2:1**	**Cumulative 10:1**	**Limit values for compliance leaching test using**		
				mg/l		**mg/kg**		**BS EN 12457-3 at L/S 10 l/kg (mg/kg)**		
Arsenic	A-T-025	Y	N	0.010	0.008	0.022	**0.080**	0.5	2	25
Barium	A-T-025	Y	N	0.014	0.006	0.032	**0.070**	20	100	300
Cadmium	A-T-025	Y	N	0.016	0.017	0.036	**0.170**	0.04	1	5
Chromium	A-T-025	Y	N	0.063	0.014	0.143	**0.190**	0.5	10	70
Copper	A-T-025	Y	N	0.004	0.004	0.010	**0.040**	2	50	100
Mercury	A-T-025	Y	N	<0.0005	<0.0005	<0.001	**<0.005**	0.01	0.2	2
Molybdenum	A-T-025	Y	N	0.009	0.002	0.021	**0.030**	0.5	10	30
Nickel	A-T-025	Y	N	0.090	0.048	0.204	**0.530**	0.4	10	40
Lead	A-T-025	Y	N	0.012	0.038	0.028	**0.370**	0.5	10	50
Antimony	A-T-025	Y	N	0.004	0.002	0.008	**0.020**	0.06	0.7	5
Selenium	A-T-025	Y	N	<0.001	<0.001	<0.002	**<0.01**	0.1	0.5	7
Zinc	A-T-025	Y	N	0.038	0.054	0.087	**0.540**	4	50	200
Chloride	A-T-026	Y	N	9	<1.00	21	**<10**	800	15000	25000
Fluoride	A-T-026	Y	N	0.3	0.2	0.6	**2.0**	10	150	500
Sulphate as SO_4	A-T-026	Y	N	115	2	259	**122**	1000	20000	50000
Total Dissolved Solids	A-T-035	N	N	202	34	456	**498**	4000	60000	100000
Phenol Index	A-T-050	N	N	<0.01	<0.01	<0.02	**<0.1**	1	-	-
Dissolved Organic Carbon	A-T-032	N	N	<20.0	<20.0	<40	**<200**	500	800	1000
Leach Test Information										
pH (pH Units)	A-T-031	N	Y	7.7	8.1					
Conductivity (µS/cm)	A-T-037	N	N	404	68					
Mass Sample (kg)				0.200						
Dry Matter (%)	A-T-044	N	N	84.3						
Stage 1										
Volume Leachant, L_2 (l)	A-T-046			0.350						
Filtered Eluate Volume, VE_1 (l)	A-T-046			0.150						
Stage 2										
Volume Leachant, L_8 (l)	A-T-046			1.350						

Stated acceptance limits are for guidance only and Envirolab cannot be held responsible for any discrepancies with current legislation

Units 7 & 8 Sandpits Business Park
Mottram Road, Hyde, Cheshire, SK14 3AR

FINAL ANALYTICAL TEST REPORT

Envirolab Job Number: 17/07997
Issue Number: 1 **Date:** 30 November, 2017

Client: Structural Soils Limited (Castleford Lab)
The Potteries
Pottery Street
Castleford
West Yorkshire
UK
WF10 1NJ

Project Manager: Mark Athorne
Project Name: Falkland
Project Ref: 782886
Order No: N/A
Date Samples Received: 22/11/17
Date Instructions Received: 24/11/17
Date Analysis Completed: 30/11/17

Prepared by:	**Approved by:**
Richard Wong	Georgia King
Client Manager	Admins & Client Services Supervisor

1247

Envirolab Job Number: 17/07997

Client Project Name: Falkland

Client Project Ref: 782886

Lab Sample ID	17/07997/1	17/07997/2	17/07997/3	17/07997/4	17/07997/5	17/07997/6			Units	Method ref
Client Sample No										
Client Sample ID	WS01	WS02	WS 02	TP01	WS01	WS01				
Depth to Top	2.60	3.70	0.80		0.45	1.65				
Depth To Bottom			0.90							
Date Sampled	15-Nov-17	15-Nov-17	15-Nov-17	15-Nov-17	15-Nov-17	15-Nov-17				
Sample Type	Soil	Soil	Soil	Soil	Soil	Soil				
Sample Matrix Code	5	5	6A	6A	6A	5A				
% Stones >10mm$_A$	<0.1	<0.1	13.9	-	9.7	2.1			% w/w	A-T-044
pH BRE$_D$	8.26	7.79	-	-	-	-			pH	A-T-031s
Sulphate BRE (water sol 2:1)$_D^{M\#}$	426	2800	-	-	-	-			mg/l	A-T-026s
Arsenic$_D^{M\#}$	-	-	-	-	-	2			mg/kg	A-T-024s
Cadmium$_D^{M\#}$	-	-	-	-	-	64.0			mg/kg	A-T-024s
Copper$_D^{M\#}$	-	-	-	-	-	13			mg/kg	A-T-024s
Chromium$_D^{M\#}$	-	-	-	-	-	69			mg/kg	A-T-024s
Chromium (hexavalent)$_D$	-	-	-	-	-	<1			mg/kg	A-T-040s
Lead$_D^{M\#}$	-	-	-	-	-	23			mg/kg	A-T-024s
Mercury$_D$	-	-	-	-	-	<0.17			mg/kg	A-T-024s
Nickel$_D^{M\#}$	-	-	-	-	-	114			mg/kg	A-T-024s
Selenium$_D^{M\#}$	-	-	-	-	-	<1			mg/kg	A-T-024s
Zinc$_D^{M\#}$	-	-	-	-	-	83			mg/kg	A-T-024s
TPH total (>C6-C40)$_A$	-	-	-	-	<10	-			mg/kg	A-T-007s

Envirolab Job Number: 17/07997 **Client Project Name: Falkland**

Client Project Ref: 782886

Lab Sample ID	17/07997/1	17/07997/2	17/07997/3	17/07997/4	17/07997/5	17/07997/6			Units	Method ref
Client Sample No										
Client Sample ID	WS01	WS02	WS 02	TP01	WS01	WS01				
Depth to Top	2.60	3.70	0.80		0.45	1.65				
Depth To Bottom			0.90							
Date Sampled	15-Nov-17	15-Nov-17	15-Nov-17	15-Nov-17	15-Nov-17	15-Nov-17				
Sample Type	Soil	Soil	Soil	Soil	Soil	Soil				
Sample Matrix Code	5	5	6A	6A	6A	5A				
Asbestos in Soil (inc. matrix)										
Asbestos in soil$_A$#	-	-	-	NAD	-	-				A-T-045
Asbestos ACM - Suitable for Water Absorption Test?	-	-	-	N/A	-	-				

REPORT NOTES

General:
This report shall not be reproduced, except in full, without written approval from Envirolab.
All samples contained within this report, and any received with the same delivery, will be disposed of one month after the date of this report.
Analytical results reflect the quality of the sample at the time of analysis only.
Opinions and interpretations expressed are outside the scope of our accreditation.
If results are in italic font they are associated with an AQC failure and there is insufficient sample to repeat the analysis. These are not accredited and are unreliable.
A deviating samples report is appended and will indicate if samples or tests have been found to be deviating. Any test results affected may not be an accurate record of the concentration at the time of sampling and, as a result, may be invalid.

Soil chemical analysis:
All results are reported as dry weight (<40°C).
For samples with Matrix Codes 1 - 6 natural stones, brick and concrete fragments >10mm and any extraneous material (visible glass, metal or twigs) are removed and excluded from the sample prior to analysis and reported results corrected to a whole sample basis. This is reported as '% stones >10mm'.
For samples with Matrix Code 7 the whole sample is dried and crushed prior to analysis and this supersedes any “A” subscripts
All analysis is performed on the sample as received for soil samples which are positive for asbestos or the client has informed asbestos may be present and/or if they are from outside the European Union and this supersedes any "D" subscripts.

TPH analysis of water by method A-T-007:
Free and visible oils are excluded from the sample used for analysis so that the reported result represents the dissolved phase only.

Electrical Conductivity of water by Method A-T-037:
Results greater than 12900µS/cm @ 25°C / 11550µS/cm @ 20°C fall outside the calibration range and as such are unaccredited.

Asbestos:
Asbestos in soil analysis is performed on a dried aliquot of the submitted sample and cannot guarantee to identify asbestos if only present in small numbers as discrete fibres/fragments in the original sample.
Stones etc. are not removed from the sample prior to analysis.
Quantification of asbestos is a 3 stage process including visual identification, hand picking and weighing and fibre counting by sedimentation/phase contrast optical microscopy if required. If asbestos is identified as being present but is not in a form that is suitable for analysis by hand picking and weighing (normally if the asbestos is present as free fibres) quantification by sedimentation is performed. Where ACMs are found a percentage asbestos is assigned to each with reference to 'HSG264, Asbestos: The survey guide' and the calculated asbestos content is expressed as a percentage of the dried soil sample aliquot used.

Predominant Matrix Codes:
1 = SAND, 2 = LOAM, 3 = CLAY, 4 = LOAM/SAND, 5 = SAND/CLAY, 6 = CLAY/LOAM, 7 = OTHER, 8 = Asbestos bulk ID sample.
Samples with Matrix Code 7 & 8 are not predominantly a SAND/LOAM/CLAY mix and are not covered by our BSEN 17025 or MCERTS accreditations, with the exception of bulk asbestos which are BSEN 17025 accredited.

Secondary Matrix Codes:
A = contains stones, B = contains construction rubble, C = contains visible hydrocarbons, D = contains glass/metal,
E = contains roots/twigs.

Key:
IS indicates Insufficient Sample for analysis.
US indicates Unsuitable Sample for analysis.
NDP indicates No Determination Possible.
NAD indicates No Asbestos Detected.
N/A indicates Not Applicable.
Superscript # indicates method accredited to ISO 17025.
Superscript "M" indicates method accredited to MCERTS.
Subscript "A" indicates analysis performed on the sample as received.
Subscript "D" indicates analysis performed on the dried sample, crushed to pass a 2mm sieve

Please contact us if you need any further information.

STRUCTURAL SOILS LTD

TEST REPORT

Report No. 782886 R1 — 1774

Date	30-November-2017	Contract	Falkland

Client: Ashton Bennett Consultancy
Address: Unit K
Bridge Mills
Huddersfield Road
Holmfirth HD9 3TW

For the Attention of: Frances Bennett

Samples submitted by client	17/11/2017	Client Reference	
Testing Started	21/11/2017	Client Order No.	
Testing Completed	30/11/2017	Instruction Type	Written

UKAS Accredited Tests Undertaken

Moisture Content (oven drying method) BS1377:Part 2:1990,clause 3.2 (superseded) **
Liquid Limit (definitive method) BS1377:Part 2:1990,clause 4.3
Plastic Limit BS1377:Part 2:1990,clause 5.3
Plasticity Index Derivation BS1377:Part 2:1990,clause 5.4

* This clause of BS1377 is no longer the most up to date method due to the publication of ISO17892

Please Note: Remaining samples will be retained for a period of one month from today and will then be disposed of.
Test were undertaken on samples 'as received' unless otherwise stated.
Opinions and interpretations expressed in this report are outside the scope of accreditation for this laboratory.

Structural Soils Ltd, The Potteries, Pottery Street, Castleford, WF10 1NJ Tel.01977 552255. E-mail mark.athorne@soils.co.uk

SUMMARY OF SOIL CLASSIFICATION TESTS

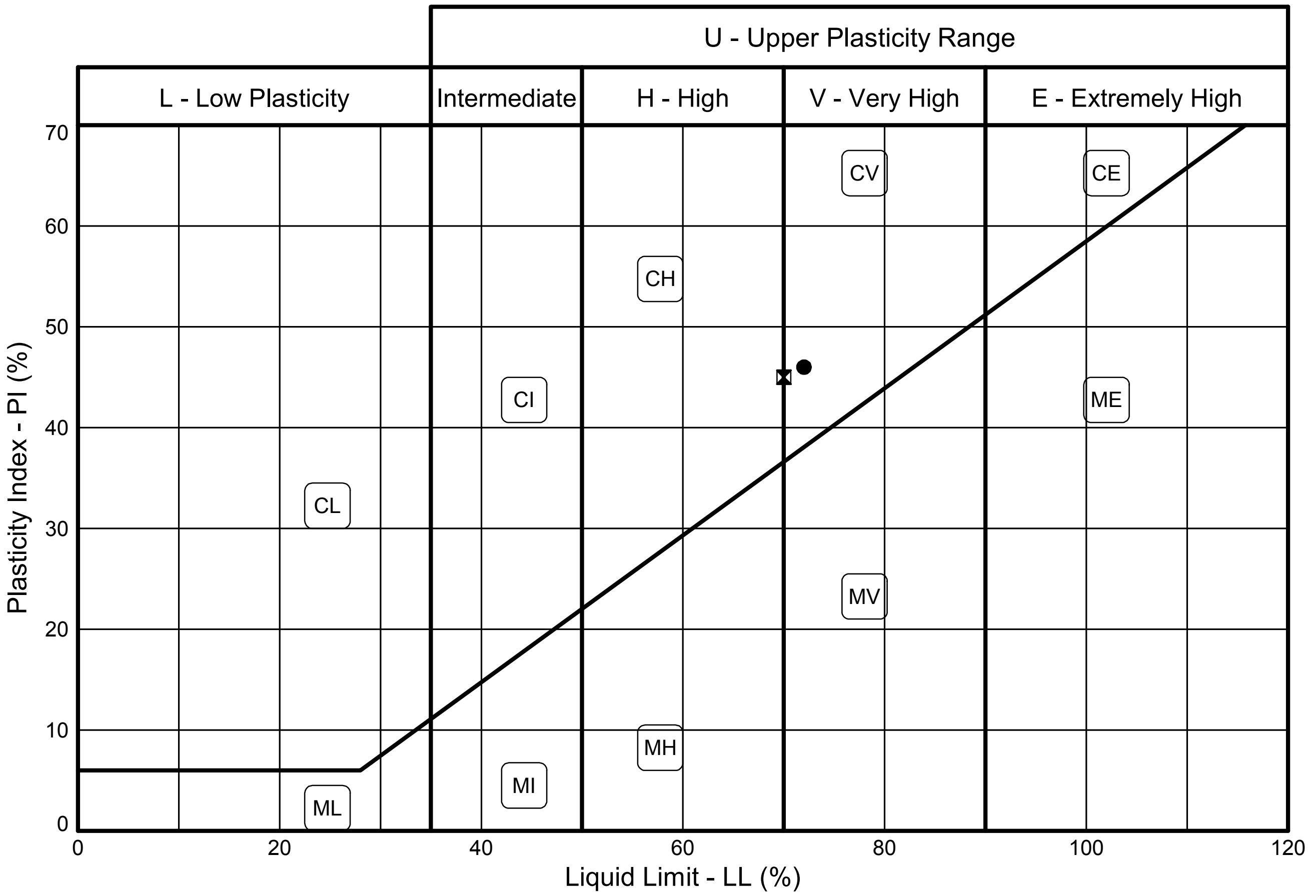
In accordance with clauses 3.2,4.3,4.4,5.3,5.4,7.2,8.2,8.3 of BS1377:Part 2:1990

Exploratory Position ID	Sample Ref	Sample Type	Depth (m)	Moisture Content %	Liquid Limit %	Plastic Limit %	Plasticity Index	% <425um	Description of Sample
WS1	4	B	3.60	29	72	26	46	97	Brown slightly sandy slightly gravelly CLAY
WS2	1	B	2.50	27	70	25	45	99	Brown slightly sandy slightly gravelly CLAY

STRUCTURAL SOILS LTD	Contract: Falkland	Contract Ref: 782886

PLASTICITY CHART - PI Vs LL

In accordance with BS5930:2015
Testing in accordance with BS1377-2:1990



Sample Identification			BS Test Method #	Preparation Method +	MC %	LL %	PL %	PI %	<425um %	Lab location
Exploratory Position ID	Sample	Depth (m)								
● WS1	4B	3.60	3.2/4.3/5.3/5.4	4.2.4	29	72	26	46	97	C
⊠ WS2	1B	2.50	3.2/4.3/5.3/5.4	4.2.4	27	70	25	45	99	C

Tested in accordance with the following clauses of BS1377-2:1990.

3.2 - Moisture Content
4.3 - Cone Penetrometer Method
4.4 - One Point Cone Penetrometer Method
4.6 - One Point Casagrande Method
5.3 - Plastic Limit Method
5.4 - Plasticity Index

+ Tested in accordance with the following clauses of BS1377-2:1990.

4.2.3 - Natural State
4.2.4 - Wet Sieved

Key: * = Non-standard test, NP = Non plastic.

Lab location: B = Bristol (BS3 4AG), C = Castleford (WF10 1NJ), H = Hemel Hempstead (HP3 9RT), T = Tonbridge (TN11 9HU)

STRUCTURAL SOILS
The Potteries
Pottery Street
Castleford
W. Yorkshire WF10 1NJ

Compiled By		Date
M. Fisher	MAUREEN FISHER	30/11/17

Contract	Contract Ref:
Falkland	782886

AGS

Appendix **E**

Job Number: 171203

CROFT STRUCTURAL ENGINEERS

Croft Structural Engineers
Clock Shop Mews
Rear of 60 Saxon Road
London SE25 5EH

T: 020 8684 4744
E: enquiries@croftse.co.uk
W: www.croftse.co.uk

Basement Impact Assessment

Property Details
1&2 Falkland Mews
London
NW5 2PP

Client Information
A. Patel and P. Winford

Structural Design Reviewed by	**Above Ground Drainage Reviewed by**
Chris Tomlin MEng CEng MIStructE	Phil Henry BEng MEng MICE

Hydrogeology Report	**Land Stability Report**
(Separate Report) Ashton Bennett	(Separate Report) Ashton Bennett

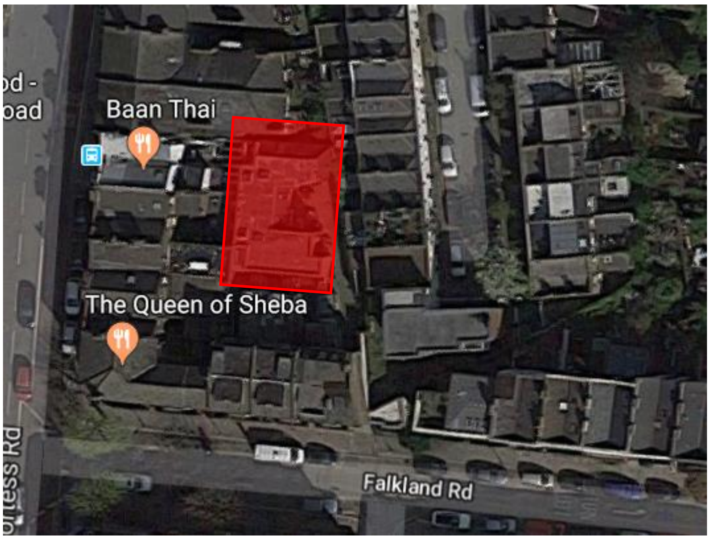
Revision	Date	Comment
-	02.02.2018	First Issue
1	18.04.2018	Minor Alterations
2	20.11.2018	Minor Alterations

Contents

CROFT STRUCTURAL ENGINEERS

Executive (non-technical) Summary

	The London Borough of Camden requires a Basement Impact Assessment (BIA) to be prepared for developments that include basements and lightwells. This document forms the main part of the BIA and gives details on the impact of surface water flow. The scheme design for the proposed subterranean structure is also included. This document should be used in conjunction with the Land Stability BIA and the Groundwater BIA. These are separate reports and are referred to, where relevant, within this document. This BIA follows the requirements contained within Camden Council's planning guidance CPG Basements (2018). In summary, the council will only allow basement construction to proceed if it does not: • cause harm to the built or natural environment and local amenity • result in flooding • lead to ground instability. In order to comply with the above clauses, a BIA must undertake five stages detailed in CPG Basements (2018). This report has been produced in line with Camden planning guidance and associated supporting documents such as CPG1, DP23, DP26, DP25 and DP27. Technical information from 'Camden geological, hydrogeological and hydrological study - Guidance for subterranean development', Issue 01, November 2010 (GSD, hereafter) was also used and is referred to in this assessment.
Existing Property	The site comprises a building which is two storeys high above street level. The building is separated into two properties and is constructed from traditional building materials (brickwork and timber). A chartered engineer (refer to cover for qualifications) has reviewed the topography of the site and the surrounding area and has concluded that there is a very low risk of landslip.

Proposed Development	The proposed development involves the construction of a new basement below the existing building with front lightwells.  *Figure 1: Map / Aerial view with approx. site area indicated*

1. Structural Desk Study

This section identifies the relevant features of the site and its immediate surroundings, providing further scoping where required.

Desk Study and Walkover Survey

Site & Existing Property

The property is located on Falkland Road close to its junction with Leverton Street.

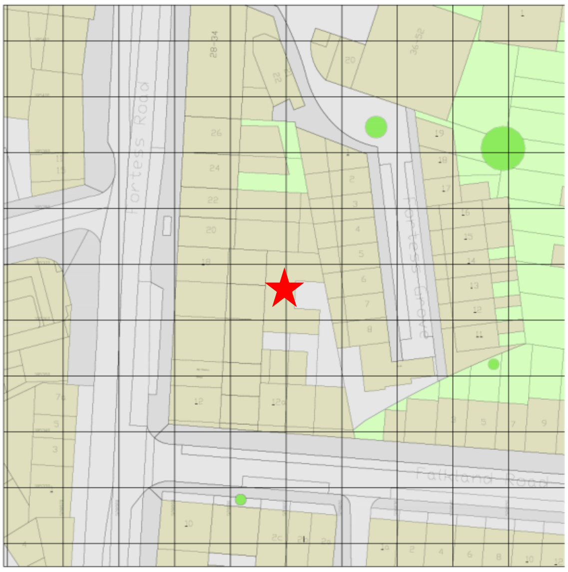


Figure 2: Site Plan

A structural engineer from Croft visited the property on 12th January 2018

The site comprises a building which is two storeys high above street level. The building is separated into two properties and is constructed from traditional building materials (brickwork and timber).

Figure 3: No.1 Falkland Mews

Figure 4: No.2 Falkland Mews

The building is constructed from traditional building materials (brickwork and timber) and is believed to be approximately 100 years old. Paving is present immediately outside of the building perimeter.

Proposed Development

The proposed development involves the construction of a basement under the footprint of the property and front lightwells.
More details on the proposed structure are given later in this report.

Plans showing the extent of the proposed basement are shown on Architectural drawings (EXISTING and PROPOSED, by Bashkal & Associates, available separately). Drawings showing the structural scheme are appended.

The depth of the basement will be approximately 3m below street level.

An aerial view of the area is shown below. This site is indicated. In addition to the basement area, this also includes areas that are likely to be temporarily occupied for construction purposes.

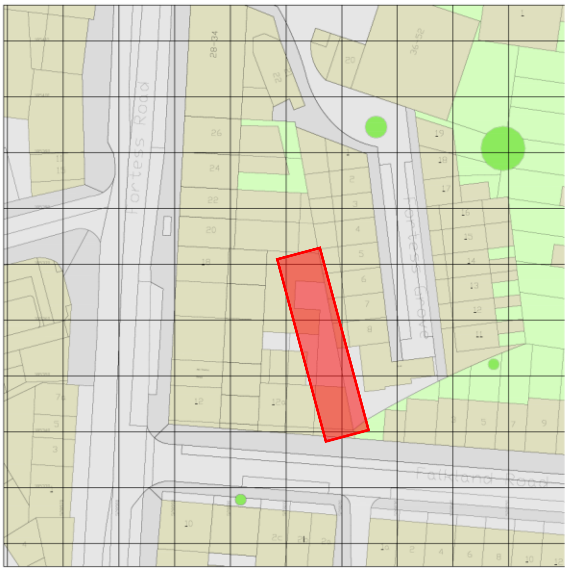


Figure 5: Aerial view with approx. site area indicated

The outline construction sequence is appended to this report.

London Underground

From inspection, the site is more than 20m away from the nearest subterranean train line. It is unlikely that the development will have any significant effect on tunnels and vice-versa.

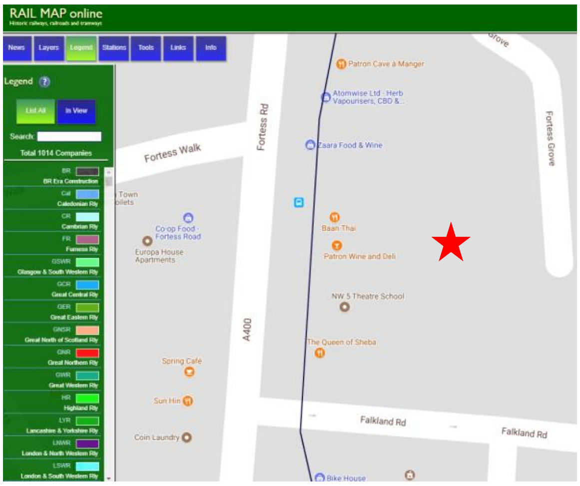


Figure 6: Extract from RAIL MAP online showing proximity of rail lines and subterranean lines

LUL have been informed of this proposal (e-mails are appended). At detailed design stage, after the planning stage is concluded, the design team should continue to liaise with the relevant Asset Protection Team for as long as is considered necessary by LUL.

Proximity of Trees

There are no trees close by, in the neighbouring land.

Structural Stability of Adjacent Properties

The external facades of the neighbouring properties have been inspected.

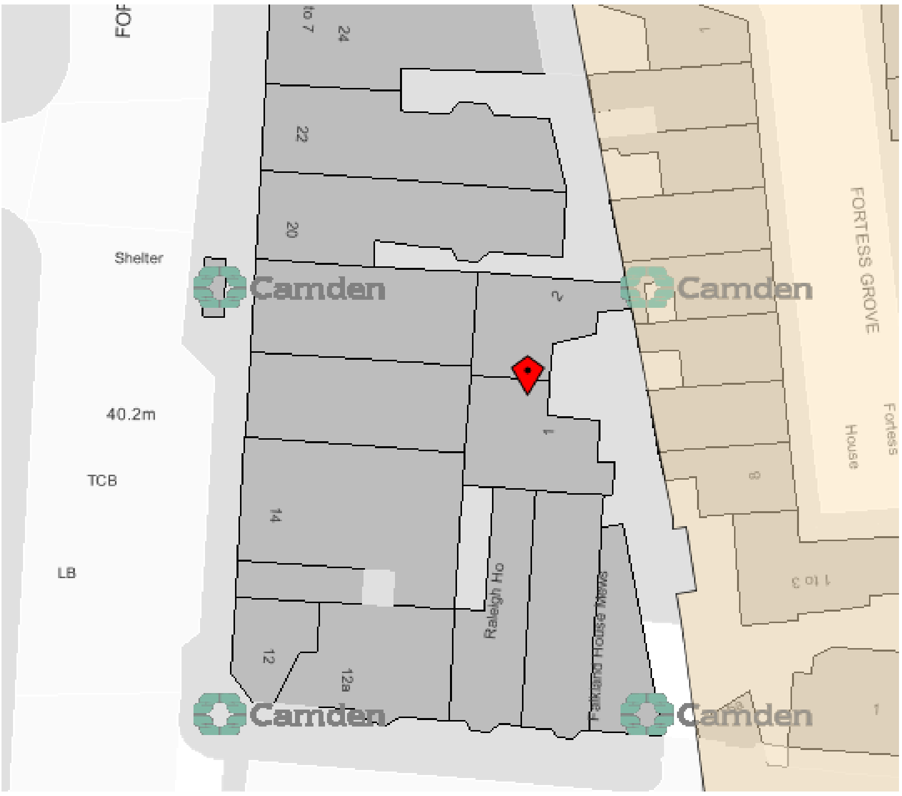


Figure 7: Plan view of site (approx. area outlined in red) and the surrounding properties

Descriptions of the properties are given below.

CROFT STRUCTURAL ENGINEERS

Nos 20 Fortess Road –	Property age : mid-Victorian (~150 years old) Property use : Commercial (ground floor), Residential (other floors) Number of storeys : the property is four storeys high above ground level. Is a basement present? : **No** – there is no basement. Camden Council's website confirm that there is no basement.  *Figure 8: front of No 20 Fortess Road* Structural Defects Noted: None noted

CROFT STRUCTURAL ENGINEERS

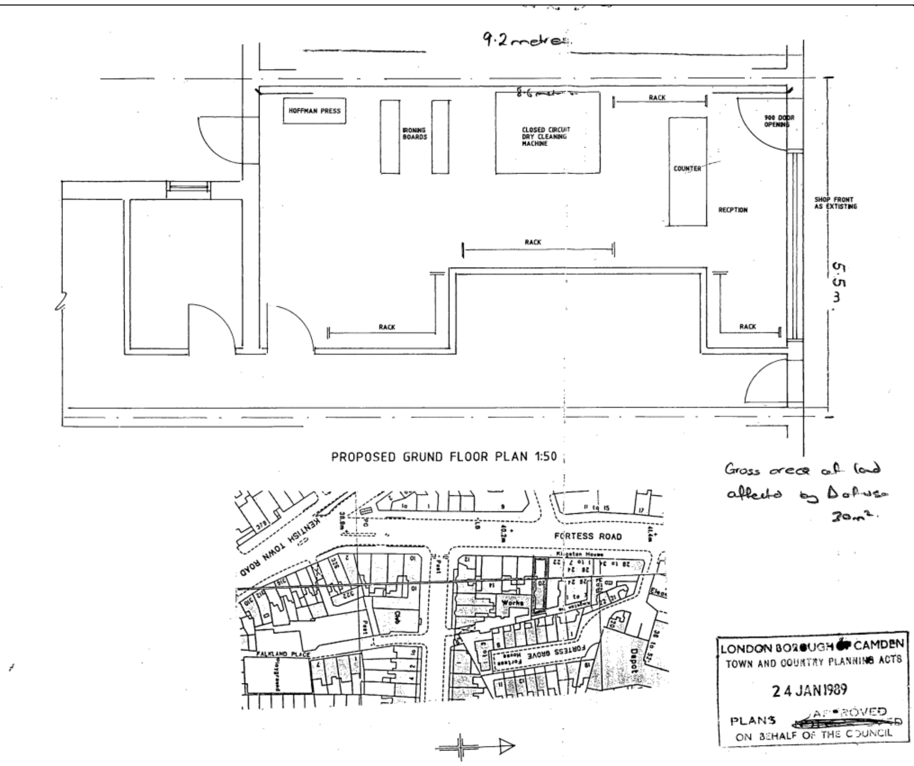
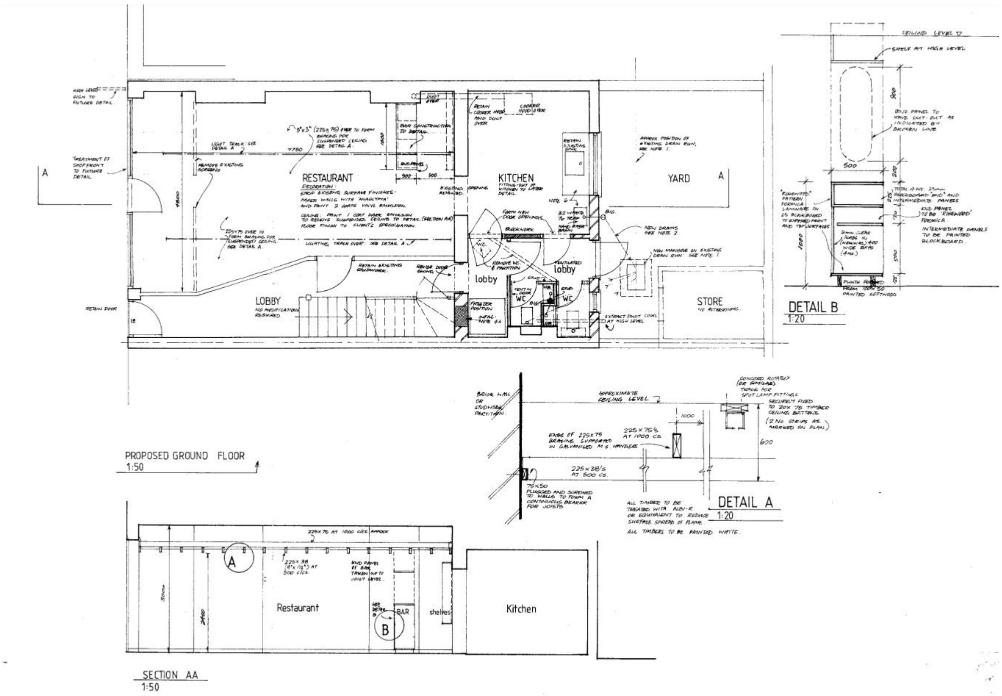
	 Structural assessment of ongoing movement: from observing the external façade of the building, there were no visible signs of movement.
Nos 18 Fortess Road –	Property age : mid-Victorian (~150 years old) Property use : Commercial (ground floor), Residential (other floors) Number of storeys : the property is three storeys high above ground level. Is a basement present? : **No** – there is no basement. Camden Council's website confirm that there is no basement.

Figure 9: front of No 18 Fortess Road

Structural Defects Noted: None noted



Structural assessment of ongoing movement: from observing the external façade of the building, there were no visible signs of movement.

CROFT STRUCTURAL ENGINEERS

Nos 16 Fortess Road –

Property age : mid-Victorian (~150 years old)

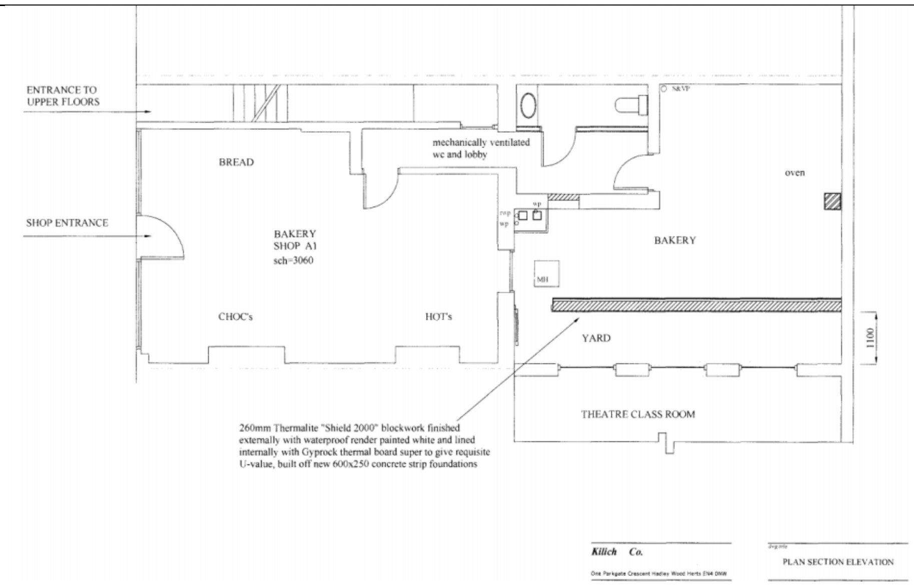
Property use : Commercial (ground floor), Residential (other floors)

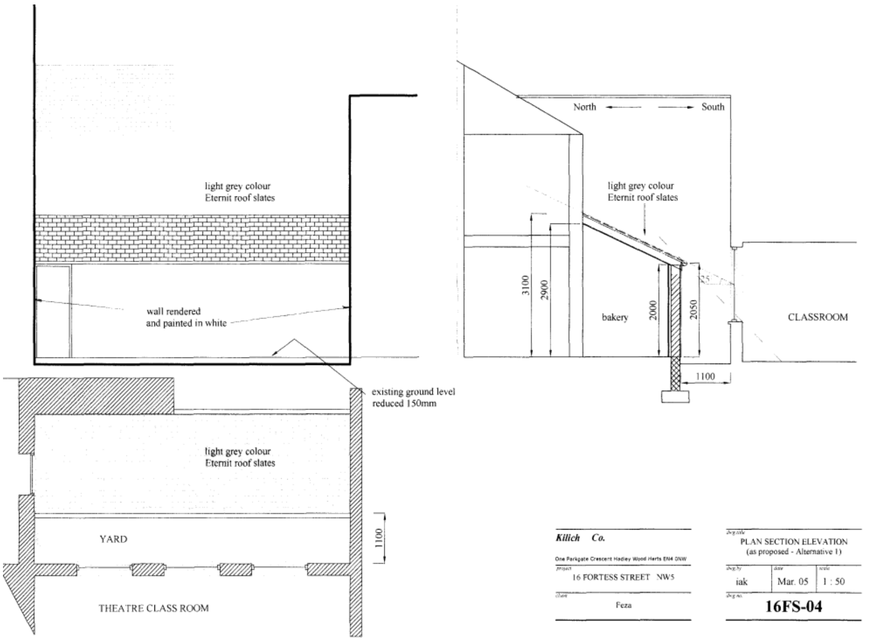
Number of storeys : the property is three storeys high above ground level.

Is a basement present? : **No** – there is no basement. Camden Council's website confirm that there is no basement.

Figure 10: front of No 16 Fortess Road

Structural Defects Noted: None noted





Structural assessment of ongoing movement: from observing the external façade of the building, there were no visible signs of movement.

Nos 14 Fortess Road –

Property age : mid-Victorian (~150 years old)

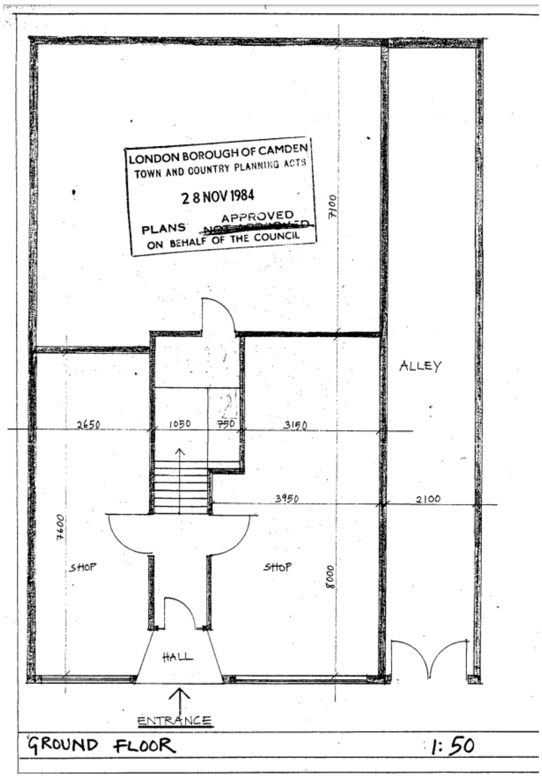
Property use : Commercial (ground floor), Residential (other floors)

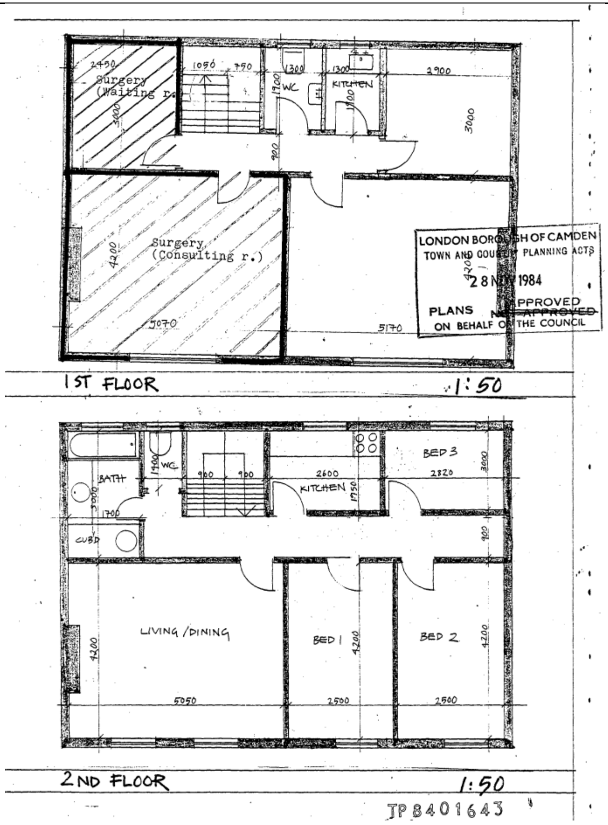
Number of storeys : the property is three storeys high above ground level.

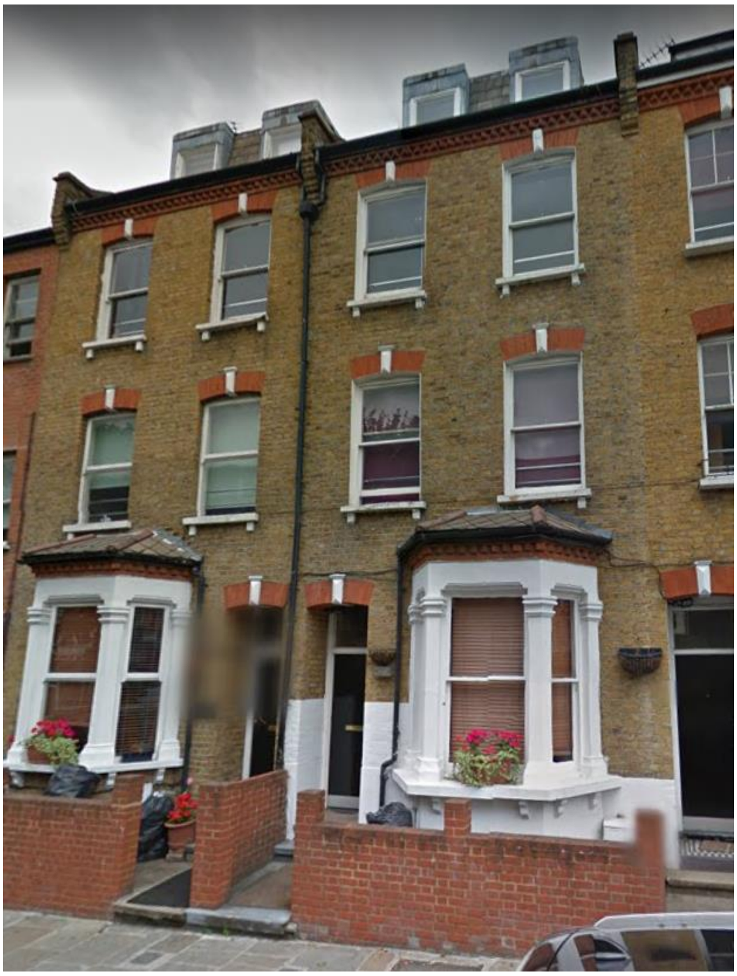
Is a basement present? : **No** – there is no basement. Camden Council's website confirm that there is no basement.

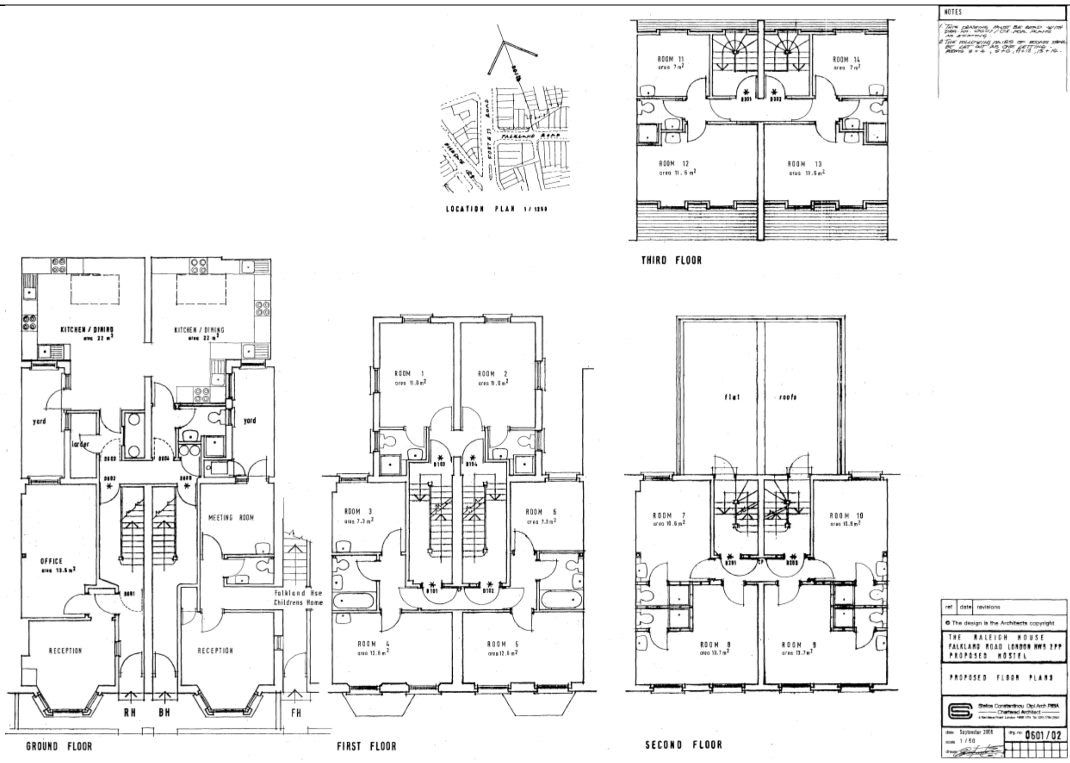
Figure 11: front of No 14 Fortess Road

Structural Defects Noted: None noted



	 Structural assessment of ongoing movement: from observing the external façade of the building, there were no visible signs of movement.

Raleigh House	Property age : mid-Victorian (~150 years old) Property use : Commercial (ground floor), Residential (other floors) Number of storeys : the property is four storeys high above ground level. Is a basement present? : **No** – there is no basement. Camden Council's website confirm that there is no basement.  *Figure 12: front of No 14 Fortess Road* Structural Defects Noted: None noted



Structural assessment of ongoing movement: from observing the external façade of the building, there were no visible signs of movement.

Mitigation Measures Ground Movement

The BIA by Ashton Bennett emphasised the requirement for best practice construction methods to limit any ground movements and associated damage to the neighbouring properties.

The proposed construction method, appended to this report, aims to limit damage to acceptable levels. For this development, suitable temporary propping during the construction phase will limit the amount of movement due to the basement works. The procedures will mitigate the impacts that the construction of the basement will have on nearby properties.

The works must be carried out in accordance with the Party Wall Act and condition surveys will be necessary at the beginning and the end of the works. A Party Wall Application will be required with Nos.20, 18, 16,14 , Fortess Road and with Raleigh House. The Party Wall Approval procedure will reinforce the use of the proposed method statement and, if necessary, require it to be developed in more detail with more stringent requirements than those required at planning stage.

It is not expected that any cracking will occur in nearby structures during the works. However, Croft's experience advises that there is a risk of movement to the neighbouring property.

To reduce the risk to the development:

- Employ a reputable firm that has extensive knowledge of basement works.
- Employ suitably qualified consultants Croft Structural Engineers has completed over 500 basements in the last five years.
- Provide method statements for the contractors to follow
- Investigate the ground this has now been done.
- Record and monitor the properties close by. This is completed by a condition survey under the Party Wall Act, before and after the works are completed. Refer to the end of the appended Basement Construction Method Statement.

Monitoring of Structures

	In order to safeguard the existing structures during underpinning and new basement construction, movement monitoring is to be undertaken.
Risk Assessment	(see below)

Monitoring Level proposed	Type of Works.
Monitoring 4 Visual inspection and production of condition survey by Party Wall Surveyors at the beginning of the works and also at the end of the works. Visual inspection of existing party wall during the works. Inspection of the footing to ensure that the footings are stable and adequate. Vertical monitoring movement by standard optical equipment Lateral movement between walls by laser measurements	New basements greater than 2.5m and shallower than 4m Deep in gravels Basements up to 4.5m deep in clays Underpinning works to Grade I listed building

Before the works begin, a detailed monitoring report is required to confirm the implementation of the monitoring. The items that this should cover are:

- Risk Assessment to determine level of monitoring
- Scope of Works
- Applicable standards
- Specification for Instrumentation
- Monitoring of Existing cracks
- Monitoring of movement
- Reporting
- Trigger Levels using a RED / AMBER / GREEN System

Recommend levels are shown within the proposed monitoring statement (appended).

Basement Design & Construction Impacts and Initial Design Considerations	
Design Concept	The basement will consist of RC (reinforced concrete) cantilevered retaining walls. These will be designed to resist the lateral loads around the perimeter of the basement. The basement floor structure will comprise reinforced concrete that will be part of the bases of the retaining walls. The RC walls will also transfer vertical loads to the ground. A very small amount of heave is predicted. This will be mitigated by the applied vertical pressure from the base of the retaining walls. The investigations highlight that water is present. The walls are designed to resist the hydrostatic pressure. The water table was recorded at 0.79m at the shallowest. The design of the walls considers long term scenarios. It is possible that a water main may break causing a local higher water table. To account for this, the wall should be designed for a water level at full height of the basement. The design of basements often considers floatation as a risk. The design for the basement at this site accounts for the weight of the building and the uplift forces from the water. The weight of the building is greater than the uplift, resulting in a stable structure. Drawings showing the structural scheme design are appended.

Additional loading requirements	The lateral earth pressure exerts a horizontal force on the retaining walls. The retaining walls will be checked for resistance to the overturning force this produces. Lateral forces will be applied from: • Soil loads • Hydrostatic pressures • Surcharge loading from behind the retaining walls Surcharge Loading The following will be applied as surcharge loads to the retaining walls: • Surcharge for adjacent property 1.5kN/m2 + 4kN/m2 for concrete ground bearing slab The appended calculations show the design of one of the most heavily loaded retaining wall. The most critical parameters have been used for this. Due to heave of the clay soil, there is for potential upward pressure from the soil to be applied to the basement slab. Appended calculations show the amount of area that will be subjected to heave. Measures to mitigate this include forming permanent void between the soil and the slab. This can be achieved by the use of clayboard or a similar void forming product. The exact material and thickness should be determined at detailed design stage, after the planning application is concluded.
Temporary Works	Prior to and works on site, a utility search survey should be done at detailed design stage. In addition to this the contractor should incorporate into his method statement proposals to confirm the presence or absence of services below ground level. The majority of standard services are usually within 900mm of the ground surface. The contractor may consider stipulating trial excavations done by hand to this depth. Temporary propping details will be required. This must be provided by the contractor. Their details should be forwarded to the design stage engineer. Water levels should be monitored for at least one month prior to starting on site and throughout the construction process. Localised dewatering to pin excavations may be necessary. Construction Management Camden Council may require a management plan for construction, construction traffic and demolition. Proposals for what this should account for are described in the next section. An outline construction programme is appended.

Construction Impact Mitigation	The contractor should control the impacts on the local amenity. A management plan for demolition and construction will be required at detailed design stage. Considerations that the contractor and the design team should account for in the construction management plan are described below. <u>Noise Control</u> • The hours of working will be limited to those allowed: 8am to 5pm Monday to Friday and Saturday, 8am to 1pm. The hours of working will further be defined within the Party Wall Act and the requirements of Camden Council. • The site will be hoarded with 8' site hoarding to prevent access. • Working in the basement generally requires hand tools to be used. The level of noise generally will be no greater than that of digging of soil. The noise is reduced and muffled by the works being undertaken underground. The level of noise from basement construction works is lower than typical ground level construction due to this. • None of the construction practices cause undue noise greater than what is expected on a typical construction site (a conveyor belt typically runs at around 70dB). Site hoarding acts as a partial acoustic screen and will reduce the level of direct noise from the site. <u>Dust and Vibration Control</u> • Reduce the need to use vibrating and percussive machinery. • Use well-maintained and modern machinery • Plant/vehicles should be cleaned before exiting the site. • Water should be applied to suppress dust • Skips and storage of fine materials should be covered <u>Traffic Control</u> • Consideration of site traffic to, from and along Falkland Mews should be considered carefully; this should include identifying access and exit routes, planned delivery times and vehicle swept paths. • Banksmen should assist with vehicle movements close to and within the site to ensure the safety of site staff, visitors and other people close to the site. • Construction vehicle movements should be co-ordinated with deliveries to other properties close by and vehicle movements for other construction sites in the vicinity. • A Construction Traffic Management Plan should implement the above. This should be developed at detailed design stage.

CROFT STRUCTURAL ENGINEERS

The contractor is to follow the good working practices and guidance laid down in the 'Considerate Constructors Scheme'. This scheme commits construction sites to commit to care about appearance, respect the community, protect the environment and secure everyone's safety. The scheme will reinforce the measures above described above.

Figure 13: Examples of sites registered with the Considerate Constructors Scheme

With good construction practices adopted, the impact on the local amenity will be minimised.

Appendix A: Structural Calculations

CPG Basements (2018) highlights that other permits and requirements will be necessary after planning. Item 5.1 highlights that Building Regulations will be required. As part of the building control pack full calculations must be undertaken and provided at detailed design stage once planning permission is granted. The calculations must be completed to a recognised Standard (BS or Euro Codes). The calculations must take into account the findings of this report and the recommendations of the auditors.

The design must resist:

- Vertical loads from the proposed works and adjacent properties
- Lateral loads from wind, soil water and adjacent properties
- Loadings in the temporary condition
- All other applied loads on the building
- Uplift forces from hydrostatic effects and soil heave

The final proposed scheme must:

- Provide stability in the temporary condition to all forces
- Provide stability to all forces in the permanent condition

As part of the planning Croft structural engineers has considered some of the pertinent parts of the basement structure to ensure that it can be constructed. <u>The following calculations are not a full set of calculations for the final design </u>which must be provided for building regulations. The structural calculations we consider pertinent and included in this appendix for this development are:

1. Basement wall analysis and partial design

W1 - RC WALL - PERMANENT CONDITION

The proposed basement will be founded at approximately 3m below the existing ground floor level. The basement calculations below will address the retaining wall in temporary and permanent case.

The **temporary case** will include calcuations with no water applied to the back of the wall and the pressure will be taken at rest. Loads are calculating taking in consideration neigbhour's structure.

Location	Area			Type	L	Action	Actions, kN or kN/m			
	L	W	m^2			kN/m^2	Perm., g_k	%	Var., q_k	Total
W1 - RC Wall										
Ground floor*2	2	2	4	g_k		5.00	20.0			
				q_k		1.50			6.0	
First floor*2	2	2	4	g_k		0.63	2.5			
				q_k		1.50			6.0	
External wall	6	1	6	g_k		3.98	23.9			
Roof	2	2	4	g_k		1.03	4.1			
				q_k		0.75			3.0	
							50.5	kN/m	15.0	kN/m

RETAINING WALL ANALYSIS

In accordance with EN1997-1:2004 incorporating Corrigendum dated February 2009 and the UK National Annex incorporating Corrigendum No.1

Tedds calculation version 2.6.11

Retaining wall details

Stem type	Cantilever
Stem height	h_{stem} = **3000** mm
Stem thickness	t_{stem} = **300** mm
Angle to rear face of stem	α = **90** deg
Stem density	γ_{stem} = **25** kN/m^3
Toe length	l_{toe} = **1500** mm
Base thickness	t_{base} = **300** mm
Base density	γ_{base} = **25** kN/m^3
Height of retained soil	h_{ret} = **3000** mm
Angle of soil surface	β = **0** deg
Depth of cover	d_{cover} = **0** mm
Height of water	h_{water} = **3000** mm
Water density	γ_w = **9.8** kN/m^3

Retained soil properties

Soil type	Stiff clay
Moist density	γ_{mr} = **19** kN/m^3
Saturated density	γ_{sr} = **19** kN/m^3
Characteristic effective shear resistance angle	$\phi'_{r.k}$ = **18** deg

Characteristic wall friction angle $\delta_{r.k}$ = **9** deg

Base soil properties

Soil type: Stiff clay

Soil density γ_b = **19** kN/m^3

Characteristic effective shear resistance angle $\phi'_{b.k}$ = **18** deg

Characteristic wall friction angle $\delta_{b.k}$ = **9** deg

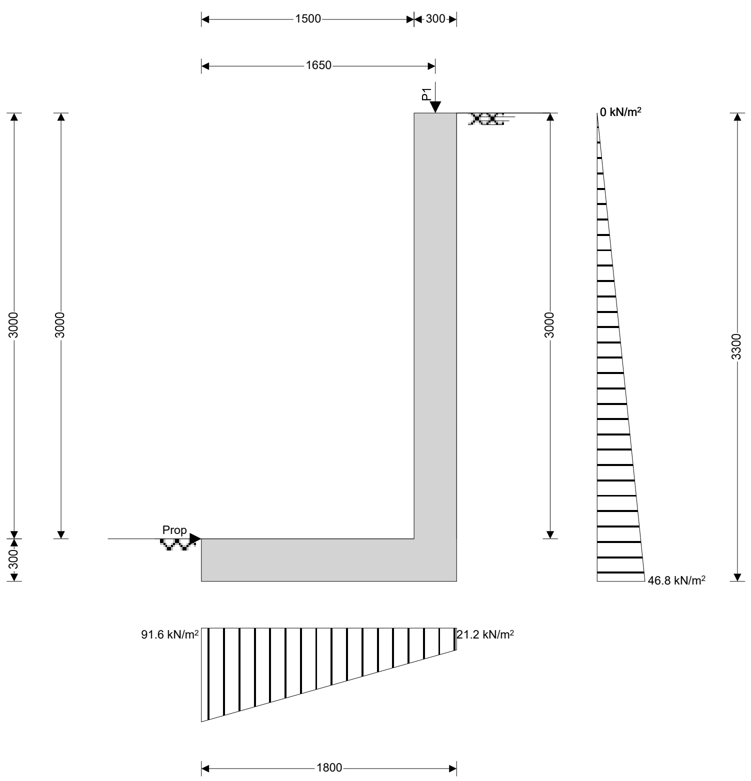
Characteristic base friction angle $\delta_{bb.k}$ = **12** deg

Presumed bearing capacity $P_{bearing}$ = **100** kN/m^2

Loading details

Vertical line load at 1650 mm P_{G1} = **50.5** kN/m

P_{Q1} = **15** kN/m



General arrangement

Calculate retaining wall geometry

Base length $l_{base} = l_{toe} + t_{stem}$ = **1800** mm

Saturated soil height $h_{sat} = h_{water} + d_{cover}$ = **3000** mm

Moist soil height $h_{moist} = h_{ret} - h_{water}$ = **0** mm

Retained surface length $l_{sur} = l_{heel}$ = **0** mm

Effective height of wall $h_{eff} = h_{base} + d_{cover} + h_{ret}$ = **3300** mm

Area of wall stem $A_{stem} = h_{stem} \times t_{stem}$ = **0.9** m^2

- Distance to vertical component $x_{stem} = l_{toe} + t_{stem} / 2$ = **1650** mm

Area of wall base $A_{base} = l_{base} \times t_{base}$ = **0.54** m^2

- Distance to vertical component $x_{base} = l_{base} / 2$ = **900** mm

Using Coulomb theory

Active pressure coefficient $K_A = \sin(\alpha + \phi'_{r.k})^2 / (\sin(\alpha)^2 \times \sin(\alpha - \delta_{r.k}) \times [1 + \sqrt{[\sin(\phi'_{r.k} + \delta_{r.k}) \times \sin(\phi'_{r.k} - \beta) / (\sin(\alpha - \delta_{r.k}) \times \sin(\alpha + \beta))]}]^2) = \mathbf{0.483}$

Passive pressure coefficient $K_P = \sin(90 - \phi'_{b.k})^2 / (\sin(90 + \delta_{b.k}) \times [1 - \sqrt{[\sin(\phi'_{b.k} + \delta_{b.k}) \times \sin(\phi'_{b.k}) / (\sin(90 + \delta_{b.k}))]}]^2) = \mathbf{2.359}$

Bearing pressure check

Vertical forces on wall

Wall stem $F_{stem} = A_{stem} \times \gamma_{stem} = \mathbf{22.5}$ kN/m

Wall base $F_{base} = A_{base} \times \gamma_{base} = \mathbf{13.5}$ kN/m

Line loads $F_{P\_v} = P_{G1} + P_{Q1} = \mathbf{65.5}$ kN/m

Total $F_{total\_v} = F_{stem} + F_{base} + F_{water\_v} + F_{P\_v} = \mathbf{101.5}$ kN/m

Horizontal forces on wall

Saturated retained soil $F_{sat\_h} = K_A \times \cos(\delta_{r.d}) \times (\gamma_{sr}' - \gamma_w') \times (h_{sat} + h_{base})^2 / 2 = \mathbf{23.9}$ kN/m

Water $F_{water\_h} = \gamma_w' \times (h_{water} + d_{cover} + h_{base})^2 / 2 = \mathbf{53.4}$ kN/m

Moist retained soil $F_{moist\_h} = K_A \times \cos(\delta_{r.d}) \times \gamma_{mr}' \times ((h_{eff} - h_{sat} - h_{base})^2 / 2 + (h_{eff} - h_{sat} - h_{base}) \times (h_{sat} + h_{base})) = \mathbf{0}$ kN/m

Base soil $F_{pass\_h} = -K_P \times \cos(\delta_{b.d}) \times \gamma_b' \times (d_{cover} + h_{base})^2 / 2 = \mathbf{-2}$ kN/m

Total $F_{total\_h} = F_{sat\_h} + F_{moist\_h} + F_{pass\_h} + F_{water\_h} = \mathbf{75.3}$ kN/m

Moments on wall

Wall stem $M_{stem} = F_{stem} \times x_{stem} = \mathbf{37.1}$ kNm/m

Wall base $M_{base} = F_{base} \times x_{base} = \mathbf{12.2}$ kNm/m

Line loads $M_P = (P_{G1} + P_{Q1}) \times p_1 = \mathbf{108.1}$ kNm/m

Saturated retained soil $M_{sat} = -F_{sat\_h} \times x_{sat\_h} = \mathbf{-26.3}$ kNm/m

Water $M_{water} = -F_{water\_h} \times x_{water\_h} = \mathbf{-58.8}$ kNm/m

Moist retained soil $M_{moist} = -F_{moist\_h} \times x_{moist\_h} = \mathbf{0}$ kNm/m

Total $M_{total} = M_{stem} + M_{base} + M_{sat} + M_{moist} + M_{water} + M_P = \mathbf{72.3}$ kNm/m

Check bearing pressure

Propping force $F_{prop\_base} = F_{total\_h} = \mathbf{75.3}$ kN/m

Distance to reaction $\bar{x} = M_{total} / F_{total\_v} = \mathbf{713}$ mm

Eccentricity of reaction $e = \bar{x} - l_{base} / 2 = \mathbf{-187}$ mm

Loaded length of base $l_{load} = l_{base} = \mathbf{1800}$ mm

Bearing pressure at toe $q_{toe} = F_{total\_v} / l_{base} \times (1 - 6 \times e / l_{base}) = \mathbf{91.6}$ kN/m^2

Bearing pressure at heel $q_{heel} = F_{total\_v} / l_{base} \times (1 + 6 \times e / l_{base}) = \mathbf{21.2}$ kN/m^2

Factor of safety $FoS_{bp} = P_{bearing} / \max(q_{toe}, q_{heel}) = \mathbf{1.092}$

PASS - Allowable bearing pressure exceeds maximum applied bearing pressure

RETAINING WALL DESIGN

In accordance with EN1992-1-1:2004 incorporating Corrigendum dated January 2008 and the UK National Annex incorporating National Amendment No.1

Tedds calculation version 2.6.11

Concrete details - Table 3.1 - Strength and deformation characteristics for concrete

Concrete strength class C30/37

Characteristic compressive cylinder strength $f_{ck} = \mathbf{30}$ N/mm^2

Characteristic compressive cube strength $f_{ck,cube} = \mathbf{37}$ N/mm^2

Mean value of compressive cylinder strength $f_{cm} = f_{ck} + 8$ N/mm^2 $= \mathbf{38}$ N/mm^2

Croft Structural Engineers Clock Shop Mews 60 Saxon Road, SE25 5EH 0208 6844744	Project: 1&2 Falkland Mews Section: Basement Calculations Calc. by: CC — Date: 18-Apr-18 — Chk'd by: — Date:	Job Ref.: 171203 Sheet no./rev.: 4 App'd by: — Date:

Mean value of axial tensile strength — $f_{ctm} = 0.3\ N/mm^2 \times (f_{ck} / 1\ N/mm^2)^{2/3} = \mathbf{2.9}\ N/mm^2$

5% fractile of axial tensile strength — $f_{ctk,0.05} = 0.7 \times f_{ctm} = \mathbf{2.0}\ N/mm^2$

Secant modulus of elasticity of concrete — $E_{cm} = 22\ kN/mm^2 \times (f_{cm} / 10\ N/mm^2)^{0.3} = \mathbf{32837}\ N/mm^2$

Partial factor for concrete - Table 2.1N — $\gamma_C = \mathbf{1.50}$

Compressive strength coefficient - cl.3.1.6(1) — $\alpha_{cc} = \mathbf{0.85}$

Design compressive concrete strength - exp.3.15 — $f_{cd} = \alpha_{cc} \times f_{ck} / \gamma_C = \mathbf{17.0}\ N/mm^2$

Maximum aggregate size — $h_{agg} = \mathbf{20}$ mm

Reinforcement details

Characteristic yield strength of reinforcement — $f_{yk} = \mathbf{500}\ N/mm^2$

Modulus of elasticity of reinforcement — $E_s = \mathbf{200000}\ N/mm^2$

Partial factor for reinforcing steel - Table 2.1N — $\gamma_S = \mathbf{1.15}$

Design yield strength of reinforcement — $f_{yd} = f_{yk} / \gamma_S = \mathbf{435}\ N/mm^2$

Cover to reinforcement

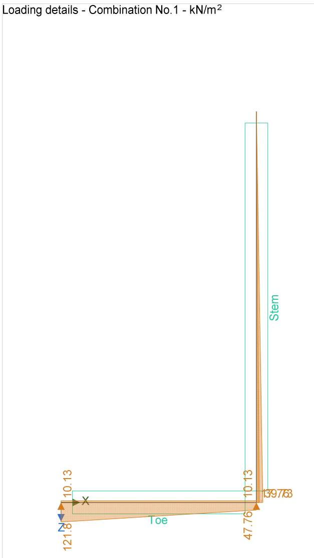
Front face of stem — $c_{sf} = \mathbf{40}$ mm

Rear face of stem — $c_{sr} = \mathbf{50}$ mm

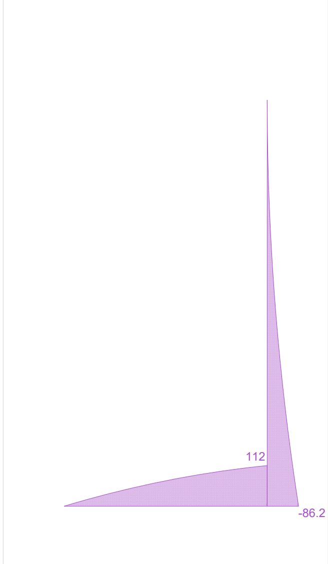
Top face of base — $c_{bt} = \mathbf{50}$ mm

Bottom face of base — $c_{bb} = \mathbf{75}$ mm

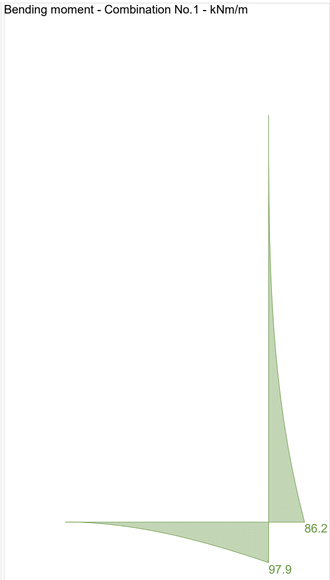
Loading details - Combination No.1 - kN/m²



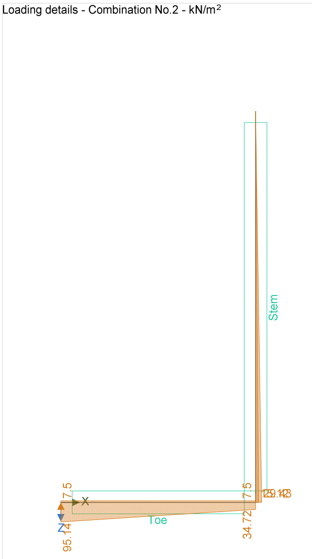
Shear force - Combination No.1 - kN/m



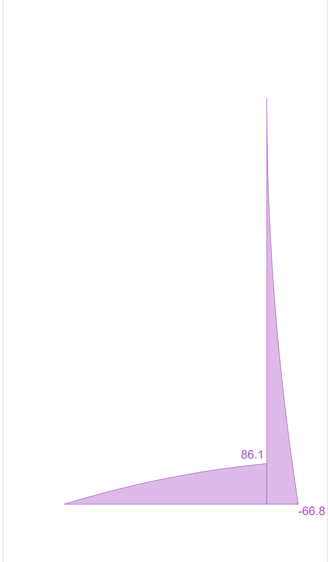
Bending moment - Combination No.1 - kNm/m



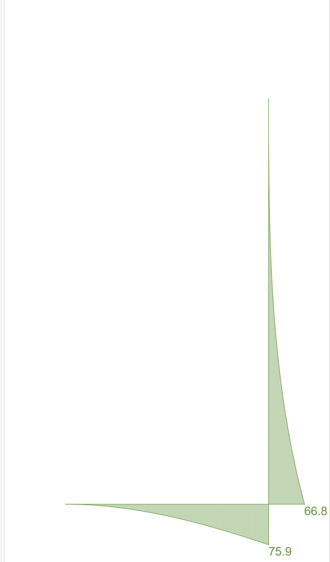
Loading details - Combination No.2 - kN/m²



Shear force - Combination No.2 - kN/m



Bending moment - Combination No.2 - kNm/m



Check stem design at base of stem

Depth of section — h = **300** mm

Rectangular section in flexure - Section 6.1

Design bending moment combination 1 — M = **86.2** kNm/m

Depth to tension reinforcement — $d = h - c_{sr} - \phi_{sr} / 2$ = **242** mm

$K = M / (d^2 \times f_{ck})$ = **0.049**

K' = **0.207**

K' > K - No compression reinforcement is required

Lever arm — $z = \min(0.5 + 0.5 \times (1 - 3.53 \times K)^{0.5}, 0.95) \times d$ = **230** mm

Depth of neutral axis — $x = 2.5 \times (d – z)$ = **30** mm

Area of tension reinforcement required — $A_{sr.req} = M / (f_{yd} \times z)$ = **863** mm²/m

Tension reinforcement provided — 16 dia.bars @ 150 c/c

Area of tension reinforcement provided — $A_{sr.prov} = \pi \times \phi_{sr}^2 / (4 \times s_{sr})$ = **1340** mm²/m

Minimum area of reinforcement - exp.9.1N — $A_{sr.min} = \max(0.26 \times f_{ctm} / f_{yk}, 0.0013) \times d$ = **364** mm²/m

Maximum area of reinforcement - cl.9.2.1.1(3) — $A_{sr.max} = 0.04 \times h$ = **12000** mm²/m

$\max(A_{sr.req}, A_{sr.min}) / A_{sr.prov}$ = **0.644**

PASS - Area of reinforcement provided is greater than area of reinforcement required

Deflection control - Section 7.4

Reference reinforcement ratio — $\rho_0 = \sqrt{(f_{ck} / 1\ N/mm^2)} / 1000$ = **0.005**

Required tension reinforcement ratio — $\rho = A_{sr.req} / d$ = **0.004**

Required compression reinforcement ratio — $\rho' = A_{sr.2.req} / d_2$ = **0.000**

Structural system factor - Table 7.4N — K_b = **0.4**

Reinforcement factor - exp.7.17 — $K_s = \min(500\ N/mm^2 / (f_{yk} \times A_{sr.req} / A_{sr.prov}), 1.5)$ = **1.5**

Limiting span to depth ratio - exp.7.16.a — $K_s \times K_b \times [11 + 1.5 \times \sqrt{(f_{ck} / 1\ N/mm^2)} \times \rho_0 / \rho + 3.2 \times \sqrt{(f_{ck} / 1\ N/mm^2)} \times (\rho_0 / \rho - 1)^{3/2}]$ = **18.3**

Actual span to depth ratio — h_{stem} / d = **12.4**

PASS - Span to depth ratio is less than deflection control limit

Crack control - Section 7.3

Limiting crack width — w_{max} = **0.3** mm

Variable load factor - EN1990 – Table A1.1 — ψ_2 = **0.6**

Serviceability bending moment	M_{sls} = **63.9** kNm/m
Tensile stress in reinforcement	$\sigma_s = M_{sls} / (A_{sr.prov} \times z)$ = **207.3** N/mm^2
Load duration	Long term
Load duration factor	k_t = **0.4**
Effective area of concrete in tension	$A_{c.eff}$ = min(2.5 × (h - d), (h – x) / 3, h / 2) = **89917** mm^2/m
Mean value of concrete tensile strength	$f_{ct.eff} = f_{ctm}$ = **2.9** N/mm^2
Reinforcement ratio	$\rho_{p.eff} = A_{sr.prov} / A_{c.eff}$ = **0.015**
Modular ratio	$\alpha_e = E_s / E_{cm}$ = **6.091**
Bond property coefficient	k_1 = **0.8**
Strain distribution coefficient	k_2 = **0.5**
	k_3 = **3.4**
	k_4 = **0.425**
Maximum crack spacing - exp.7.11	$s_{r.max} = k_3 \times c_{sr} + k_1 \times k_2 \times k_4 \times \phi_{sr} / \rho_{p.eff}$ = **352** mm
Maximum crack width - exp.7.8	$w_k = s_{r.max} \times \max(\sigma_s - k_t \times (f_{ct.eff} / \rho_{p.eff}) \times (1 + \alpha_e \times \rho_{p.eff}), 0.6 \times \sigma_s) / E_s$
	w_k = **0.219** mm
	w_k / w_{max} = **0.731**

PASS - Maximum crack width is less than limiting crack width

Rectangular section in shear - Section 6.2

Design shear force	V = **86.2** kN/m
	$C_{Rd,c} = 0.18 / \gamma_C$ = **0.120**
	k = min(1 + √(200 mm / d), 2) = **1.909**
Longitudinal reinforcement ratio	$\rho_l = \min(A_{sr.prov} / d, 0.02)$ = **0.006**
	$v_{min} = 0.035\ N^{1/2}/mm \times k^{3/2} \times f_{ck}^{0.5}$ = **0.506** N/mm^2
Design shear resistance - exp.6.2a & 6.2b	$V_{Rd.c} = \max(C_{Rd.c} \times k \times (100\ N^2/mm^4 \times \rho_l \times f_{ck})^{1/3}, v_{min}) \times d$
	$V_{Rd.c}$ = **141.5** kN/m
	$V / V_{Rd.c}$ = **0.610**

PASS - Design shear resistance exceeds design shear force

Horizontal reinforcement parallel to face of stem - Section 9.6

Minimum area of reinforcement – cl.9.6.3(1)	$A_{sx.req} = \max(0.25 \times A_{sr.prov}, 0.001 \times t_{stem})$ = **335** mm^2/m
Maximum spacing of reinforcement – cl.9.6.3(2)	s_{sx_max} = **400** mm
Transverse reinforcement provided	10 dia.bars @ 200 c/c
Area of transverse reinforcement provided	$A_{sx.prov} = \pi \times \phi_{sx}^2 / (4 \times s_{sx})$ = **393** mm^2/m

PASS - Area of reinforcement provided is greater than area of reinforcement required

Check base design at toe

Depth of section	h = **300** mm

Rectangular section in flexure - Section 6.1

Design bending moment combination 1	M = **97.9** kNm/m
Depth to tension reinforcement	$d = h - c_{bb} - \phi_{bb} / 2$ = **217** mm
	$K = M / (d^2 \times f_{ck})$ = **0.069**
	K' = **0.207**

K' > K - No compression reinforcement is required

Lever arm	$z = \min(0.5 + 0.5 \times (1 - 3.53 \times K)^{0.5}, 0.95) \times d$ = **203** mm
Depth of neutral axis	x = 2.5 × (d – z) = **35** mm
Area of tension reinforcement required	$A_{bb.req} = M / (f_{yd} \times z)$ = **1110** mm^2/m
Tension reinforcement provided	16 dia.bars @ 100 c/c
Area of tension reinforcement provided	$A_{bb.prov} = \pi \times \phi_{bb}^2 / (4 \times s_{bb})$ = **2011** mm^2/m

Minimum area of reinforcement - exp.9.1N	$A_{bb.min}$ = max(0.26 × f_{ctm} / f_{yk}, 0.0013) × d = **327** mm²/m
Maximum area of reinforcement - cl.9.2.1.1(3)	$A_{bb.max}$ = 0.04 × h = **12000** mm²/m
	max($A_{bb.req}$, $A_{bb.min}$) / $A_{bb.prov}$ = **0.552**

PASS - Area of reinforcement provided is greater than area of reinforcement required

Crack control - Section 7.3

Limiting crack width	w_{max} = **0.3** mm
Variable load factor - EN1990 – Table A1.1	ψ_2 = **0.6**
Serviceability bending moment	M_{sls} = **72.6** kNm/m
Tensile stress in reinforcement	σ_s = M_{sls} / ($A_{bb.prov}$ × z) = **178.1** N/mm²
Load duration	Long term
Load duration factor	k_t = **0.4**
Effective area of concrete in tension	$A_{c.eff}$ = min(2.5 × (h - d), (h – x) / 3, h / 2) = **88170** mm²/m
Mean value of concrete tensile strength	$f_{ct.eff}$ = f_{ctm} = **2.9** N/mm²
Reinforcement ratio	$\rho_{p.eff}$ = $A_{bb.prov}$ / $A_{c.eff}$ = **0.023**
Modular ratio	α_e = E_s / E_{cm} = **6.091**
Bond property coefficient	k_1 = **0.8**
Strain distribution coefficient	k_2 = **0.5**
	k_3 = **3.4**
	k_4 = **0.425**
Maximum crack spacing - exp.7.11	$s_{r.max}$ = k_3 × c_{bb} + k_1 × k_2 × k_4 × ϕ_{bb} / $\rho_{p.eff}$ = **374** mm
Maximum crack width - exp.7.8	w_k = $s_{r.max}$ × max(σ_s – k_t × ($f_{ct.eff}$ / $\rho_{p.eff}$) × (1 + α_e × $\rho_{p.eff}$), 0.6 × σ_s) / E_s
	w_k = **0.225** mm
	w_k / w_{max} = **0.75**

PASS - Maximum crack width is less than limiting crack width

Rectangular section in shear - Section 6.2

Design shear force	V = **112** kN/m
	$C_{Rd,c}$ = 0.18 / γ_C = **0.120**
	k = min(1 + √(200 mm / d), 2) = **1.960**
Longitudinal reinforcement ratio	ρ_l = min($A_{bb.prov}$ / d, 0.02) = **0.009**
	v_{min} = 0.035 $N^{1/2}$/mm × $k^{3/2}$ × $f_{ck}^{0.5}$ = **0.526** N/mm²
Design shear resistance - exp.6.2a & 6.2b	$V_{Rd.c}$ = max($C_{Rd.c}$ × k × (100 N²/mm⁴ × ρ_l × f_{ck})$^{1/3}$, v_{min}) × d
	$V_{Rd.c}$ = **154.6** kN/m
	V / $V_{Rd.c}$ = **0.724**

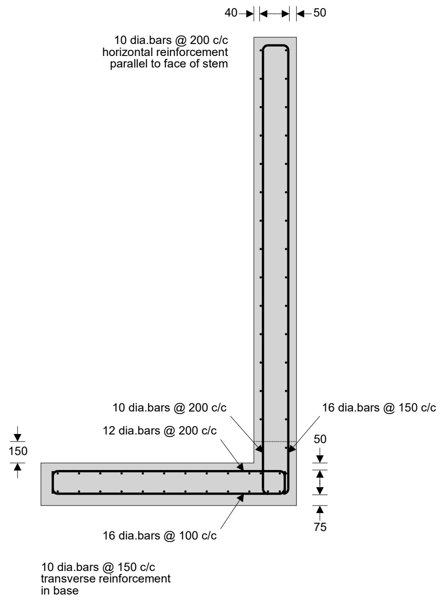
PASS - Design shear resistance exceeds design shear force

Secondary transverse reinforcement to base - Section 9.3

Minimum area of reinforcement – cl.9.3.1.1(2)	$A_{bx.req}$ = 0.2 × $A_{bb.prov}$ = **402** mm²/m
Maximum spacing of reinforcement – cl.9.3.1.1(3)	s_{bx_max} = **450** mm
Transverse reinforcement provided	10 dia.bars @ 150 c/c
Area of transverse reinforcement provided	$A_{bx.prov}$ = π × ϕ_{bx}^2 / (4 × s_{bx}) = **524** mm²/m

PASS - Area of reinforcement provided is greater than area of reinforcement required

Croft Structural Engineers Clock Shop Mews 60 Saxon Road, SE25 5EH 0208 6844744	Project 1&2 Falkland Mews			Job Ref. 171203	
	Section Basement Calculations			Sheet no./rev. 8	
	Calc. by CC	Date 18-Apr-18	Chk'd by	Date	App'd by	Date



Reinforcement details

W1 - RC WALL - TEMPORARY CONDITION

Location	Area			Type	L	Action	Actions, kN or kN/m			
	L	W	m^2			kN/m^2	Perm., g_k	%	Var., q_k	Total
W1 - RC Wall										
Ground floor*2	2	2	4	g_k		5.00	20.0			
				q_k		1.50			6.0	
First floor*2	2	2	4	g_k		0.63	2.5			
				q_k		1.50			6.0	
External wall	6	1	6	g_k		3.98	23.9			
Roof	2	2	4	g_k		1.03	4.1			
				q_k		0.75			3.0	
							50.5	kN/m	15.0	kN/m

RETAINING WALL ANALYSIS

In accordance with EN1997-1:2004 incorporating Corrigendum dated February 2009 and the UK National Annex incorporating Corrigendum No.1

Tedds calculation version 2.6.11

Retaining wall details

Stem type	Propped cantilever
Stem height	h_{stem} = **3000** mm
Prop height	h_{prop} = **3000** mm
Stem thickness	t_{stem} = **300** mm
Angle to rear face of stem	α = **90** deg
Stem density	γ_{stem} = **25** kN/m^3
Toe length	l_{toe} = **1500** mm
Base thickness	t_{base} = **300** mm
Base density	γ_{base} = **25** kN/m^3
Height of retained soil	h_{ret} = **3000** mm
Angle of soil surface	β = **0** deg
Depth of cover	d_{cover} = **0** mm

Retained soil properties

Soil type	Stiff clay
Moist density	γ_{mr} = **19** kN/m^3
Saturated density	γ_{sr} = **19** kN/m^3
Characteristic effective shear resistance angle	$\phi'_{r.k}$ = **18** deg
Characteristic wall friction angle	$\delta_{r.k}$ = **9** deg

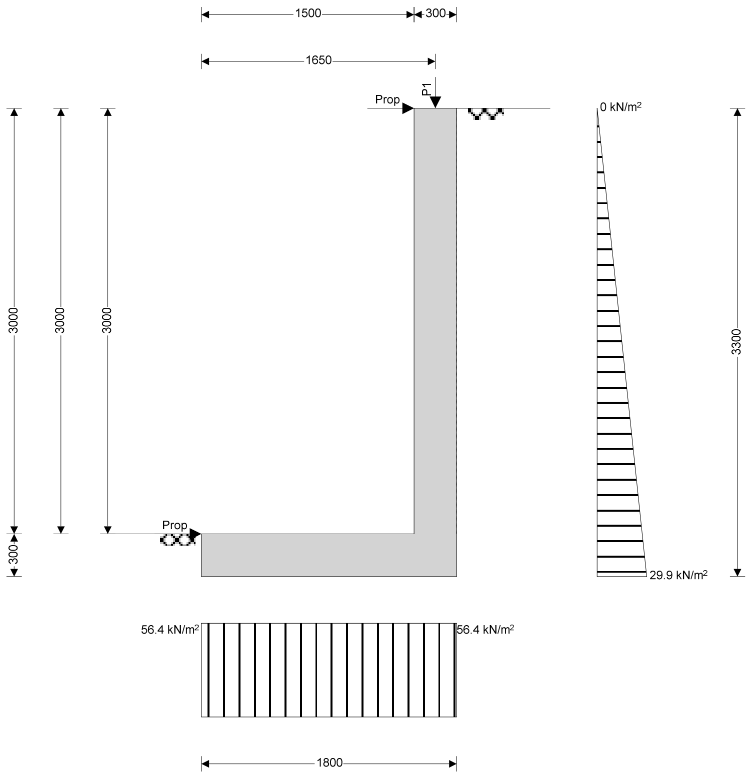
Base soil properties

Soil type	Stiff clay
Soil density	γ_b = **19** kN/m^3
Characteristic effective shear resistance angle	$\phi'_{b.k}$ = **18** deg
Characteristic wall friction angle	$\delta_{b.k}$ = **9** deg
Characteristic base friction angle	$\delta_{bb.k}$ = **12** deg
Presumed bearing capacity	$P_{bearing}$ = **100** kN/m^2

Loading details

Vertical line load at 1650 mm — P_{G1} = **50.5** kN/m

P_{Q1} = **15** kN/m



General arrangement

Calculate retaining wall geometry

Base length — $l_{base} = l_{toe} + t_{stem}$ = **1800** mm

Moist soil height — $h_{moist} = h_{soil}$ = **3000** mm

Retained surface length — $l_{sur} = l_{heel}$ = **0** mm

Effective height of wall — $h_{eff} = h_{base} + d_{cover} + h_{ret}$ = **3300** mm

Area of wall stem — $A_{stem} = h_{stem} \times t_{stem}$ = **0.9** m²

- Distance to vertical component — $x_{stem} = l_{toe} + t_{stem} / 2$ = **1650** mm

Area of wall base — $A_{base} = l_{base} \times t_{base}$ = **0.54** m²

- Distance to vertical component — $x_{base} = l_{base} / 2$ = **900** mm

Using Coulomb theory

Active pressure coefficient — $K_A = \sin(\alpha + \phi'_{r.k})^2 / (\sin(\alpha)^2 \times \sin(\alpha - \delta_{r.k}) \times [1 + \sqrt{[\sin(\phi'_{r.k} + \delta_{r.k}) \times \sin(\phi'_{r.k} - \beta) / (\sin(\alpha - \delta_{r.k}) \times \sin(\alpha + \beta))]}]^2)$ = **0.483**

Passive pressure coefficient — $K_P = \sin(90 - \phi'_{b.k})^2 / (\sin(90 + \delta_{b.k}) \times [1 - \sqrt{[\sin(\phi'_{b.k} + \delta_{b.k}) \times \sin(\phi'_{b.k}) / (\sin(90 + \delta_{b.k}))]}]^2)$ = **2.359**

Bearing pressure check

Vertical forces on wall

Wall stem — $F_{stem} = A_{stem} \times \gamma_{stem}$ = **22.5** kN/m

Wall base — $F_{base} = A_{base} \times \gamma_{base}$ = **13.5** kN/m

Line loads — $F_{P\_v} = P_{G1} + P_{Q1}$ = **65.5** kN/m

Total $F_{total\_v} = F_{stem} + F_{base} + F_{P\_v} = \mathbf{101.5}$ kN/m

Horizontal forces on wall

Moist retained soil $F_{moist\_h} = K_A \times \cos(\delta_{r.d}) \times \gamma_{mr}' \times h_{eff}^2 / 2 = \mathbf{49.4}$ kN/m

Base soil $F_{pass\_h} = -K_P \times \cos(\delta_{b.d}) \times \gamma_b' \times (d_{cover} + h_{base})^2 / 2 = \mathbf{-2}$ kN/m

Total $F_{total\_h} = F_{moist\_h} + F_{pass\_h} = \mathbf{47.4}$ kN/m

Moments on wall

Wall stem $M_{stem} = F_{stem} \times x_{stem} = \mathbf{37.1}$ kNm/m

Wall base $M_{base} = F_{base} \times x_{base} = \mathbf{12.2}$ kNm/m

Line loads $M_P = (P_{G1} + P_{Q1}) \times p_1 = \mathbf{108.1}$ kNm/m

Moist retained soil $M_{moist} = -F_{moist\_h} \times x_{moist\_h} = \mathbf{-54.3}$ kNm/m

Total $M_{total} = M_{stem} + M_{base} + M_{moist} + M_P = \mathbf{103.1}$ kNm/m

Check bearing pressure

Propping force to stem $F_{prop\_stem} = (F_{total\_v} \times l_{base} / 2 - M_{total}) / (h_{prop} + t_{base}) = \mathbf{-3.5}$ kN/m

Propping force to base $F_{prop\_base} = F_{total\_h} - F_{prop\_stem} = \mathbf{50.9}$ kN/m

Moment from propping force $M_{prop} = F_{prop\_stem} \times (h_{prop} + t_{base}) = \mathbf{-11.7}$ kNm/m

Distance to reaction $\bar{x} = (M_{total} + M_{prop}) / F_{total\_v} = \mathbf{900}$ mm

Eccentricity of reaction $e = \bar{x} - l_{base} / 2 = \mathbf{0}$ mm

Loaded length of base $l_{load} = l_{base} = \mathbf{1800}$ mm

Bearing pressure at toe $q_{toe} = F_{total\_v} / l_{base} \times (1 - 6 \times e / l_{base}) = \mathbf{56.4}$ kN/m^2

Bearing pressure at heel $q_{heel} = F_{total\_v} / l_{base} \times (1 + 6 \times e / l_{base}) = \mathbf{56.4}$ kN/m^2

Factor of safety $FoS_{bp} = P_{bearing} / \max(q_{toe}, q_{heel}) = \mathbf{1.773}$

PASS - Allowable bearing pressure exceeds maximum applied bearing pressure

RETAINING WALL DESIGN

In accordance with EN1992-1-1:2004 incorporating Corrigendum dated January 2008 and the UK National Annex incorporating National Amendment No.1

Tedds calculation version 2.6.11

Concrete details - Table 3.1 - Strength and deformation characteristics for concrete

Concrete strength class C30/37

Characteristic compressive cylinder strength $f_{ck} = \mathbf{30}$ N/mm^2

Characteristic compressive cube strength $f_{ck,cube} = \mathbf{37}$ N/mm^2

Mean value of compressive cylinder strength $f_{cm} = f_{ck} + 8$ N/mm^2 = **38** N/mm^2

Mean value of axial tensile strength $f_{ctm} = 0.3$ N/mm$^2 \times (f_{ck} / 1$ N/mm$^2)^{2/3}$ = **2.9** N/mm^2

5% fractile of axial tensile strength $f_{ctk,0.05} = 0.7 \times f_{ctm} = \mathbf{2.0}$ N/mm^2

Secant modulus of elasticity of concrete $E_{cm} = 22$ kN/mm$^2 \times (f_{cm} / 10$ N/mm$^2)^{0.3}$ = **32837** N/mm^2

Partial factor for concrete - Table 2.1N $\gamma_C = \mathbf{1.50}$

Compressive strength coefficient - cl.3.1.6(1) $\alpha_{cc} = \mathbf{0.85}$

Design compressive concrete strength - exp.3.15 $f_{cd} = \alpha_{cc} \times f_{ck} / \gamma_C = \mathbf{17.0}$ N/mm^2

Maximum aggregate size $h_{agg} = \mathbf{20}$ mm

Reinforcement details

Characteristic yield strength of reinforcement $f_{yk} = \mathbf{500}$ N/mm^2

Modulus of elasticity of reinforcement $E_s = \mathbf{200000}$ N/mm^2

Partial factor for reinforcing steel - Table 2.1N $\gamma_S = \mathbf{1.15}$

Design yield strength of reinforcement $f_{yd} = f_{yk} / \gamma_S = \mathbf{435}$ N/mm^2

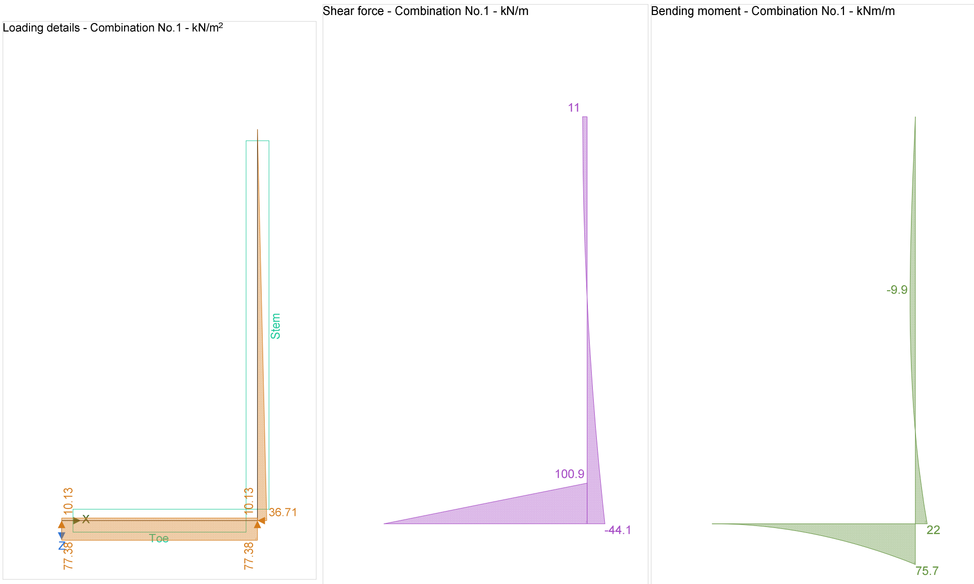
Cover to reinforcement

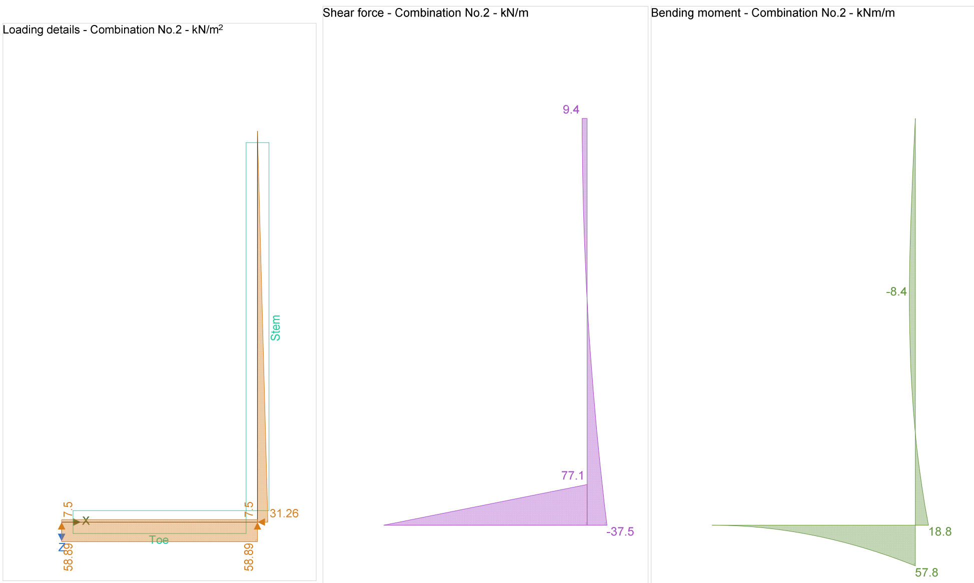
Front face of stem $c_{sf} = \mathbf{40}$ mm

Rear face of stem c_{sr} = **50** mm

Top face of base c_{bt} = **50** mm

Bottom face of base c_{bb} = **75** mm





Check stem design at 1658 mm

Depth of section h = **300** mm

Rectangular section in flexure - Section 6.1

Design bending moment combination 1 M = **9.9** kNm/m

Depth to tension reinforcement $d = h - c_{sf} - \phi_{sx} - \phi_{sfM} / 2$ = **245** mm

$K = M / (d^2 \times f_{ck})$ = **0.005**

K' = **0.207**

K' > K - No compression reinforcement is required

Lever arm $z = \min(0.5 + 0.5 \times (1 - 3.53 \times K)^{0.5}, 0.95) \times d$ = **233** mm

Depth of neutral axis — $x = 2.5 \times (d - z) = \mathbf{31}$ mm

Area of tension reinforcement required — $A_{sfM.req} = M / (f_{yd} \times z) = \mathbf{97}$ mm^2/m

Tension reinforcement provided — 10 dia.bars @ 200 c/c

Area of tension reinforcement provided — $A_{sfM.prov} = \pi \times \phi_{sfM}^2 / (4 \times s_{sfM}) = \mathbf{393}$ mm^2/m

Minimum area of reinforcement - exp.9.1N — $A_{sfM.min} = \max(0.26 \times f_{ctm} / f_{yk}, 0.0013) \times d = \mathbf{369}$ mm^2/m

Maximum area of reinforcement - cl.9.2.1.1(3) — $A_{sfM.max} = 0.04 \times h = \mathbf{12000}$ mm^2/m

$\max(A_{sfM.req}, A_{sfM.min}) / A_{sfM.prov} = \mathbf{0.94}$

PASS - Area of reinforcement provided is greater than area of reinforcement required

Deflection control - Section 7.4

Reference reinforcement ratio — $\rho_0 = \sqrt{(f_{ck} / 1\ N/mm^2)} / 1000 = \mathbf{0.005}$

Required tension reinforcement ratio — $\rho = A_{sfM.req} / d = \mathbf{0.000}$

Required compression reinforcement ratio — $\rho' = A_{sfM.2.req} / d_2 = \mathbf{0.000}$

Structural system factor - Table 7.4N — $K_b = \mathbf{1}$

Reinforcement factor - exp.7.17 — $K_s = \min(500\ N/mm^2 / (f_{yk} \times A_{sfM.req} / A_{sfM.prov}), 1.5) = \mathbf{1.5}$

Limiting span to depth ratio - exp.7.16.a — $K_s \times K_b \times [11 + 1.5 \times \sqrt{(f_{ck} / 1\ N/mm^2)} \times \rho_0 / \rho + 3.2 \times \sqrt{(f_{ck} / 1\ N/mm^2)} \times (\rho_0 / \rho - 1)^{3/2}] = \mathbf{1388.2}$

Actual span to depth ratio — $h_{prop} / d = \mathbf{12.2}$

PASS - Span to depth ratio is less than deflection control limit

Crack control - Section 7.3

Limiting crack width — $w_{max} = \mathbf{0.3}$ mm

Variable load factor - EN1990 – Table A1.1 — $\psi_2 = \mathbf{0.6}$

Serviceability bending moment — $M_{sls} = \mathbf{7.3}$ kNm/m

Tensile stress in reinforcement — $\sigma_s = M_{sls} / (A_{sfM.prov} \times z) = \mathbf{79.8}$ N/mm^2

Load duration — Long term

Load duration factor — $k_t = \mathbf{0.4}$

Effective area of concrete in tension — $A_{c.eff} = \min(2.5 \times (h - d), (h - x) / 3, h / 2) = \mathbf{89792}$ mm^2/m

Mean value of concrete tensile strength — $f_{ct.eff} = f_{ctm} = \mathbf{2.9}$ N/mm^2

Reinforcement ratio — $\rho_{p.eff} = A_{sfM.prov} / A_{c.eff} = \mathbf{0.004}$

Modular ratio — $\alpha_e = E_s / E_{cm} = \mathbf{6.091}$

Bond property coefficient — $k_1 = \mathbf{0.8}$

Strain distribution coefficient — $k_2 = \mathbf{0.5}$

$k_3 = \mathbf{3.4}$

$k_4 = \mathbf{0.425}$

Maximum crack spacing - exp.7.11 — $s_{r.max} = k_3 \times c_{sf} + k_1 \times k_2 \times k_4 \times \phi_{sfM} / \rho_{p.eff} = \mathbf{525}$ mm

Maximum crack width - exp.7.8 — $w_k = s_{r.max} \times \max(\sigma_s - k_t \times (f_{ct.eff} / \rho_{p.eff}) \times (1 + \alpha_e \times \rho_{p.eff}), 0.6 \times \sigma_s) / E_s$

$w_k = \mathbf{0.126}$ mm

$w_k / w_{max} = \mathbf{0.419}$

PASS - Maximum crack width is less than limiting crack width

Check stem design at base of stem

Depth of section — $h = \mathbf{300}$ mm

Rectangular section in flexure - Section 6.1

Design bending moment combination 1 — $M = \mathbf{22}$ kNm/m

Depth to tension reinforcement — $d = h - c_{sr} - \phi_{sr} / 2 = \mathbf{244}$ mm

$K = M / (d^2 \times f_{ck}) = \mathbf{0.012}$

$K' = \mathbf{0.207}$

K' > K - No compression reinforcement is required

Lever arm	$z = \min(0.5 + 0.5 \times (1 - 3.53 \times K)^{0.5}, 0.95) \times d =$ **232** mm
Depth of neutral axis	$x = 2.5 \times (d - z) =$ **31** mm
Area of tension reinforcement required	$A_{sr.req} = M / (f_{yd} \times z) =$ **219** mm^2/m
Tension reinforcement provided	12 dia.bars @ 200 c/c
Area of tension reinforcement provided	$A_{sr.prov} = \pi \times \phi_{sr}^2 / (4 \times s_{sr}) =$ **565** mm^2/m
Minimum area of reinforcement - exp.9.1N	$A_{sr.min} = \max(0.26 \times f_{ctm} / f_{yk}, 0.0013) \times d =$ **368** mm^2/m
Maximum area of reinforcement - cl.9.2.1.1(3)	$A_{sr.max} = 0.04 \times h =$ **12000** mm^2/m
	$\max(A_{sr.req}, A_{sr.min}) / A_{sr.prov} =$ **0.65**

PASS - Area of reinforcement provided is greater than area of reinforcement required

Deflection control - Section 7.4

Reference reinforcement ratio	$\rho_0 = \sqrt{(f_{ck} / 1\ N/mm^2)} / 1000 =$ **0.005**
Required tension reinforcement ratio	$\rho = A_{sr.req} / d =$ **0.001**
Required compression reinforcement ratio	$\rho' = A_{sr.2.req} / d_2 =$ **0.000**
Structural system factor - Table 7.4N	$K_b =$ **1**
Reinforcement factor - exp.7.17	$K_s = \min(500\ N/mm^2 / (f_{yk} \times A_{sr.req} / A_{sr.prov}), 1.5) =$ **1.5**
Limiting span to depth ratio - exp.7.16.a	$K_s \times K_b \times [11 + 1.5 \times \sqrt{(f_{ck} / 1\ N/mm^2)} \times \rho_0 / \rho + 3.2 \times \sqrt{(f_{ck} / 1\ N/mm^2)} \times (\rho_0 / \rho - 1)^{3/2}] =$ **395.9**
Actual span to depth ratio	$h_{prop} / d =$ **12.3**

PASS - Span to depth ratio is less than deflection control limit

Crack control - Section 7.3

Limiting crack width	$w_{max} =$ **0.3** mm
Variable load factor - EN1990 – Table A1.1	$\psi_2 =$ **0.6**
Serviceability bending moment	$M_{sls} =$ **16.3** kNm/m
Tensile stress in reinforcement	$\sigma_s = M_{sls} / (A_{sr.prov} \times z) =$ **124.5** N/mm^2
Load duration	Long term
Load duration factor	$k_t =$ **0.4**
Effective area of concrete in tension	$A_{c.eff} = \min(2.5 \times (h - d), (h - x) / 3, h / 2) =$ **89833** mm^2/m
Mean value of concrete tensile strength	$f_{ct.eff} = f_{ctm} =$ **2.9** N/mm^2
Reinforcement ratio	$\rho_{p.eff} = A_{sr.prov} / A_{c.eff} =$ **0.006**
Modular ratio	$\alpha_e = E_s / E_{cm} =$ **6.091**
Bond property coefficient	$k_1 =$ **0.8**
Strain distribution coefficient	$k_2 =$ **0.5**
	$k_3 =$ **3.4**
	$k_4 =$ **0.425**
Maximum crack spacing - exp.7.11	$s_{r.max} = k_3 \times c_{sr} + k_1 \times k_2 \times k_4 \times \phi_{sr} / \rho_{p.eff} =$ **494** mm
Maximum crack width - exp.7.8	$w_k = s_{r.max} \times \max(\sigma_s - k_t \times (f_{ct.eff} / \rho_{p.eff}) \times (1 + \alpha_e \times \rho_{p.eff}), 0.6 \times \sigma_s) / E_s$
	$w_k =$ **0.185** mm
	$w_k / w_{max} =$ **0.615**

PASS - Maximum crack width is less than limiting crack width

Rectangular section in shear - Section 6.2

Design shear force	$V =$ **44.1** kN/m
	$C_{Rd,c} = 0.18 / \gamma_C =$ **0.120**
	$k = \min(1 + \sqrt{(200\ mm / d)}, 2) =$ **1.905**
Longitudinal reinforcement ratio	$\rho_l = \min(A_{sr.prov} / d, 0.02) =$ **0.002**
	$v_{min} = 0.035\ N^{1/2}/mm \times k^{3/2} \times f_{ck}^{0.5} =$ **0.504** N/mm^2
Design shear resistance - exp.6.2a & 6.2b	$V_{Rd.c} = \max(C_{Rd.c} \times k \times (100\ N^2/mm^4 \times \rho_l \times f_{ck})^{1/3}, v_{min}) \times d$

	$V_{Rd.c}$ = **123** kN/m
	$V / V_{Rd.c}$ = **0.358**

PASS - Design shear resistance exceeds design shear force

Check stem design at prop

Depth of section	h = **300** mm

Rectangular section in shear - Section 6.2

Design shear force	V = **11** kN/m
	$C_{Rd,c} = 0.18 / \gamma_C$ = **0.120**
	$k = \min(1 + \sqrt{(200 \text{ mm} / d)}, 2)$ = **1.905**
Longitudinal reinforcement ratio	$\rho_l = \min(A_{sr1.prov} / d, 0.02)$ = **0.002**
	$v_{min} = 0.035 \text{ N}^{1/2}/\text{mm} \times k^{3/2} \times f_{ck}^{0.5}$ = **0.504** N/mm^2
Design shear resistance - exp.6.2a & 6.2b	$V_{Rd.c} = \max(C_{Rd.c} \times k \times (100 \text{ N}^2/\text{mm}^4 \times \rho_l \times f_{ck})^{1/3}, v_{min}) \times d$
	$V_{Rd.c}$ = **123** kN/m
	$V / V_{Rd.c}$ = **0.090**

PASS - Design shear resistance exceeds design shear force

Horizontal reinforcement parallel to face of stem - Section 9.6

Minimum area of reinforcement – cl.9.6.3(1)	$A_{sx.req} = \max(0.25 \times A_{sr.prov}, 0.001 \times t_{stem})$ = **300** mm^2/m
Maximum spacing of reinforcement – cl.9.6.3(2)	s_{sx_max} = **400** mm
Transverse reinforcement provided	10 dia.bars @ 200 c/c
Area of transverse reinforcement provided	$A_{sx.prov} = \pi \times \phi_{sx}^2 / (4 \times s_{sx})$ = **393** mm^2/m

PASS - Area of reinforcement provided is greater than area of reinforcement required

Check base design at toe

Depth of section	h = **300** mm

Rectangular section in flexure - Section 6.1

Design bending moment combination 1	M = **75.7** kNm/m
Depth to tension reinforcement	$d = h - c_{bb} - \phi_{bb} / 2$ = **217** mm
	$K = M / (d^2 \times f_{ck})$ = **0.054**
	K' = **0.207**

K' > K - No compression reinforcement is required

Lever arm	$z = \min(0.5 + 0.5 \times (1 - 3.53 \times K)^{0.5}, 0.95) \times d$ = **206** mm
Depth of neutral axis	$x = 2.5 \times (d - z)$ = **27** mm
Area of tension reinforcement required	$A_{bb.req} = M / (f_{yd} \times z)$ = **844** mm^2/m
Tension reinforcement provided	16 dia.bars @ 150 c/c
Area of tension reinforcement provided	$A_{bb.prov} = \pi \times \phi_{bb}^2 / (4 \times s_{bb})$ = **1340** mm^2/m
Minimum area of reinforcement - exp.9.1N	$A_{bb.min} = \max(0.26 \times f_{ctm} / f_{yk}, 0.0013) \times d$ = **327** mm^2/m
Maximum area of reinforcement - cl.9.2.1.1(3)	$A_{bb.max} = 0.04 \times h$ = **12000** mm^2/m
	$\max(A_{bb.req}, A_{bb.min}) / A_{bb.prov}$ = **0.63**

PASS - Area of reinforcement provided is greater than area of reinforcement required

Crack control - Section 7.3

Limiting crack width	w_{max} = **0.3** mm
Variable load factor - EN1990 – Table A1.1	ψ_2 = **0.6**
Serviceability bending moment	M_{sls} = **55** kNm/m
Tensile stress in reinforcement	$\sigma_s = M_{sls} / (A_{bb.prov} \times z)$ = **199** N/mm^2
Load duration	Long term
Load duration factor	k_t = **0.4**
Effective area of concrete in tension	$A_{c.eff} = \min(2.5 \times (h - d), (h - x) / 3, h / 2)$ = **90958** mm^2/m

Mean value of concrete tensile strength	$f_{ct.eff} = f_{ctm} =$ **2.9** N/mm^2
Reinforcement ratio	$\rho_{p.eff} = A_{bb.prov} / A_{c.eff} =$ **0.015**
Modular ratio	$\alpha_e = E_s / E_{cm} =$ **6.091**
Bond property coefficient	$k_1 =$ **0.8**
Strain distribution coefficient	$k_2 =$ **0.5**
	$k_3 =$ **3.4**
	$k_4 =$ **0.425**
Maximum crack spacing - exp.7.11	$s_{r.max} = k_3 \times c_{bb} + k_1 \times k_2 \times k_4 \times \phi_{bb} / \rho_{p.eff} =$ **440** mm
Maximum crack width - exp.7.8	$w_k = s_{r.max} \times \max(\sigma_s - k_t \times (f_{ct.eff} / \rho_{p.eff}) \times (1 + \alpha_e \times \rho_{p.eff}), 0.6 \times \sigma_s) / E_s$
	$w_k =$ **0.262** mm
	$w_k / w_{max} =$ **0.875**

PASS - Maximum crack width is less than limiting crack width

Rectangular section in shear - Section 6.2

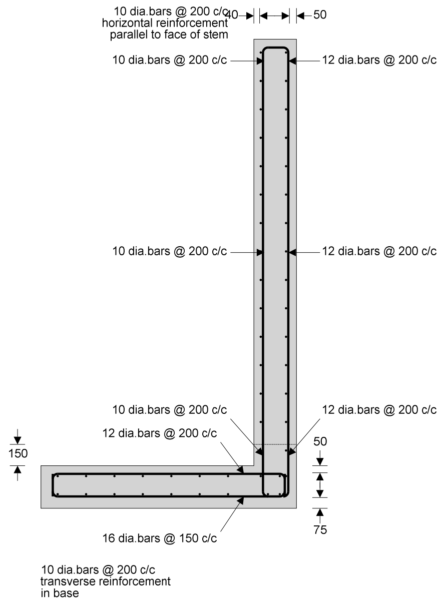
Design shear force	V = **100.9** kN/m
	$C_{Rd,c} = 0.18 / \gamma_C =$ **0.120**
	$k = \min(1 + \sqrt{(200 \text{ mm} / d)}, 2) =$ **1.960**
Longitudinal reinforcement ratio	$\rho_l = \min(A_{bb.prov} / d, 0.02) =$ **0.006**
	$v_{min} = 0.035 N^{1/2}/mm \times k^{3/2} \times f_{ck}^{0.5} =$ **0.526** N/mm^2
Design shear resistance - exp.6.2a & 6.2b	$V_{Rd.c} = \max(C_{Rd.c} \times k \times (100 N^2/mm^4 \times \rho_l \times f_{ck})^{1/3}, v_{min}) \times d$
	$V_{Rd.c} =$ **135.1** kN/m
	$V / V_{Rd.c} =$ **0.747**

PASS - Design shear resistance exceeds design shear force

Secondary transverse reinforcement to base - Section 9.3

Minimum area of reinforcement – cl.9.3.1.1(2)	$A_{bx.req} = 0.2 \times A_{bb.prov} =$ **268** mm^2/m
Maximum spacing of reinforcement – cl.9.3.1.1(3)	$s_{bx\_max} =$ **450** mm
Transverse reinforcement provided	10 dia.bars @ 200 c/c
Area of transverse reinforcement provided	$A_{bx.prov} = \pi \times \phi_{bx}^2 / (4 \times s_{bx}) =$ **393** mm^2/m

PASS - Area of reinforcement provided is greater than area of reinforcement required



Reinforcement details

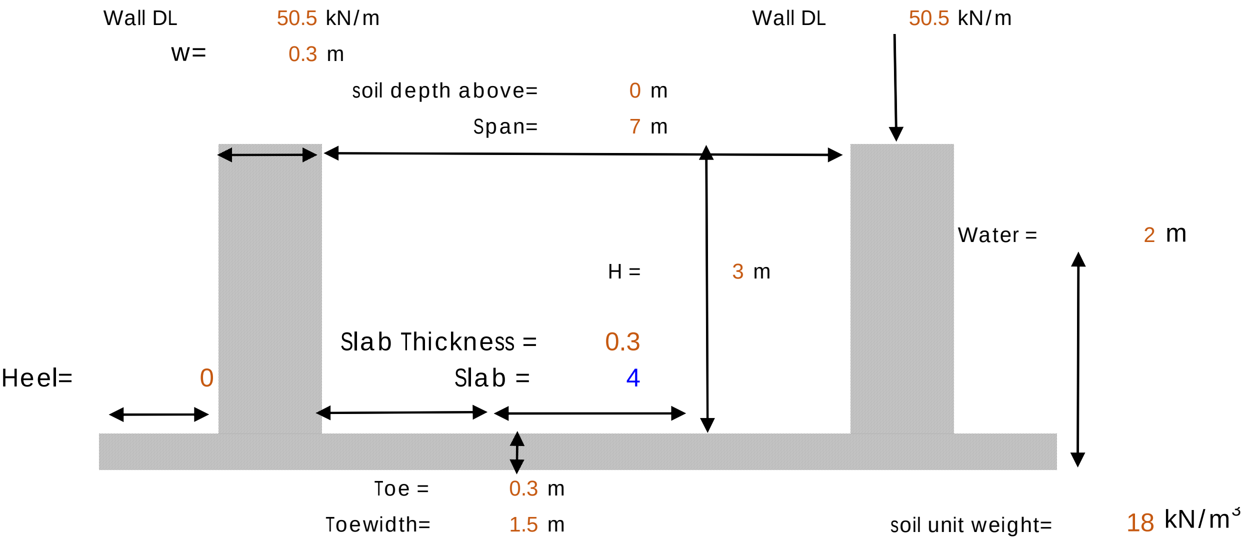
CROFT STRUCTURAL ENGINEERS

CROFT STRUCTURAL ENGINEERS
Clock Shop Mews
Rear of 60 Saxon Road
London SE25 5EH
T: 020 8684 4744
W: www.croftse.co.uk

Project: 1&2 Falkland Mews		Section	Sheet
Date Feb-18 By CC Checked CT	Rev Date Description		
Job No 171008	Status		Rev

Ref

Slab Uplift



Uplift Calc

Total Dead Load =	Slab=	30 kN/m					
	Toe and heel =	27 kN/m					
	Wall =	45					
	Soil=(	0 +	0) x 2 +	0 =		0	14
	Total Dead load =	203 kN/m					
Total Uplift Force=		152 kN/m	f.o.s.=	**1.34 No Global Uplift**			

Slab Uplift

Slab =	7.5 kN/m	Uplift =	20
Service Moment =	-76.5625 kNm/m		
Factored Design moment=	-93.4063 kNm/m		
Factored Design shear =	-53.375 kN/m	*At detailed design stage, apply to design of slab*	

Global Heave

Weight of building =	203 kN/m		
Weight of soil removed =	410.4		
% change =	51%	place	51% of Slab area as heave protection
width of heave protection =	3.840741 m	place	3.84 m of Slab area as heave protection

Provide Clayboard or similar (final details TBA at detailed design stage)

CROFT STRUCTURAL ENGINEERS

Appendix B: Construction Programme

<u>The Contractor is responsible for the final construction programme</u>

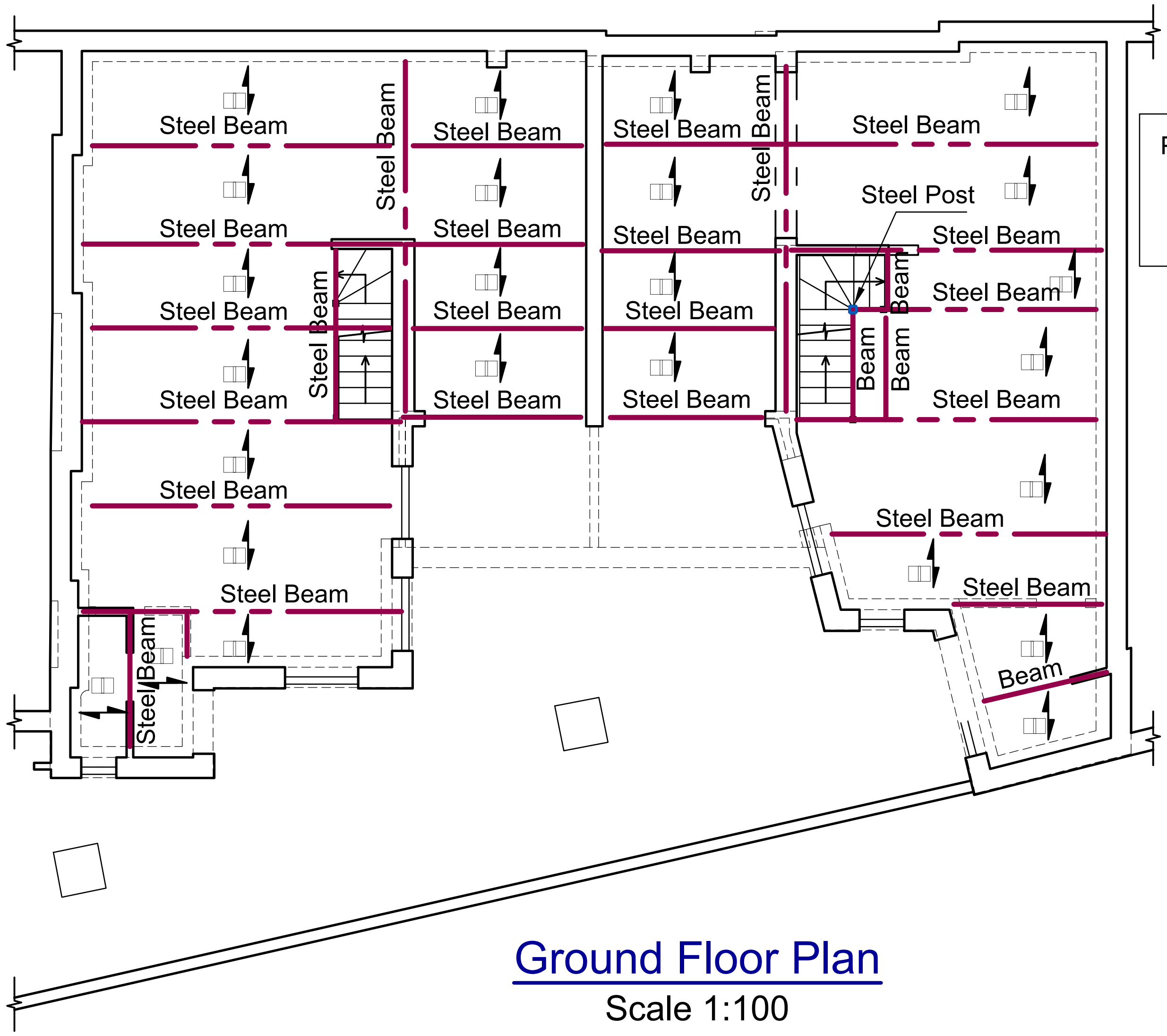
Outline construction Programme

(For planning purposes only)

	Months															
	1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16
Planning approval																
Detailed Design																
Tender																
Party Walls																
Monitoring of Adjacent structures																
Enabling works																
Basement Construction																
Superstructure construction																

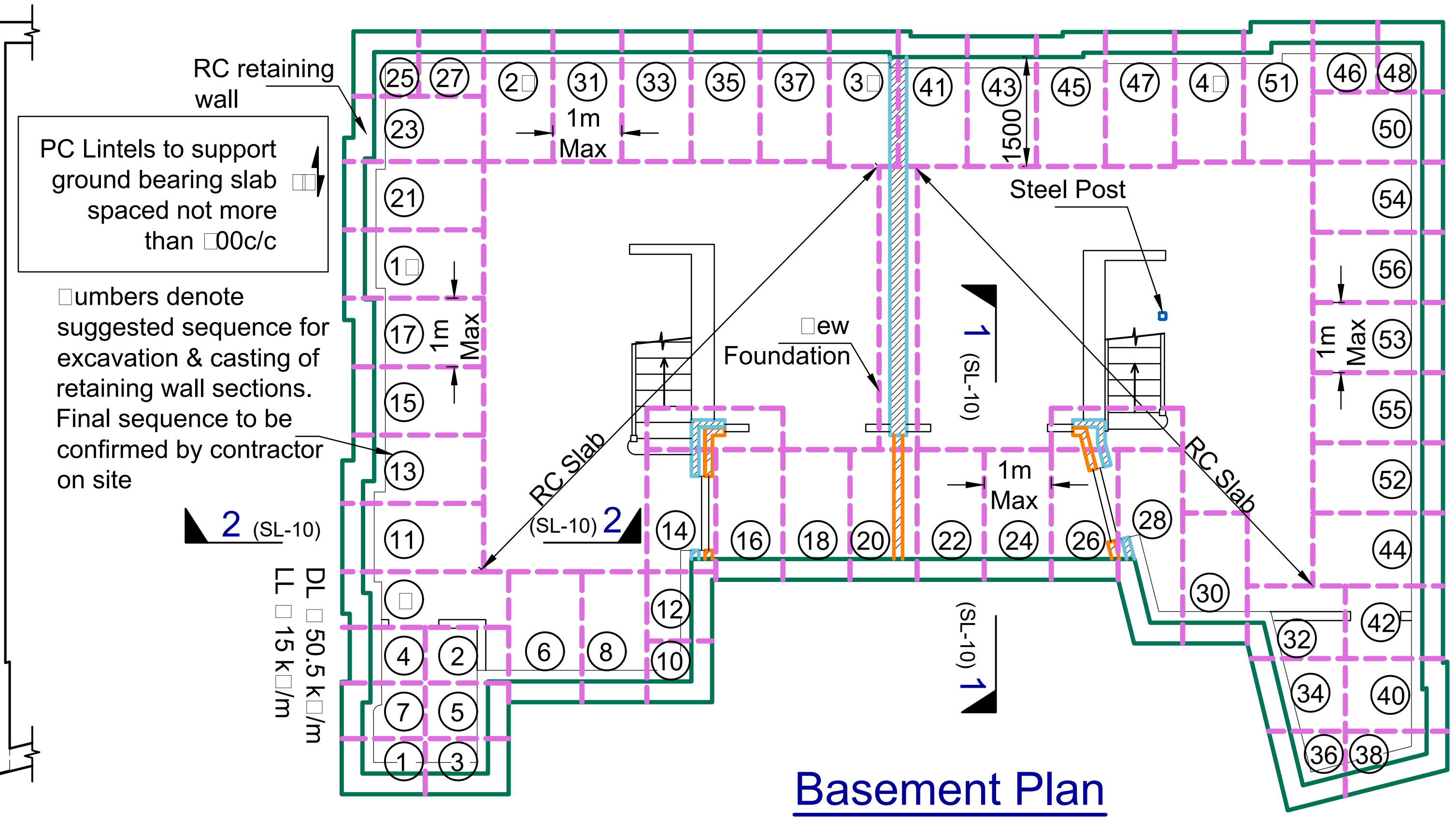
Appendix C: Structural Drawings

1:100 Basement Plan
1:100 Ground Floor plan
1:50 Sections



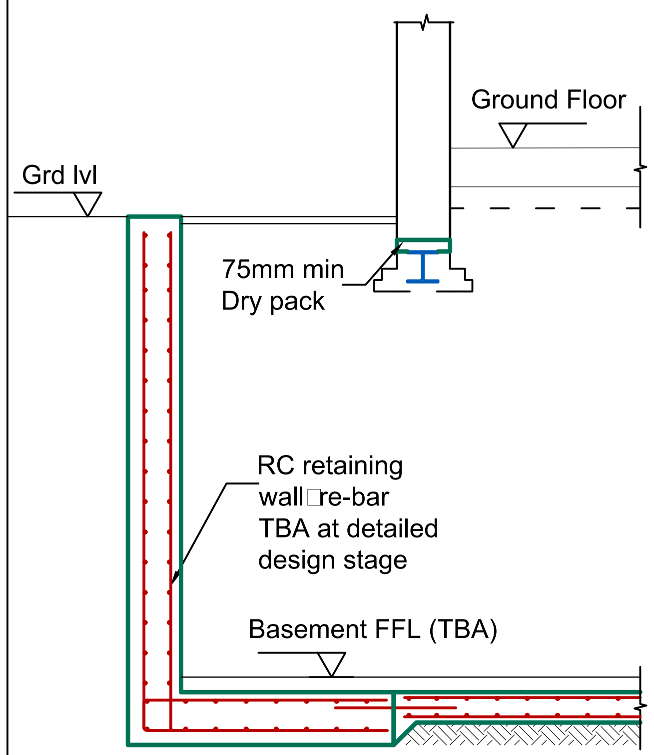
Ground Floor Plan

Scale 1:100



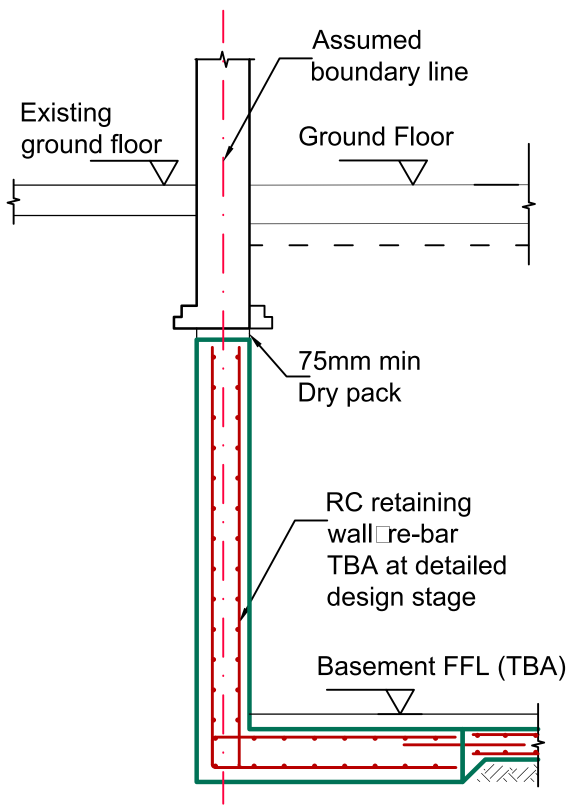
Basement Plan

Scale 1:100



Section 1-1

Scale 1:50



Section 2-2

Scale 1:50

Rev	Date	Amendments
-	02/02/2018	First issue for comment

Job Number	Date
171203	Feb '18
Dwg Number: SL-10	Rev: -
Drawn: CC	Ch'kd: CT
Scale: As shown @ A3	

Client: Frances A Bennett

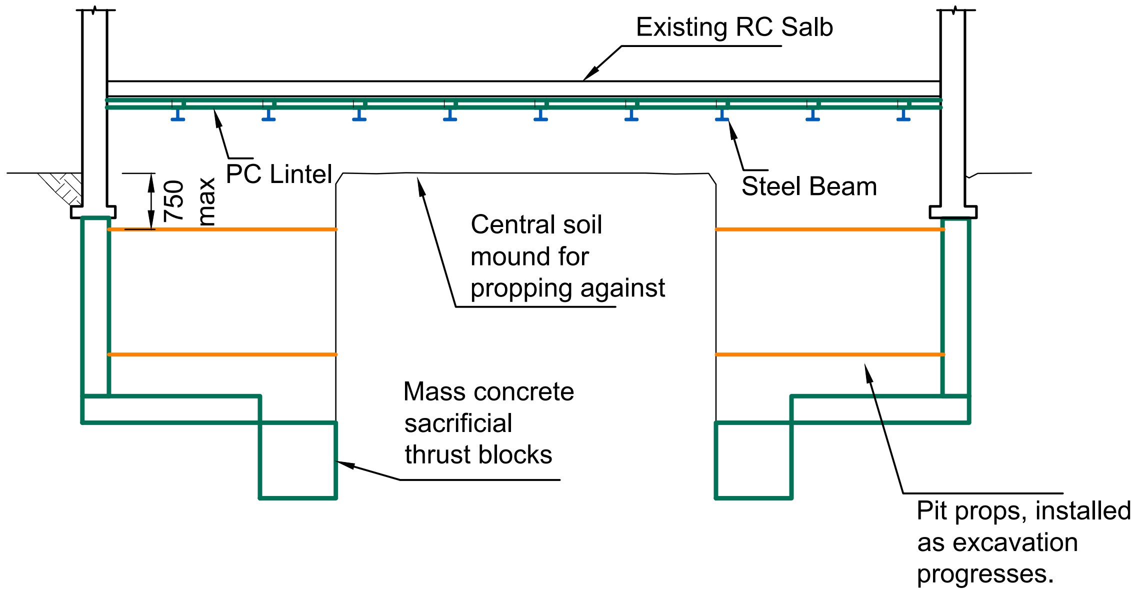
Project: 1 & 2 Falkland Mews

Title : Floor plans and Sections

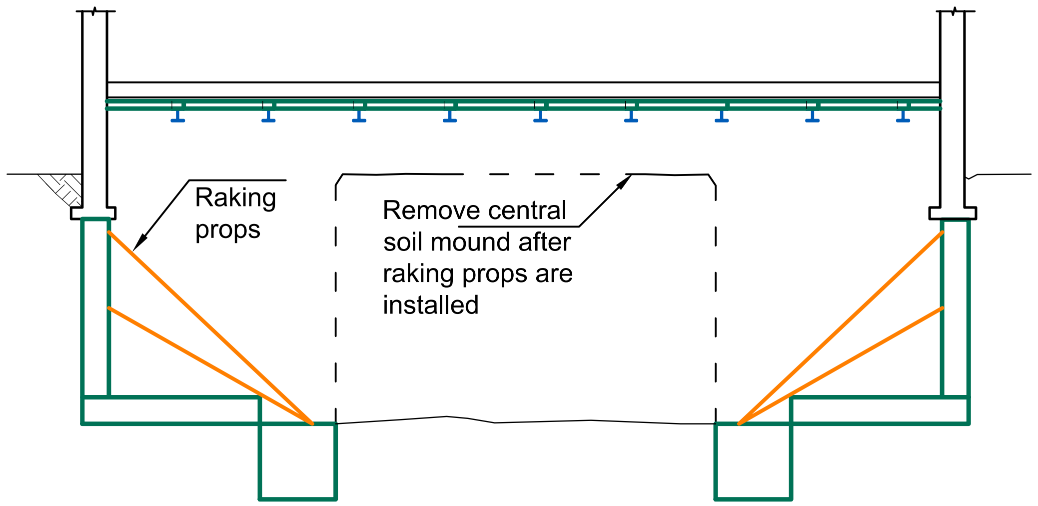
Croft Structural Engineers
Clockshop Mews, r/o 60 Saxon Rd, London, SE25 5EH.
020 8684 4744
www.croftse.co.uk

Appendix D: Construction Sequence and Temporary Works Proposals

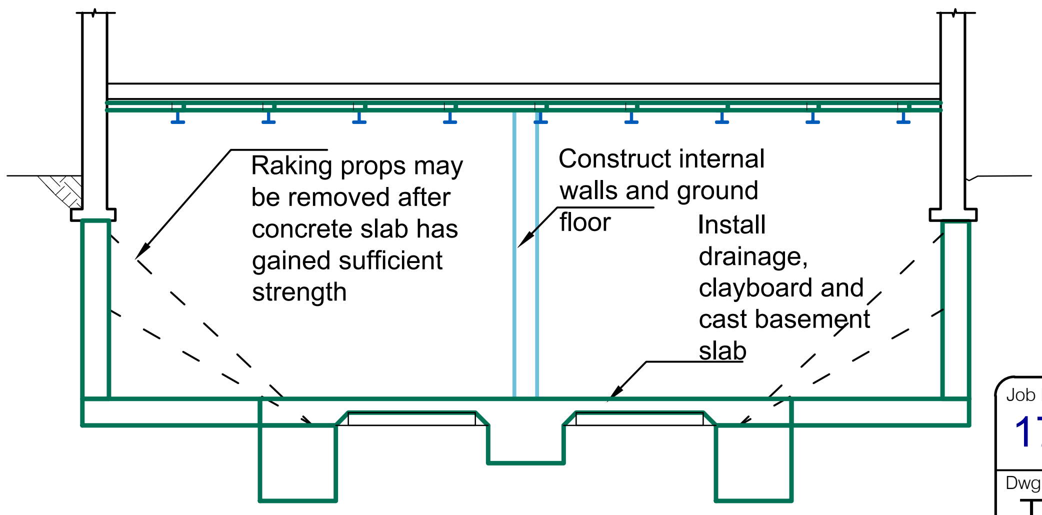
Phase 1



Phase 2



Phase 3



Typical section through building showing construction sequence

(1:100)

PHASE 1

1.1. Excavate lightwell to front of property down to 600mm below external ground level.
1.2. Excavate first front corner of lightwell.
1.3. Excavate second front corner of lightwell.
1.4. Continue excavating section pins to form front lightwell.
1.5. Excavate out first 1.2m around front opening, prop floor and erect conveyor.
1.6. Prop the existing ground bearing slab with PC Lintels
1.7. Excavate underpins in a hit and miss procedure following the sequence shown in plan on structural engineer's drawing (SL-10)
    1.7.1. Prop pits against central soil mound as excavation progresses
    1.7.2. Do not commence excavation for pin until at least 48 hours after drypacking for adjacent pin is complete (24hours minimum is possible if Conbextra 100 cement accelerator is added to dry pack mix)
    1.7.3. For every second pin, extend excavation to allow for subesquent construction of mass concrete thrust block below formation level

PHASE 2

2.1. Install raking props to wall and prop against thrust blocks.
2.2. Excavate central soil mass

PHASE 3

3.1. Excavate soil between pins
3.2. Install below slab drainage and clayboard
3.3. Construct internal foundations
3.4. Cast concrete floor slab.
    3.4.1. Cast around bases of raking props to allow for removal
3.5. After basement slab has gained sufficient strength, remove raking props.
3.6. Proceed with construction of internal walls and ground floor structure.

- PLANNING ISSUE -
NOT FOR CONSTRUCTION

Rev	Date	Amendments
-	02/02/2018	First issue for comment

Job Number	Date
171203	Feb '18
Dwg Number: TW-01	Rev: -
Drawn: CC	Ch'kd: CT
Scale: As shown @ A3	

Client: Frances A Bennett

Project: 1 & 2 Falkland Mews

Title : Construction Sequence and Temporary Works Proposals

Croft Structural Engineers
Clockshop Mews,
r/o 60 Saxon Rd,
London, SE25 5EH.
020 8684 4744
www.croftse.co.uk

Appendix E: Proposed Monitoring Statement

Job Number: 171203

CROFT STRUCTURAL ENGINEERS

Croft Structural Engineers
Clock Shop Mews
Rear of 60 Saxon Road
London SE25 5EH

T: 020 8684 4744
E: enquiries@croftse.co.uk
W: www.croftse.co.uk

Structural Monitoring Statement

Property Details
1&2 Falkland Mews
London
NW5 2PP

Client Information
Aarish Patel

Revision	Date	Comment
-	02.02.2018	First Issue

Contents

CROFT STRUCTURAL ENGINEERS

1. Introduction

Basement works are intended at 1&2 Falkland Mews. The structural works for this may require monitoring, depending on the requirements made during the planning application and the stipulations of the subsequent Party Wall Awards. . This statement describes the procedures for the Principal Contractor to follow to observe any movement that may occur to the existing properties, and also describes mitigation measures to apply if necessary.

2. Risk Assessment

The purpose of this risk assessment is to consider the impact of the proposed works and how they impact the party wall. There are varying levels of inspection that can be undertaken and not all works, soil conditions and properties require the same level of protection. In the table below, Monitoring Level 5 is considered the most appropriate.

Monitoring Level Proposed	Type of Works.
Monitoring 1 Visual inspection and production of condition survey by Party wall surveyors at the beginning of the works and also at the end of the works.	Loft conversions, cross wall removals, insertion of padstones Survey of LUL and Network Rail tunnels. Mass concrete, reinforced and piled foundations to new build properties

Monitoring 2 Visual inspection and production of condition survey by Party Wall Surveyors at the beginning of the works and also at the end of the works. Visual inspection of existing party wall during the works. Inspection of the footing to ensure that the footings are stable and adequate.	Removal of lateral stability and insertion of new stability fames Removal of main masonry load bearing walls. Underpinning works less than 1.2m deep
Monitoring 3 Visual inspection and production of condition survey by Party Wall Surveyors at the beginning of the works and also at the end of the works. Visual inspection of existing party wall during the works. Inspection of the footing to ensure that the footings are stable and adequate. Vertical monitoring movement by standard optical equipment	Lowering of existing basement and cellars more than 2.5m Underpinning works less than 3.0m deep in clays Basements up to 2.5m deep in clays
Monitoring 4 Visual inspection and production of condition survey by Party Wall Surveyors at the beginning of the works and also at the end of the works. Visual inspection of existing party wall during the works. Inspection of the footing to ensure that the footings are stable and adequate. Vertical monitoring movement by standard optical equipment Lateral movement between walls by laser measurements	New basements greater than 2.5m and shallower than 4m Deep in gravels Basements up to 4.5m deep in clays Underpinning works to Grade I listed building

Monitoring 5 Visual inspection and production of condition survey by Party wall surveyors at the beginning of the works and also at the end of the works. Visual inspection of existing party wall during the works. Inspection of the footing to ensure that the footings are stable and adequate. Vertical & lateral monitoring movement by theodolite at specific times during the projects.	Underpinning works to Grade I listed buildings Basements to Listed building Basements deeper than 4m in gravels Basements deeper than 4.5m in clays Underpinning, basements to buildings that are expressing defects.
Monitoring 6 Visual inspection and production of condition survey by Party wall surveyors at the beginning of the works and also at the end of the works. Visual inspection of existing party wall during the works. Inspection of the footing to ensure that the footings are stable and adequate. Vertical & lateral monitoring movement by electronic means with live data gathering. Weekly interpretation	Double storey basements supported by piled retaining walls in gravels and soft sands. (N<12)
Monitoring 7 Visual inspection and production of condition survey by Party wall surveyors at the beginning of the works and also at the end of the works. Visual inspection of existing party wall during the works. Inspection of the footing to ensure that the footings are stable and adequate. Vertical & lateral monitoring movement by electronic means with live data gathering with data transfer.	Larger multi-storey basements on particular projects.

3. Scheme Details

This document has been prepared by Croft Structural Engineers Ltd. It covers the development of a new basement.

Scope of Works

The works comprise:

- Visual Monitoring of the party wall
- Attachment of Tell tales or Demec Studs to accurately record movement of significant cracks.
- Attachment of levelling targets to monitor settlement.

- The monitoring of the above instrumentation is in accordance with Appendix A. The number and precise locations of instrumentation may change during the works; this shall be subject to agreement with the Principal Contractor (PC).
- All instruments are to be adequately protected against any damage from construction plant or private vehicles using clearly visible markings and suitable head protection e.g. manhole rings or similar. Any damaged instruments are to be immediately replaced or repaired at the contractors own cost.
- Reporting of all data in a manner easily understood by all interested parties.
- Co-ordination of these monitoring works with other site operations to ensure that all instruments can be read and can be reviewed against specified trigger values both during and post construction.
- Regular site meetings by the Principal Contractor (PC) and the Monitoring Surveyor (MS) to review the data and their implications.
- Review of data by Croft Structural Engineers

In addition, the PC will have responsibility for the following:

- Review of methods of working/operations to limit movements, and
- Implementation of any emergency remedial measures if deemed necessary by the results of the monitoring.

The Monitoring Surveyor shall allow for settlement and crack monitoring measures to be installed and monitored on various parts of the structure described in Table 1 as directed by the PC and Party Wall Surveyor (PWS) for the Client.

Item	Instrumentation Type
Party Wall Brickwork	
Settlement monitoring	Levelling equipment & targets
Crack monitoring	Visual inspection of cracking, Demec studs where necessary

Table 1: Instrumentation

General

The site excavations and substructure works up to finished ground slab stage have the potential to cause vibration and ground movements in the vicinity of the site due to the following:

a) Removal of any existing redundant foundations / obstructions;
b) Installation of reinforced concrete retaining walls under the existing footings;
c) Excavations within the site

The purpose of the monitoring is a check to confirm building movements are not excessive.

This specification is aimed at providing a strategy for monitoring of potential ground and building movements at the site.

This specification is intended to define a background level of monitoring. The PC may choose to carry out additional monitoring during critical operations. Monitoring that should be carried out is as follows:

a) Visual inspection of the party wall and any pre-existing cracking

b) Settlement of the party wall

All instruments are to be protected from interference and damage as part of these works.

Access to all instrumentation or monitoring points for reading shall be the responsibility of the Monitoring Surveyor (MS). The MS shall be in sole charge for ensuring that all instruments or monitoring points can be read at each visit and for reporting of the data in a form to be agreed with the PWS. He shall inform the PC if access is not available to certain instruments and the PC will, wherever possible, arrange for access. He shall immediately report to the PC any damage. The Monitoring Surveyor and the Principal Contractor will be responsible for ensuring that all the instruments that fall under their respective remits as specified are fully operational at all times and any defective or damaged instruments are immediately identified and replaced.

The PC shall be fully responsible for reviewing the monitoring data with the MS - before passing it on to Croft Structural Engineers - determining its accuracy and assessing whether immediate action is to be taken by him and/or other contractors on site to prevent damage to instrumentation or to ensure safety of the site and personnel. All work shall comply with the relevant legislation, regulations and manufacturer's instructions for installation and monitoring of instrumentation.

Applicable Standards and References

The following British Standards and civil engineering industry references are applicable to the monitoring of ground movements related to activities on construction works sites:

1. BS 5228: Part 1: 1997 - Noise and Vibration Control on Construction and Open Sites -Part 1.Code of practice for basic information and procedures for noise and vibration control, Second Edition, BSI 1999.
2. BS 5228: Part 2: 1997 - Noise and Vibration Control on Construction and Open Sites -Part 2.Guide to noise and vibration control legislation for construction and demolition including road construction and maintenance, Second Edition, BSI 1997.
3. BS 7385-1: 1990 (ISO 4866:1990) - Evaluation and measurement for vibration in buildings - Part 1: Guide for measurement of vibrations and evaluation of their effects on buildings, First Edition, BSI 1990.
4. BS 7385-2: 1993 - Evaluation and measurement for vibration in buildings - Part 2: Guide to damage levels from ground-borne vibration, First Edition, BSI 1999.
5. CIRIA SP 201 - Response of buildings to excavation-induced ground movements, CIRIA 2001.

CROFT STRUCTURAL ENGINEERS

SPECIFICATION FOR INSTRUMENTATION

General

The Monitoring Contractor is required to monitor, protect and reinstall instruments as described. The readings are to be recorded and reported. The following instruments are defined:

a) Automatic level and targets: A device which allows the measurement of settlement in the vertical axis. To be installed by the MS.

b) Tell-tales and 3 stud sets: A device which allows measurement of movement to be made in two axes perpendicular to each other. To be installed by the MS.

Monitoring of existing cracks

The locations of tell-tales or Demec studs to monitor existing cracks shall be agreed with Croft Structural Engineers.

Instrument Installation Records and Reports

Where instrumentation is to be installed or reinstalled, the Monitoring Surveyor, or the Principal Contractor, as applicable, shall make a complete record of the work. This should include the position and level of each instrument. The records shall include base readings and measurements taken during each monitoring visit. Both tables and graphical outputs of these measurements shall be presented in a format to be agreed with the CM. The report shall include photographs of each type of instrumentation installed and clear scaled sections and plans of each instrument installed. This report shall also include the supplier's technical fact sheet on the type of instrument used and instructions on monitoring.

Two signed copies of the report shall be supplied to the PWS within one week of completion of site measurements for approval.

Installation

All instruments shall be installed to the satisfaction of the PC. No loosening or disturbance of the instrument with use or time shall be acceptable. All instruments are to be clearly marked to avoid damage.

All setting out shall be undertaken by the Monitoring Surveyor or the Principal Contractor as may be applicable. The precise locations will be agreed by the PC prior to installation of the instrument.

The installations are to be managed and supervised by the Instrumentation Engineer or the Measurement Surveyor as may be applicable.

CROFT STRUCTURAL ENGINEERS

Monitoring

The frequencies of monitoring for each Section of the Works are given in Appendix A.

The following accuracies/ tolerances shall be achieved:

Party Wall settlement	±1.5mm
Crack monitoring	±0.75mm

REPORT OF RESULTS AND TRIGGER LEVELS

General

Within 24 hours of taking the readings, the Monitoring Surveyor will submit a single page summary of the recorded movements. All readings shall be immediately reviewed by Croft Structural Engineers prior to reporting to the PWS (Party Wall Surveyor).

Within one working day of taking the readings the Monitoring Contractor shall produce a full report (see below).

The following system of control shall be employed by the PC and appropriate contractors for each section of the works. The Trigger value, at which the appropriate action shall be taken, for each section, is given in Table 2, below.

The method of construction by use of sequential underpins limits the deflections in the party wall.

Between the trigger points, which are no greater than 2 m apart, there should be no more than 4mm.

Above Monitoring Level 3, lateral movement is required to be measured and the proposed trigger limit is 3mm

During works measurements are taken, these are compared with the limits set out below:

MOVEMENT		CATEGORY	ACTION
Vertical	Horizontal		
0mm-4mm	0-3mm	Green	No action required
4mm-7mm	3-6mm	AMBER	Detailed review of Monitoring: Check studs are OK and have not moved. Ensure site staff have not moved studs. If studs have moved reposition. Relevel to ensure results are correct and tolerance is not a concern. Inform Party Wall surveyors of amber readings. Double the monitoring for 2 further readings. If stable revert back. Carry out a local structural review and inspection. Preparation for the implementation of remedial measures should be required. Double number of lateral props
7mm-10mm	6-8mm		Implement remedial measures review method of working and ground conditions
>10mm	>8mm	RED	Implement structural support as required; Cease works with the exception of necessary works for the safety and stability of the structure and personnel; Review monitoring data and implement revised method of works

Table 2 – Movement limits between adjacent sets of Tell-tales or stud sets

Any movements which exceed the individual amber trigger levels for a monitoring measure given in Table 2 shall be immediately reported to the PWS, and a review of all of the current monitoring data for all monitoring measures must be implemented to determine the possible causes of the trigger level being exceeded. Monitoring of the affected location must be increased and the actions described above implemented. Assessment of exceeded trigger levels must not be carried out in isolation from an assessment of the entire monitoring regime as the monitoring measures are inter-related. Where required, measures may be implemented or prepared as determined by the specific situation and combination of observed monitoring measurement data.

Standard Reporting

1 No. electronic copy of the report in PDF format shall be submitted to the PWS.

The Monitoring Surveyor shall report whether the movements are within (or otherwise) the Trigger Levels indicated in Table 2. A summary of the extent of completion of any of the elements of works and any other significant events shall be given. These works shall be shown in the form of annotated plans (and sections) for each survey visit both local to the instrumentation and over a wider area. The associated changes to readings at each survey or monitoring point shall be then regulated to the construction activity so that the cause of any change, if it occurs, can be determined.

The Monitoring Surveyor shall also give details of any events on site which in his opinion could affect the validity of the results of any of the surveys.

The report shall contain as a minimum, for each survey visit the following information:

a) The date and time of each reading:
b) The weather on the day:
c) The name of the person recording the data on site and the person analysing the readings together with their company affiliations;
d) Any damage to the instrumentation or difficulties in reading;
e) Tables comparing the latest reading with the last reading and the base reading and the changes between these recorded data;
f) Graphs showing variations in crack width with time for the crack measuring gauges; and
g) Construction activity as described. It is very important that each set of readings is associated with the extent of excavation and construction at that time. Readings shall be accompanied by information describing the extent of works at the time of readings. This shall be agreed with the PC.

Spread-sheet columns of numbers should be clearly labelled together with units. Numbers should not be reported to a greater accuracy than is appropriate. Graph axis should be linear and clearly labelled together with units. The axis scales are to be agreed with the PC before the start of monitoring and are to remain constant for the duration of the job unless agreed otherwise. The specified trigger values are also to be plotted on all graphs.

The reports are to include progress photographs of the works both general to the area of each instrument and globally to the main Works. In particular, these are to supplement annotated plans/sections described above. Wherever possible the global photographs are to be taken from approximately the same spot on each occasion. The locations of these points on site are to be determined by the engineer at detailed design stage.

Erroneous Data

All data shall be checked for errors by the Monitoring Surveyor prior to submission. If a reading that appears to be erroneous (i.e. it shows a trend which is not supported by the surrounding instrumentation), he shall notify the PC immediately, resurvey the point in question and the neighbouring points and if the error is repeated, he shall attempt to identify the cause of the error. Both sets of readings shall be processed and submitted, together with the reasons for the errors and details of remedial works. If the error persists at subsequent survey visits, the Monitoring Surveyor shall agree with the PC how the data should be corrected. Correction could be achieved by correcting the readings subsequent to the error first being identified to a new base reading.

The Monitoring Surveyor shall rectify any faults found in or damage caused to the instrumentation system for the duration of the specified monitoring period, irrespective of cause, at his own cost.

Trigger Values

Trigger values for maximum movements as listed in Table 2. If the movement exceeds these values then action may be required to limit further movement. The PC should be immediately advised of the movements in order to implement the necessary works.

It is important that all neighbouring points (not necessarily a single survey point) should be used in assessing the impact of any movements which exceed the trigger values, and that rechecks are carried out to ensure the data is not erroneous. A detailed record of all activities in the area of the survey point will also be required as specified elsewhere.

Responsibility for Instrumentation

The Monitoring Surveyor shall be responsible for: managing the installation of the instruments or measuring points, reporting of the results in a format which is user friendly to all parties; and immediately reporting to all parties any damage. The Monitoring Surveyor shall be responsible for informing the PC of any movements which exceed the specified trigger values listed in Table 2 so that the PC can implement appropriate procedures. He shall immediately inform the PWS of any decisions taken.

APPENDIX A
MONITORING FREQUENCY

INSTRUMENT	FREQUENCY OF READING
Settlement monitoring and Monitoring existing cracks	**<u>Pre-construction</u>** Monitored once. **<u>During construction</u>** Monitored after every pin is cast for first 4 no. pins to gauge effect of underpinning. If all is well, monitor after every other pin. **<u>Post construction works</u>** Monitored once.

CROFT STRUCTURAL ENGINEERS

APPENDIX B

An Analysis on allowable settlements of structures (Skempton and MacDonald (1956))

The most comprehensive studies linking self-weight settlements of buildings to structural damage were carried out in the 1950's by Skempton and MacDonald (1956) and Polshin and Tokar. These studies show that damage is most often caused by differential settlements rather than absolute settlements. More recently, similar empirical studies by Boscardin and Cording (1989) and Boone (1996) have linked structural damage to ground movements induced by excavations and tunnelling activities.

In 1955 Skempton and MacDonald identified the parameter $\delta\rho/L$ as the fundamental element on which to judge maximum admissible settlements for structures. This criterion was later confirmed in the works of GRANT *et al.* [1975] and WALSH [1981]. Another important approach to the problem was that of BURLAND and WROTH [1974], based on the criterion of maximum tensile strains.

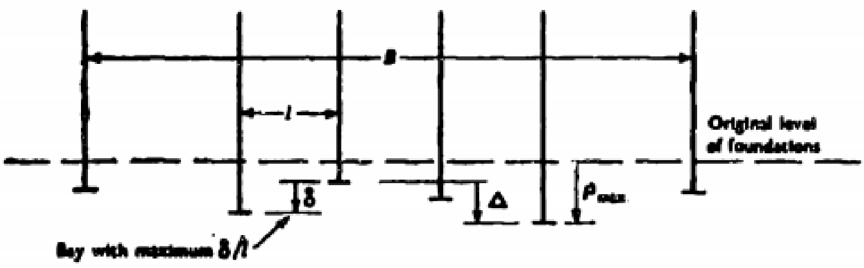


Figure 2.1 – Diagram illustrating the definitions of maximum angular distortion, δ/l, maximum settlement, ρ_{max}, and greatest differential settlement, Δ, for a building with no tilt (Skempton and MacDonald, 1956).

Figure 1: Diagram illustrating the definitions of maximum angular distortion, δ/l, maximum settlement, p_{max}, and greatest differential settlement ,Δ , for a building with no tilt (Skempton and MacDonald, 1956)

The differential settlement is defined as the greatest vertical distance between two points on the foundation of a structure that has settled, while the angular distortion, is the difference in elevation between two points, divided by the distance between those points.

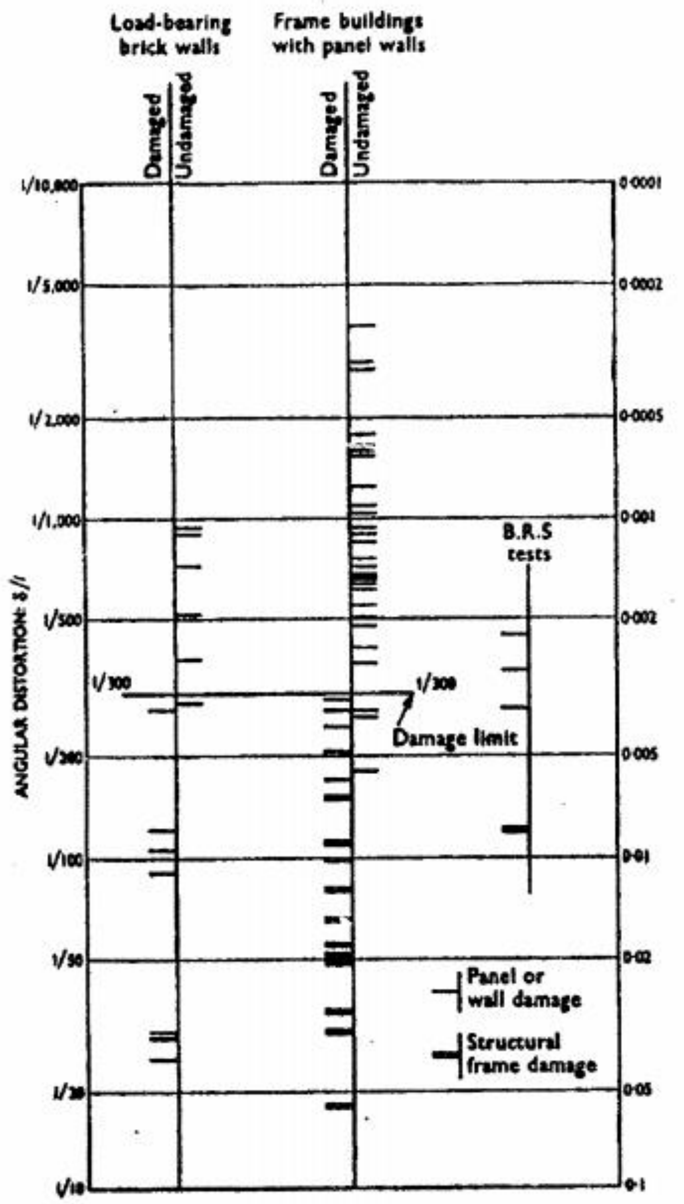
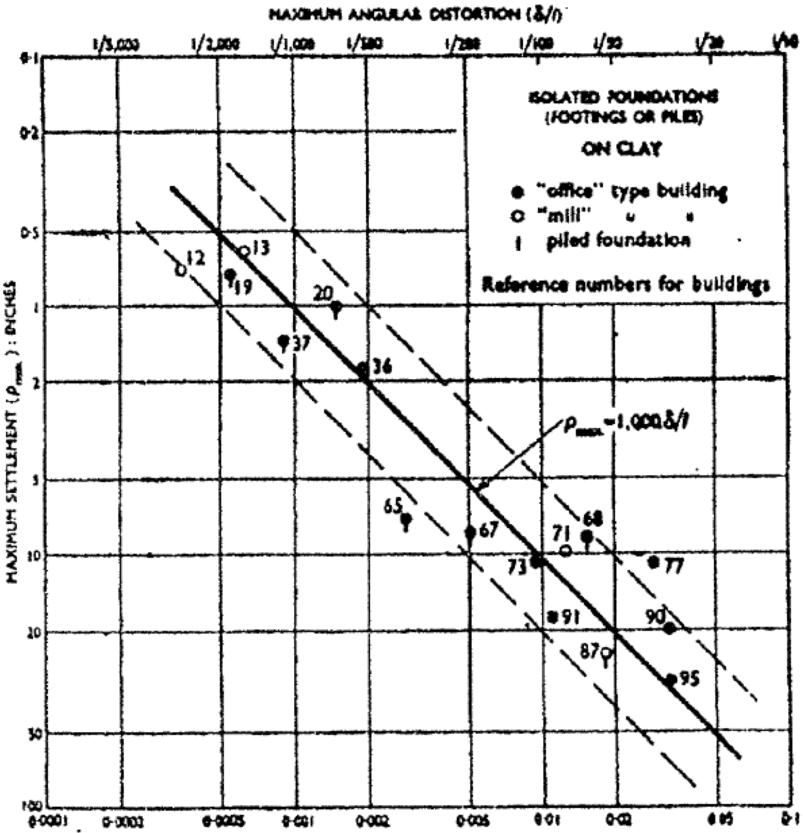
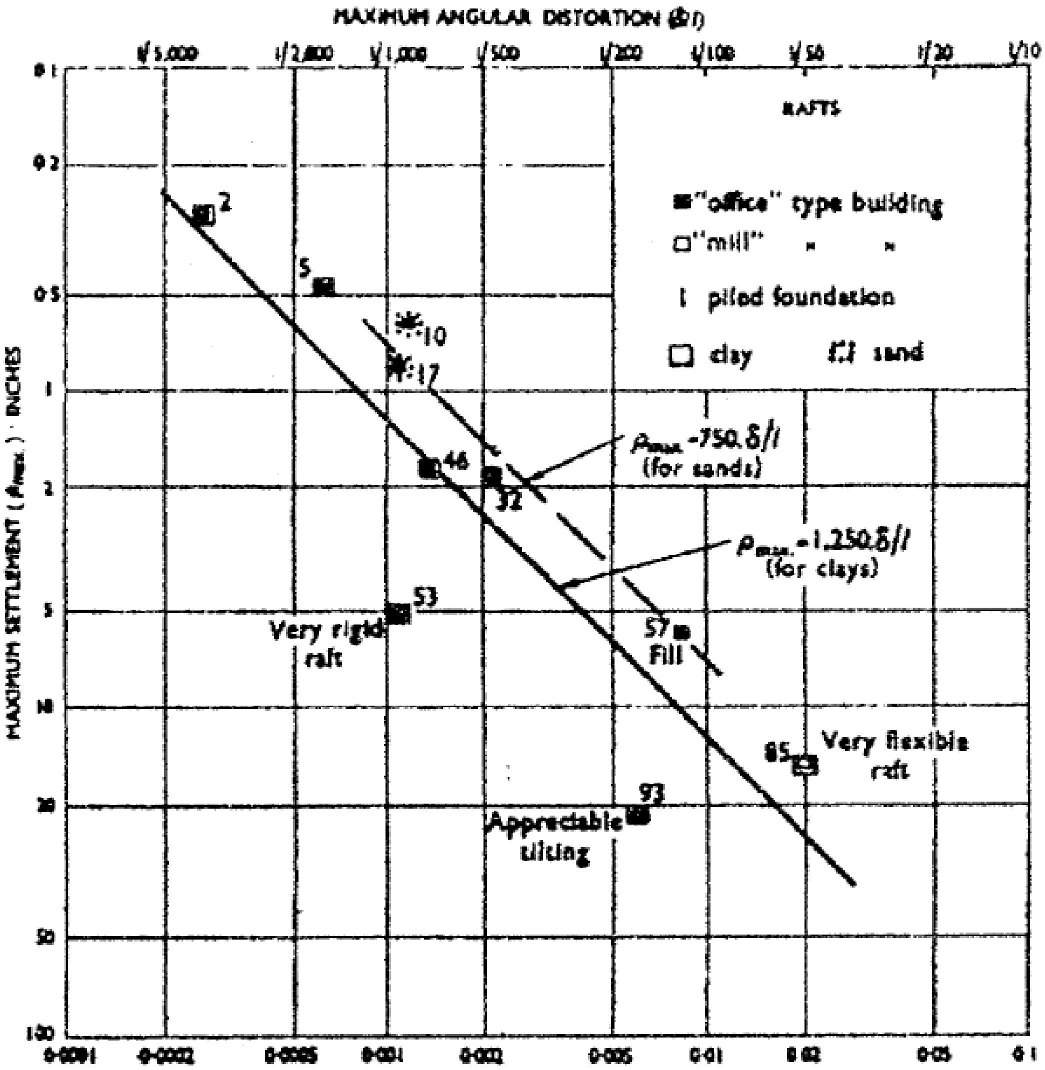


Figure 2: Skempton and MacDonald's analysis of field evidence of damage on traditional frame buildings and loadbearing brick walls

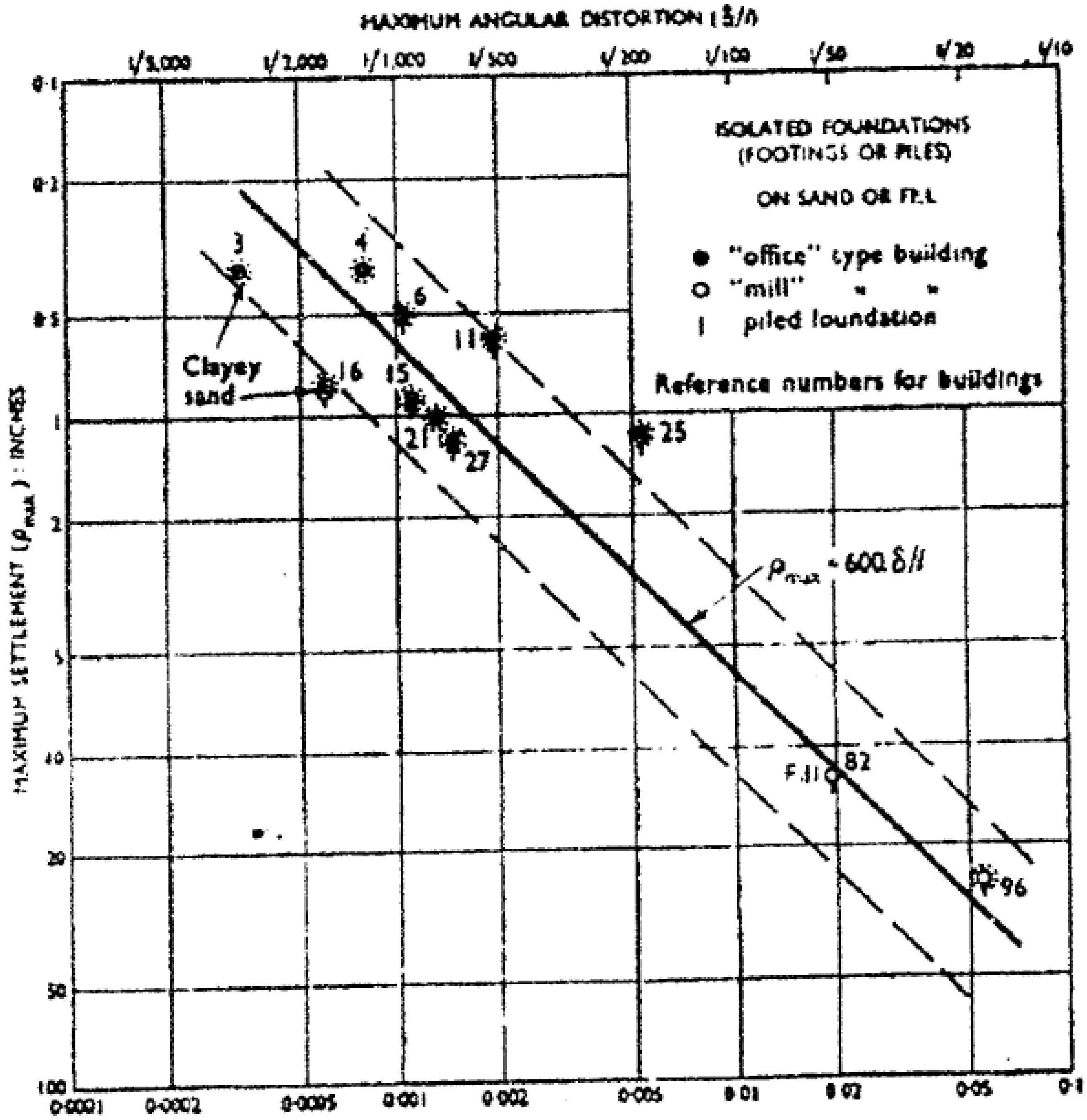
Data from Skempton and MacDonald's work suggest that the limiting value of angular distortion is 1/300. Angular distortion, greater than 1/300 produced visible cracking in the majority of buildings studied, regardless of whether it was a load bearing or a frame structure. As shown in the figure 2.

Other key findings by Skempton and MacDonald include limiting values of δ/l for structure, and a relationship between maximum settlement, ρ_{max} and δ/l for structures founded on sands and clays. The charts below show these relations for raft foundations and isolated footings.





CROFT STRUCTURAL ENGINEERS



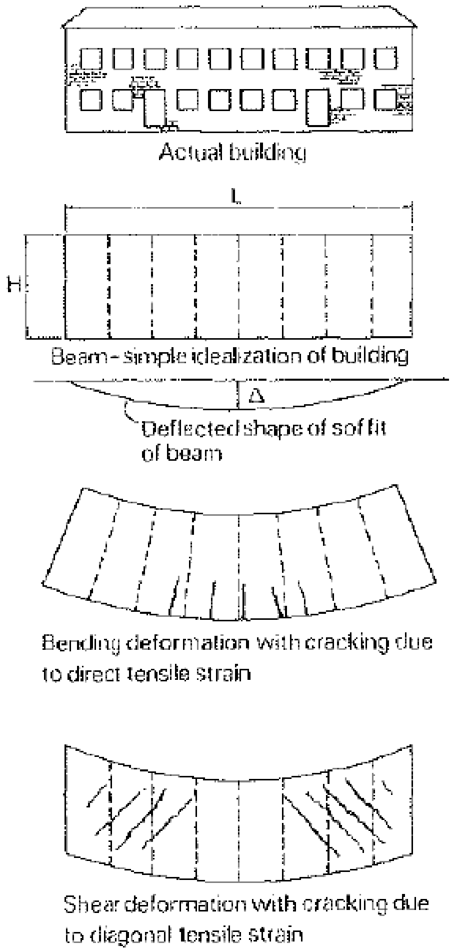


TABLE I

Angular distorsion	Characteristic situation
1/300	Cracking of the panels in frame buildings of the traditional type, or of the walls in load-bearing wall buildings;
1/150	Structural damage to the stanchions and beams;
1/500	Design limit to avoid cracking;
1/1000	Design limit to avoid any settlement damage.

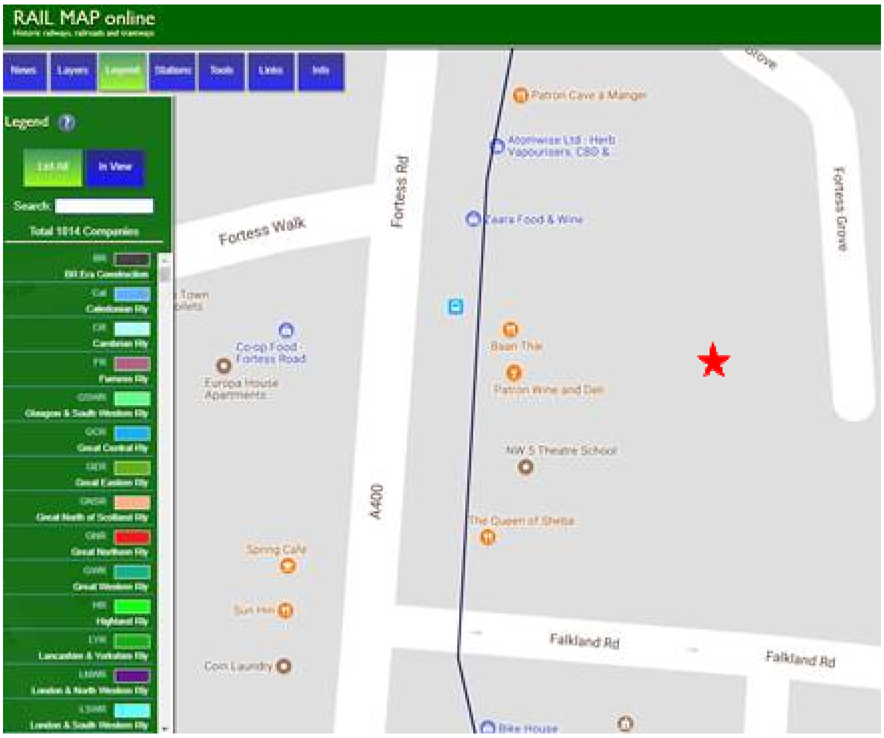
Appendix F: Communication with LUL

Concetta Cosenza

From: Concetta Cosenza <ccosenza@croftse.co.uk>
Sent: Friday, February 2, 2018 10:24 AM
To: 'john.cadman@tube.tfl.gov.uk'
Subject: Planning Stage Enquiry for 1&2 Falkland Mews, Camden NW5 2PP [Filed 02 Feb 2018 10:23]

Dear John,

We are involved in the planning application of a basement (not more than 3 m deep below ground level) for a property which is appears close to a tube line (see attached image). The property is in Camden and is possibly within 20m of the Northern line.



Please could you advise:

- At design stage, would we need a correlation survey for this?
- At design stage, would our client need to sign an RoCD?
- At design stage, will the client be expected to comply with G0023 and S050?
- At planning stage, will LUL require anything more from us besides notification (by way of this e-mail) and stating whether the above will be necessary at a later stage?

Please let us know if any of the above applies.

Kind regards

Concetta Cosenza
Structural Engineer
MSc, BEng

CROFT STRUCTURAL ENGINEERS
Clock Shop Mews, Rear of 60 Saxon Rd, SE25 5EH

t: 020 8684 4744
dir: 0208 684 4977
w: ccosenza@croftse.co.uk
Follow us at @CroftStructures

Please consider the environment before printing this email