CampbellReith
consulting engineers

Maryon House, 115-119 Goldhurst Terrace
London, NW6 3EY

Basement Impact Assessment
Audit

For

London Borough of Camden

Project Number: 12336-89
Revision: F1

December 2016

Campbell Reith Hill LLP
Friars Bridge Court
41-45 Blackfriars Road
London
SE1 8NZ

T:+44 (0)20 7340 1700
F:+44 (0)20 7340 1777
E:london@campbellreith.com
W:www.campbellreith.com

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Author	H Pham, MSc CEng MIStructE
Project Partner	E M Brown, BSc MSc CGeol FGS
Project Number	12366-89
Project Name	Maryon House, 115-119 Goldhurst Terrace, London NW6 3EY
Planning Reference	2016/3545/P

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Appendix

1.0 NON-TECHNICAL SUMMARY

1.1. CampbellReith was instructed by London Borough of Camden, (LBC) to carry out an audit on the Basement Impact Assessment submitted as part of the Planning Submission documentation for Maryon House, 115-119 Goldhurst Terrace, London NW6 3EY (planning reference 2016/3545/P). The basement is considered to fall within Category B as defined by the Terms of Reference.

1.2. The Audit reviewed the Basement Impact Assessment (BIA) for potential impact on land stability and local ground and surface water conditions arising from basement development in accordance with LBC's policies and technical procedures.

1.3. CampbellReith was able to access LBC's Planning Portal and gain access to the latest revision of submitted documentation and reviewed it against an agreed audit check list.

1.4. Subsequent to the initial audit, supplementary supporting documents have been provided by Elliott Wood and Applied Geotechnical Engineering. The documents are included in Appendix 3 of this report.

1.5. The BIA has been prepared by a firm of engineering consultants, Site Analysis Services Ltd. The Structural Engineering Report has been prepared by structural and civil engineering consultants, Elliott Wood. Following the initial audit, the authors of the submitted documents have been confirmed to possess suitable qualifications that comply with the requirements of CPG4.

1.6. It has been confirmed that the development site does not involve a listed building, or is in close proximity to a listed building.

1.7. The proposal includes the demolition of an existing four storey building and the construction of a new four storey building with a basement to provide 10 residential flats. The proposal also includes landscaping the areas to the front and rear of the site.

1.8. The BIA has stated that the proposed basement will be approximately 4.0m below ground level and will be within the London Clay, which is present between 1.5m below ground level and up to the full depth of investigation of 20.0m below ground level. The London Clay is overlaid by the Made Ground.

1.9. It is noted from the BIA that groundwater was not encountered within the boreholes and trial pits during the site investigation works. The subsequent monitoring indicates that ground water was not present within the monitoring standpipe installed in borehole. However, water was present in the window sample holes at about 1.05m below ground level. It is likely that the water encountered in the window sample holes is surface water run-off perched on top of the London Clay.

1.10. It is accepted that there are no hydrogeological or hydrological concerns with respect to the development proposals.

1.11. The BIA states that the basement walls below the party walls with no.'s 113 and 121 Goldhurst Terrace will be reinforced concrete underpins and will sit on mass concrete footings. The footings and the walls will be installed in a hit and miss sequence. The reinforced concrete walls will be fixed to the basement raft slab. Underpinning will also be used to construct the basement wall to the west and the wall is connected to the basement raft slab. To the east, the basement wall will be formed by a contiguous piled wall with reinforced concrete lining wall, designed to resist hydrostatic water pressures. Calculations for the reinforced concrete walls have been provided. Following the initial audit, calculations for ground bearing pressure and basement raft slab under superstructure loads and uplift forces from hydrostatic pressure and heave have been submitted.

1.12. It is noted that a full ground movement analysis has been carried out to assess the effect on the surrounding properties. The predicted damage category of the adjoining properties is generally Very Slight (Burland Category 1) or less, with two walls being predicted to suffer possible Category 1/Category 2 damage. Appropriate mitigation measures, and a temporary and permanent works methodology have been provided. Following the initial audit, a revised ground movement assessment has been submitted. The assessment is based on conservative engineering assumptions.

1.13. It is noted that there are two trees to the front of the site and they have been considered in the design and method of construction of the proposed basement to minimise disruption to the tree roots.

1.14. It is accepted that the new development and associated basement is at low risk of flooding and with the implementation of SUDS at the site, there will be no increase in flood risk elsewhere as a result of the development.

2.0 INTRODUCTION

2.1. CampbellReith was instructed by London Borough of Camden (LBC) on 25 July 2015 to carry out a Category B Audit on the Basement Impact Assessment (BIA) submitted as part of the Planning Submission documentation for Maryon House, 115-119 Goldhurst Terrace, London NW6 3EY, Camden Reference 2016/3435/P.

2.2. The Audit was carried out in accordance with the Terms of Reference set by LBC. It reviewed the Basement Impact Assessment for potential impact on land stability and local ground and surface water conditions arising from basement development.

2.3. A BIA is required for all planning applications with basements in Camden in general accordance with policies and technical procedures contained within:

- Guidance for Subterranean Development (GSD). Issue 01. November 2010. Ove Arup & Partners.
- Camden Planning Guidance (CPG) 4: Basements and Lightwells.
- Camden Development Policy (DP) 27: Basements and Lightwells.
- Camden Development Policy (DP) 23: Water.

2.4. The BIA should demonstrate that schemes:

a) maintain the structural stability of the building and neighbouring properties;

b) avoid adversely affecting drainage and run off or causing other damage to the water environment; and,

c) avoid cumulative impacts upon structural stability or the water environment in the local area

and evaluate the impacts of the proposed basement considering the issues of hydrology, hydrogeology and land stability via the process described by the GSD and to make recommendations for the detailed design.

2.5. LBC's Audit Instruction described the planning proposal as "Construction of four storey residential building with basement to provide 10 residential units (2 x 1 bed, 5 x 2 beds and 3 x 3 beds), associated landscaping and refuse store to the front of the site following demolition of existing four storey residential building." The Audit Instruction also confirmed the property did not involve a listed building nor was a neighbour to a listed building.

2.6. CampbellReith accessed LBC's Planning Portal on 03 August 2016 and gained access to the following relevant documents for audit purposes:

- Planning Statement dated June 2016 by Savills.
- Basement Impact Assessment dated May 2016 by Site Analytical Services Ltd.

  This report includes the following documents in the appendices.

  - Appendix A: Ground Investigation Report
  - Appendix B: Ground Movement Assessment
- Report on Phase 1 Risk Assessment dated May 2016 by Site Analytical Services Ltd.
- Design & Access Statement dated June 2016 by KSR Architects.
- Demolition Drawings, Existing Plan/Elevation Drawings, and Proposed Plan/Section/Elevation Drawings dated June 2016 by KSR Architects.
- Structural Engineering Report and Subterranean Construction Method Statement dated June 2016 by Elliott Wood.
- Construction Management Plan dated May 2016 by Motion Ltd.
- Surface Water and Flood Risk Assessment dated 09 June 2016 by Water Environment Ltd.
- SUDS Drainage Statement dated 07 June 2016 by Elliott Wood.
- Landscape Design Proposal dated 22 June 2016 by John Davies Landscape.

2.7. Subsequent to the issue of the initial audit report, further information was provided by Elliott Wood and Applied Geotechnical Engineering as detailed below:

- Supplementary structural calculation.
- Revised ground movement assessment.
- Confirmation of the qualifications of the Structural Engineering Report's authors.

The additional information is included in Appendix 3.

3.0 BASEMENT IMPACT ASSESSMENT AUDIT CHECK LIST

Item	Yes/No/NA	Comment
Are BIA Author(s) credentials satisfactory?	Yes	See BIA Section 1.
Is data required by Cl.233 of the GSD presented?	Yes	
Does the description of the proposed development include all aspects of temporary and permanent works which might impact upon geology, hydrogeology and hydrology?	Yes	See BIA and Structural Engineering Report.
Are suitable plan/maps included?	Yes	
Do the plans/maps show the whole of the relevant area of study and do they show it in sufficient detail?	Yes	
Land Stability Screening: Have appropriate data sources been consulted? Is justification provided for 'No' answers?	Yes	See BIA Table 2.
Hydrogeology Screening: Have appropriate data sources been consulted? Is justification provided for 'No' answers?	Yes	See BIA Table 2.
Hydrology Screening: Have appropriate data sources been consulted? Is justification provided for 'No' answers?	Yes	See BIA Table 2.
Is a conceptual model presented?	Yes	See Phase 1 Risk Assessment Report Section 9.
Land Stability Scoping Provided? Is scoping consistent with screening outcome?	Yes	See BIA Section 4.

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Item	Yes/No/NA	Comment
Hydrogeology Scoping Provided? Is scoping consistent with screening outcome?	Yes	See BIA Section 4. It is noted the BIA by Site Analytical Services Ltd does not include a scope for the item 3 identified in the screening section. However, a scope is included the Surface Water and Flooding Impact Assessment Section 2.
Hydrology Scoping Provided? Is scoping consistent with screening outcome?	Yes	See BIA Section 4 and Surface Water and Flooding Impact Assessment Section 2.
Is factual ground investigation data provided?	Yes	See BIA Appendix A.
Is monitoring data presented?	Yes	See BIA Section 5.3 and Ground Investigation Report Appendix B.
Is the ground investigation informed by a desk study?	Yes	It is noted that the Ground Investigation was undertaken at about the same time as the Desk Study.
Has a site walkover been undertaken?	Yes	See Phase 1 Risk Assessment.
Is the presence/absence of adjacent or nearby basements confirmed?	Yes	See Planning Statement Section 2 and Ground Movement Assessment Section 1.
Is a geotechnical interpretation presented?	Yes	See BIA Sections 5 and 6, and Ground Investigation Report.
Does the geotechnical interpretation include information on retaining wall design?	Yes	See BIA Section 6.
Are reports on other investigations required by screening and scoping presented?	Yes	Ground Investigation Report.
Are baseline conditions described, based on the GSD?	Yes	

Item	Yes/No/NA	Comment
Do the base line conditions consider adjacent or nearby basements?	Yes	
Is an Impact Assessment provided?	Yes	See BIA Section 7.
Are estimates of ground movement and structural impact presented?	Yes	See Ground Movement Assessment Report.
Is the Impact Assessment appropriate to the matters identified by screen and scoping?	Yes	
Has the need for mitigation been considered and are appropriate mitigation methods incorporated in the scheme?	Yes	See Structural Engineering Report Section 8.
Has the need for monitoring during construction been considered?	Yes	See Structural Engineering Report Section 8.
Have the residual (after mitigation) impacts been clearly identified?	Yes	
Has the scheme demonstrated that the structural stability of the building and neighbouring properties and infrastructure will be maintained?	Yes	See Structural Engineering Report and Ground Movement Assessment Report.
Has the scheme avoided adversely affecting drainage and run-off or causing other damage to the water environment?	Yes	
Has the scheme avoided cumulative impacts upon structural stability or the water environment in the local area?	Yes	See BIA and Structural Engineering.
Does report state that damage to surrounding buildings will be no worse than Burland Category 2?	Yes	See Ground Movement Assessment Report.
Are non-technical summaries provided?	Yes	See BIA Sections 3.9, 4.2, 5.6, 6.0, 7.0, and Structural Engineering Report 'Non-Technical Summary' Section.

4.0 DISCUSSION

4.1. The Basement Impact Assessment (BIA) has been prepared by Site Analysis Services Ltd. The Structural Engineering Report has been prepared by structural and civil engineering consultants, Elliott Wood. Following the initial audit, the authors of the submitted documents have been confirmed to possess suitable qualifications that comply with the requirements of CPG4.

4.2. The proposal includes the demolition of an existing four storey building and the construction of a new four storey building with a basement to provide 10 residential flats. The proposal also includes landscaping the areas to the front and rear of the site. The adjacent buildings to the north-west and south-east of the proposed site are three storeys. It is understood that none of the adjacent buildings is known to have a basement.

4.3. A ground investigation has been undertaken to identify that the geology at the site consists of Made Ground up to 1.5m below ground level, underlain by London Clay up to the depth of investigation of 20m. The proposed basement will be founded within the London Clay Formation, which typically comprises stiff and very stiff silty sandy clay with an allowable bearing pressure of 165kN/m2 at 3.0m depth. Following the initial audit, additional calculations have been submitted, which indicate the adequacy of the bearing stratum.

4.4. It is noted from the BIA that groundwater was not encountered within the boreholes and trial pits during the site investigation works. The subsequent monitoring, approximately 6 weeks after, indicates that groundwater was not present within the monitoring standpipe installed in the borehole. However, water was present in the window sample holes at about 1.05m below ground level. It is likely that the water encountered in the window sample holes is surface water run-off perched on top of the London Clay. Perched groundwater could be encountered during basement excavation and the contractor should have a plan in place to deal with any perched groundwater inflows.

4.5. It is accepted that there are no hydrogeological or hydrological concerns with respect to the development proposals.

4.6. The BIA states that the basement walls below the party walls with no.'s 113 and 121 Goldhurst Terrace will be reinforced concrete underpins and will sit on mass concrete footings. The footings and the walls will be installed in a hit and miss sequence. The reinforced concrete walls will be fixed to the basement raft slab. This type of construction should be agreed as part of the Party Wall award. Underpinning will also be used to construct the basement wall to the west and the wall is connected to the basement raft slab. To the east, the basement wall will be formed by a contiguous piled wall with reinforced concrete lining wall, designed to resist hydrostatic water pressures. Calculations for the reinforced concrete walls have been provided.

Following the initial audit, calculations for ground bearing pressure and basement raft slab have been submitted.

4.7. It is noted that a full ground movement analysis has been carried out to assess the effect on the surrounding properties. Whilst the selection of the soil stiffness in the original GMA was not considered appropriate to the excavation of a shallow basement in weathered London Clay, these parameters have now been revised and the predicted ground movements presented are accepted.

4.8. It is also noted that the predicted damage category of the adjoining properties is generally Very Slight (Burland Category 1) or less, although, for the rear walls of 111, 113 and 121-125 it is predicted as being on the boundary of Slight to Very Slight. Appropriate mitigation measures, and a temporary and permanent works methodology have been provided. The assessment recommends that consideration is given to the pre-loading of temporary props.

4.9. It is noted that there are two trees to the front of the site, which should be protected. They have been considered in the design and method of construction of the proposed basement to minimise disruption to the tree roots. It has been proposed that the underpins to the western perimeter will be excavated using hand tools to prevent excessive damage to the tree roots.

4.10. The Environment Agency flood zone maps indicate that the site is located in Flood Zone 1. It is accepted that the new development and associated basement is at low risk of flooding and with the implementation of SUDS at the site, there will be no increase in flood risk elsewhere as a result of the development.

5.0 CONCLUSIONS

5.1. The Basement Impact Assessment (BIA) has been prepared by Site Analysis Services Ltd. The Structural Engineering Report has been prepared by a well-known firm of structural and civil engineering consultants, Elliott Wood. The authors of the BIA report and the Structural Engineering Report have been confirmed to possess suitable engineering qualifications that meet LBC requirements.

5.2. The proposal includes the demolition of an existing four storey building and the construction of a new four storey building with a basement to provide 10 residential flats. The proposal also includes landscaping the areas to the front and rear of the site.

5.3. Ground investigation have been undertaken to identify that the geology at the site consists of Made Ground up to 1.5m below ground level, underlain by London Clay up to the depth of investigation of 20m. The proposed basement will be founded within the London Clay Formation.

5.4. Although, groundwater was not encountered within the boreholes and trial pits during the site investigation works, perched water was recorded on top of the London Clay.

5.5. It is accepted that there are no hydrogeological or hydrological concerns with respect to the development proposals.

5.6. The basement walls below the party walls with no.'s 113 and 121 Goldhurst Terrace will be reinforced concrete underpins on mass concrete footing installed in a hit and miss sequence. The reinforced concrete walls will be fixed to the basement raft slab. This type of construction should be agreed as part of the Party Wall award. Underpinning will also be used to construct the basement wall to the west, with a contiguous piled wall with reinforced concrete lining wall to the east. The walls are designed to resist hydrostatic water pressures. Following the initial audit report, additional calculations to check the adequacy of the bearing stratum and basement raft slab have been provided.

5.7. A ground movement analysis has predicted a damage category of typically Very Slight (Burland Category 1) or less to adjoining properties, with Slight damage being predicted to two walls. Appropriate mitigation measures and a temporary and permanent works methodology have been provided. Following the initial audit, a revised ground movement assessment has been submitted. The assessment is based on conservative engineering assumptions.

5.8. It is accepted there are no slope stability concerns with respect to the development proposals.

5.9. It is noted that there are two trees to the front of the site. They will be protected and have been considered in the design and method of construction of the proposed basement.

5.10. It is accepted that the new development and associated basement is at low risk of flooding and with the implementation of SUDS at the site, there will be no increase in flood risk elsewhere as a result of the development.

Appendix 1: Residents' Consultation Comments

Residents' Consultation Comments

Surname	Address	Date	Issue raised	Response
Caiden	Flat 2, 121 Goldhurst Terrace NW6 3EX	28/07/2016	Effects of basement excavation on the stability of the adjoining properties.	See 4.6-4.8
O'Hegarty	48 Canfield Gardens, NW6 3EB	26/07/2016	Effects on surface water and drainage.	See 4.5 & 4.9
Spencer	Charmondel Services Ltd, 23 King Street, SW1Y 6QY Representation of the owner of Flat 1, 121 Goldhurst Terrace NW6 3EX	01/08/2016	Effects of basement excavation on the stability of the adjoining properties.	See 4.6-4.8

Appendix 2: Audit Query Tracker

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Audit Query Tracker

Query No	Subject	Query	Status/Response	Date closed out
1	BIA Content	Confirmation that qualifications of authors/ reviewers of the Structural Engineering Report comply with requirements of CPG 4	See 4.1	06/12/2016
2	Stability	Checking the adequacy of the bearing stratum and calculations of basement raft slab.	See 4.6	06/12/2016
3	Stability	Soil stiffness parameters not considered appropriate. Long term heave to be confirmed.	See 4.7 and 4.8	06/12/2016

Appendix 3: Supplementary Supporting Documents

Goldhurst Terrace Basement Raft Calculations
Revised GMA / Damage Category Assessment
Email from David Whittington, Savills, 02/12/2016

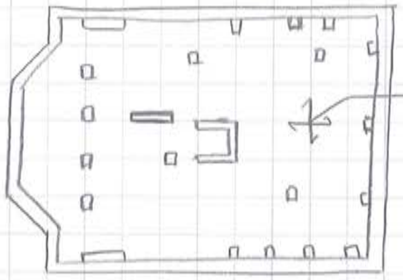
Project name:
Goldhurst Terrace

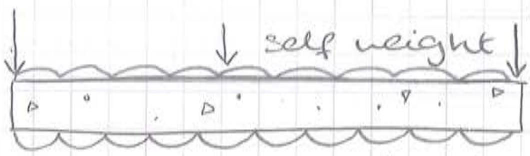
Project number: 2150657 | Sheet: 01 | Revision: P1

Date: 12/09/16 | Engineer: JLM | Checked: GWa

elliottwood

Design of basement raft





- heave:
  conservatively assume that total heave = weight of soil removed, and that 50% of total heave is relieved during construction

↳ design heave pressure = 50% weight of soil removed

$4m \times 20^{*}kN/m^3 \times 50\% = 40\ kN/m^2$ - SLS

$40 \times 1.5 = 60\ kN/m^2$ at ULS

- self weight of raft:
  ignore LL + SDL - as this is most conservative

↳ 450 mm RC = 11.25 kN/m²
50 mm screed = 1.0 kN/m²

12.25 kN/m²

multiply by a factor of 1.0, as this is a beneficial load

* γ_{sat} has been used to account for hydrostatic pressure, as the raft is founded in London Clay + it is assumed that there will be no perched water at 4m depth

- Total column/wall loads

↳

Floor	Area	DL →	ΣDL(kN)	LL →	ΣLL(kN)
Roof	150	1.5	225	0.6	90
3F	230	8.4	1930	2.5	575
2F	280	8.4	2350	2.5	700
1F	310	8.4	2640	2.5	785
GF	400	8.4	3360	2.5	1000
Facade: 14kN/m × 85m × 4 stories = 4760					
Total			15,270kN		3,150kN

↳ SLS = 15,270 + 3,150 = 18,420kN
ULS = 1.35 (15,270) + 1.5 (3,150) = 23,340kN

- area of basement = 400 m² → average load = 23,340/400 = 63kN/m²

⇒ modeling the raft:

- apply supports at each of the wall/column locations
- wall/column loads + heave loads are unfaverable, raft selfweight is favourable
  ↳ total area load applied

$$\underbrace{63\,kN/m^2}_{\text{walls/columns}} + \underbrace{60\,kN/m^2}_{\text{heave}} - \underbrace{12\,kN/m^2}_{\text{raft SW}} = 111\,kN/m^2$$

* Note - hydrostatic load has not been included, as the raft is founded in London clay. They have been included in the calculations for the retaining walls as there is a posibility of perched water at high level, but at a depth of 4m, this is deemed unlikely.

Project name: Goldhurst Terrace

Project number: 2150657 | Sheet: 03 | Revision: P1

Date: 12/09/16 | Engineer: JLM | Checked: GWa

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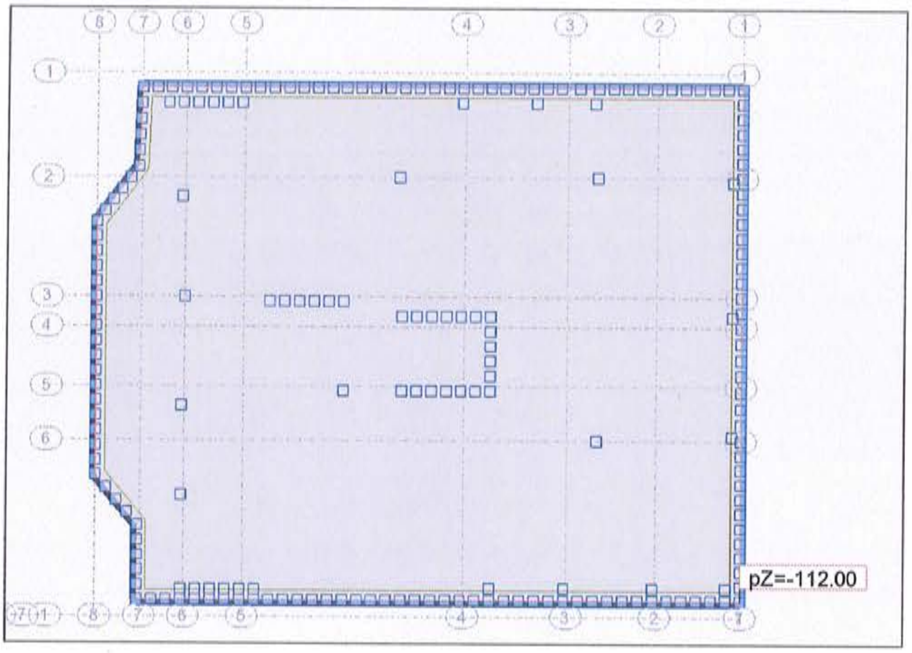
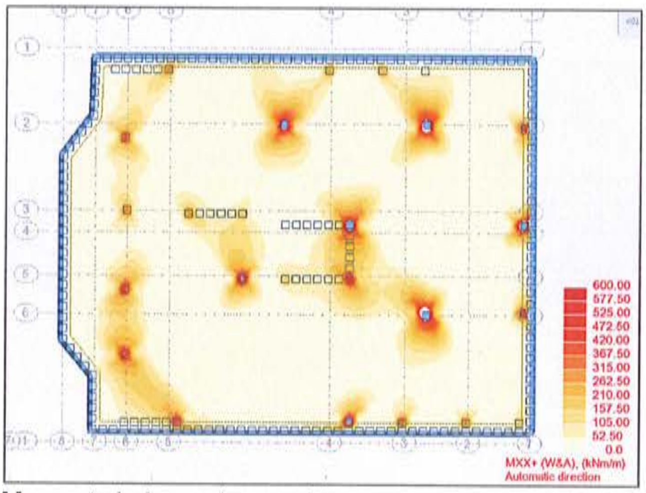
Robot model:

- Raft slab modelled as 450mm thick RC panel.
- 112kN/m^2 load applied to it. The self-weight has been removed, as this has already been considered in the 112kN/m^2 calculations.
- Supports have been applied at all wall and column positions.
- Moments in each direction have been considered

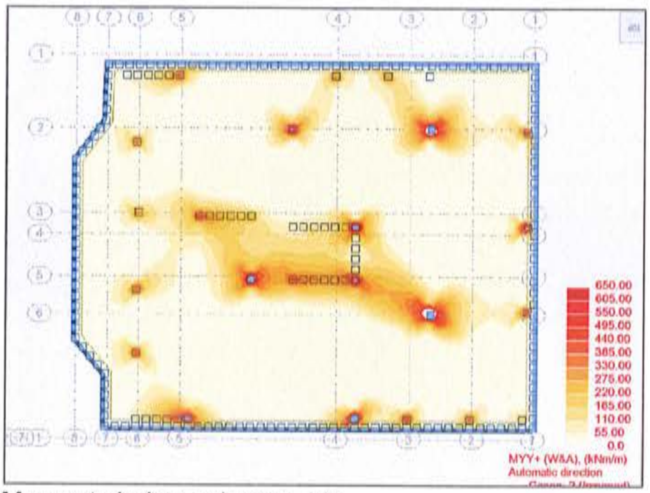
Loading applied:



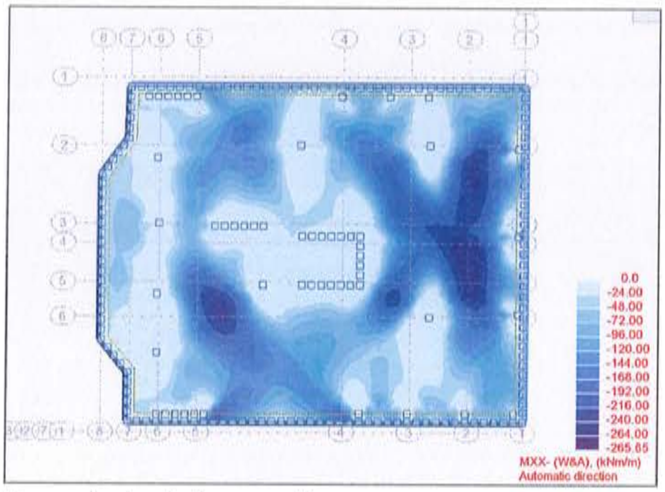
Results:



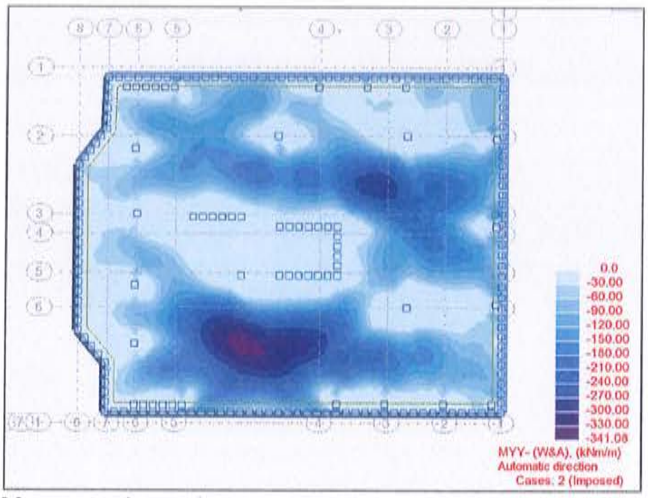
Moments below columns- Mx



Moments below columns- My



Moments due to heave - Mx



Moments due to heave - My

Design of reinforcement:

- Reinforcement in the top of the slab will be governed by the heave pressures. From moments maps, design for a moment of 337kNm
- Reinforcement in the bottom of the slab will be governed by the column/ wall point loads. Peak values have been ignored, as in practise, these would occur within the column depth. A maximum moment of 650kNm has been assumed at column/wall locations, as by inspection this is the maximum average moment at the column edges. 300kNm has been assumed in the general case.

Top reinforcement:

C_{nom}	75	b	1000
Bar ϕ	20	h	450
f_y	500	d	365
M_{ed}	341	f_{ck}	32

$k = \dfrac{M_{ed}}{bd^2f_{ck}} = \dfrac{341000000}{4263200000}$ 0.080

$z/d = 0.5(1+\sqrt{(1-(3.53k))}$ 0.92

$z =$ 337

$A_{s,req} = \dfrac{M_{ed}}{0.87zf_y} = \dfrac{341000000}{146639.8356}$ 2325

$A_{s,min} = $ 0.0013xbxd 474.5
or $0.26 \times (f_{ctm}/f)bd$ 626

$\boxed{A_{s,req} > A_{s,min} \quad \text{provide} \quad 2325 \quad mm^2}$

⇨ Provide H25 at 200mm (2454mm^2)

Bottom reinforcement:

C_{nom}	50	b	1000
Bar ϕ	20	h	450
f_y	500	d	390
M_{ed}	650	f_{ck}	32

$k = \dfrac{M_{ed}}{bd^2f_{ck}} = \dfrac{650000000}{4867200000}$ 0.134

$z/d = 0.5(1+\sqrt{(1-(3.53k))}$ 0.86

$z =$ 337

$A_{s,req} = \dfrac{M_{ed}}{0.87zf_y} = \dfrac{650000000}{146495.6954}$ 4437

$A_{s,min} = $ 0.0013xbxd 507
or $0.26 \times (f_{ctm}/f)bd$ 669

$\boxed{A_{s,req} > A_{s,min} \quad \text{provide} \quad 4437 \quad mm^2}$

⇨ Provide H25 at 100 (4909mm^2), at column locations, with H25 at 200mm elsewhere

Project name:
Coleshurst Terrace

Project number: 2150657 | Sheet: 05 | Revision: P1

Date: 12/09/16 | Engineer: JLM | Checked: GWa

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Bearing pressure check under raft:

Total SW check down to GF = 18,420 KN (from previous calcs

weights of basement box:

Retaining walls = 350mm x 4m x 25KN/m³ x 80m
= 2,800 KN

Raft = 450mm RC = 11.25
50 mm screed = 1.0
finishes = 0.5
} DL = 12.75 KN/m²

domestic = 1.5
partitions = 1.0
} LL = 2.5 KN/m²

area of basement = 400m² → 400 x (12.75 + 2.5) = 6,100 KN

=> total = 18,420 + 2,800 + 6,100 = 27,320 KN

27,320/400 = 68 KN/m² - average

↳ from SI report, bearing pressure = 165 KN/m²
∴ OK

- see Tedds calc to check bearing pressure under the more highly loaded areas

max pressure = 97.8 KN/m² < 165 KN/m² ∴ OK

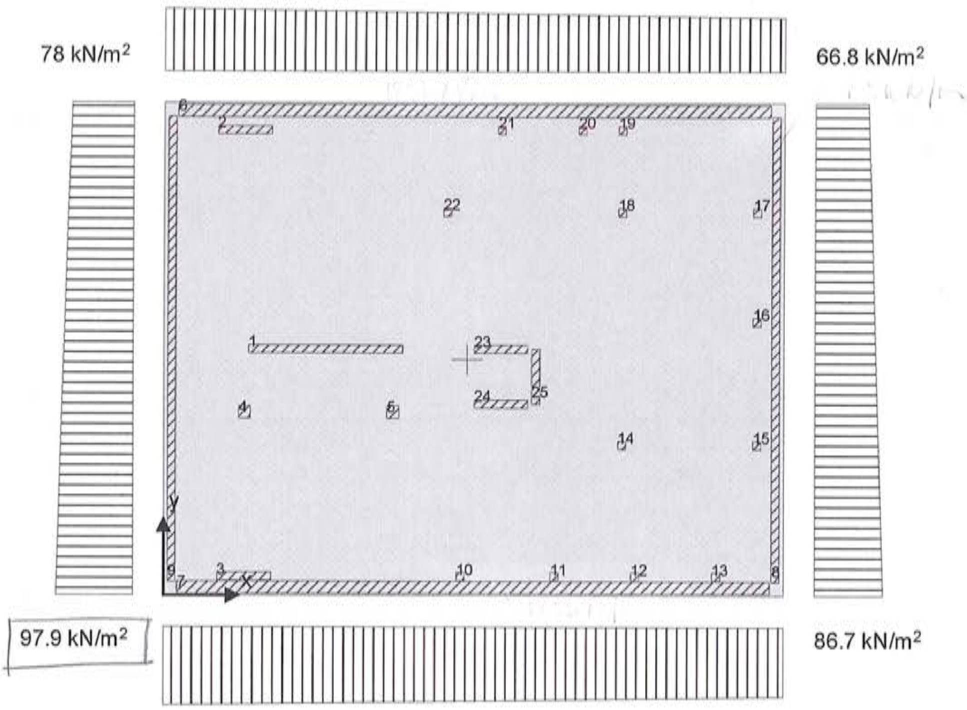
FOUNDATION ANALYSIS (EN1997-1:2004)

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

TEDDS calculation version 3.2.05

Pad foundation details

Length of foundation	L_x = **23000** mm
Width of foundation	L_y = **18000** mm
Foundation area	$A = L_x \times L_y$ = **414.000** m^2
Depth of foundation	h = **450** mm
Depth of soil over foundation	h_{soil} = **600** mm
Level of water	h_{water} = **0** mm
Density of water	γ_{water} = **9.8** kN/m^3
Density of concrete	γ_{conc} = **24.5** kN/m^3



Column no.1 details

Length of column	l_{x1} = **5700** mm
Width of column	l_{y1} = **300** mm
position in x-axis	x_1 = **6000** mm
position in y-axis	y_1 = **9000** mm

Column no.2 details

Length of column	l_{x2} = **2000** mm
Width of column	l_{y2} = **300** mm
position in x-axis	x_2 = **3000** mm
position in y-axis	y_2 = **17000** mm

Column no.3 details

Length of column	l_{x3} = **2000** mm
Width of column	l_{y3} = **300** mm

position in x-axis x_3 = **3000** mm
position in y-axis y_3 = **700** mm

Column no.4 details
Length of column l_{x4} = **450** mm
Width of column l_{y4} = **450** mm
position in x-axis x_4 = **3000** mm
position in y-axis y_4 = **6700** mm

Column no.5 details
Length of column l_{x5} = **450** mm
Width of column l_{y5} = **450** mm
position in x-axis x_5 = **8500** mm
position in y-axis y_5 = **6700** mm

Column no.6 details
Length of column l_{x6} = **22000** mm
Width of column l_{y6} = **450** mm
position in x-axis x_6 = **11500** mm
position in y-axis y_6 = **17700** mm

Column no.7 details
Length of column l_{x7} = **22000** mm
Width of column l_{y7} = **450** mm
position in x-axis x_7 = **11500** mm
position in y-axis y_7 = **300** mm

Column no.8 details
Length of column l_{x8} = **300** mm
Width of column l_{y8} = **17000** mm
position in x-axis x_8 = **22700** mm
position in y-axis y_8 = **9000** mm

Column no.9 details
Length of column l_{x9} = **300** mm
Width of column l_{y9} = **17000** mm
position in x-axis x_9 = **300** mm
position in y-axis y_9 = **9000** mm

Column no.10 details
Length of column l_{x10} = **300** mm
Width of column l_{y10} = **300** mm
position in x-axis x_{10} = **11000** mm
position in y-axis y_{10} = **700** mm

Column no.11 details
Length of column l_{x11} = **300** mm
Width of column l_{y11} = **300** mm
position in x-axis x_{11} = **14500** mm
position in y-axis y_{11} = **700** mm

Column no.12 details
Length of column l_{x12} = **300** mm
Width of column l_{y12} = **300** mm
position in x-axis x_{12} = **17500** mm

position in y-axis y_{12} = **700** mm

Column no.13 details

Length of column l_{x13} = **300** mm
Width of column l_{y13} = **300** mm
position in x-axis x_{13} = **20500** mm
position in y-axis y_{13} = **700** mm

Column no.14 details

Length of column l_{x14} = **300** mm
Width of column l_{y14} = **300** mm
position in x-axis x_{14} = **17000** mm
position in y-axis y_{14} = **5500** mm

Column no.15 details

Length of column l_{x15} = **300** mm
Width of column l_{y15} = **300** mm
position in x-axis x_{15} = **22000** mm
position in y-axis y_{15} = **5500** mm

Column no.16 details

Length of column l_{x16} = **300** mm
Width of column l_{y16} = **300** mm
position in x-axis x_{16} = **22000** mm
position in y-axis y_{16} = **10000** mm

Column no.17 details

Length of column l_{x17} = **300** mm
Width of column l_{y17} = **300** mm
position in x-axis x_{17} = **22000** mm
position in y-axis y_{17} = **14000** mm

Column no.18 details

Length of column l_{x18} = **300** mm
Width of column l_{y18} = **300** mm
position in x-axis x_{18} = **17000** mm
position in y-axis y_{18} = **14000** mm

Column no.19 details

Length of column l_{x19} = **300** mm
Width of column l_{y19} = **300** mm
position in x-axis x_{19} = **17000** mm
position in y-axis y_{19} = **17000** mm

Column no.20 details

Length of column l_{x20} = **300** mm
Width of column l_{y20} = **300** mm
position in x-axis x_{20} = **15500** mm
position in y-axis y_{20} = **17000** mm

Column no.21 details

Length of column l_{x21} = **300** mm
Width of column l_{y21} = **300** mm
position in x-axis x_{21} = **12500** mm
position in y-axis y_{21} = **17000** mm

Column no.22 details

Length of column	l_{x22} = **300** mm
Width of column	l_{y22} = **300** mm
position in x-axis	x_{22} = **10500** mm
position in y-axis	y_{22} = **14000** mm

Column no.23 details

Length of column	l_{x23} = **2000** mm
Width of column	l_{y23} = **300** mm
position in x-axis	x_{23} = **12500** mm
position in y-axis	y_{23} = **9000** mm

Column no.24 details

Length of column	l_{x24} = **2000** mm
Width of column	l_{y24} = **300** mm
position in x-axis	x_{24} = **12500** mm
position in y-axis	y_{24} = **7000** mm

Column no.25 details

Length of column	l_{x25} = **300** mm
Width of column	l_{y25} = **2000** mm
position in x-axis	x_{25} = **13800** mm
position in y-axis	y_{25} = **8000** mm

Soil properties

Density of soil	γ_{soil} = **18.0** kN/m^3
Characteristic cohesion	c'_k = **0** kN/m^2
Characteristic effective shear resistance angle	ϕ'_k = **30** deg
Characteristic friction angle	δ_k = **20** deg

Foundation loads

Permanent surcharge load	F_{Gsur} = **1.5** kN/m^2
Variable surcharge load	F_{Qsur} = **2.5** kN/m^2
Self weight	$F_{swt} = h \times \gamma_{conc}$ = **11.0** kN/m^2
Soil weight	$F_{soil} = h_{soil} \times \gamma_{soil}$ = **10.8** kN/m^2

Column no.1 loads

Permanent load in z	F_{Gz1} = **1280.0** kN
Variable load in z	F_{Qz1} = **1000.0** kN

Column no.2 loads

Permanent load in z	F_{Gz2} = **700.0** kN
Variable load in z	F_{Qz2} = **500.0** kN

Column no.3 loads

Permanent load in z	F_{Gz3} = **875.0** kN
Variable load in z	F_{Qz3} = **500.0** kN

Column no.4 loads

Permanent load in z	F_{Gz4} = **375.0** kN
Variable load in z	F_{Qz4} = **300.0** kN

Column no.5 loads

Permanent load in z	F_{Gz5} = **600.0** kN
Variable load in z	F_{Qz5} = **500.0** kN

Column no.6 loads

Permanent load in z F_{Gz6} = **1750.0** kN

Variable load in z F_{Qz6} = **1000.0** kN

Column no.7 loads

Permanent load in z F_{Gz7} = **1750.0** kN

Variable load in z F_{Qz7} = **1000.0** kN

Column no.8 loads

Permanent load in z F_{Gz8} = **225.0** kN

Variable load in z F_{Qz8} = **200.0** kN

Column no.9 loads

Permanent load in z F_{Gz9} = **225.0** kN

Variable load in z F_{Qz9} = **200.0** kN

Column no.10 loads

Permanent load in z F_{Gz10} = **600.0** kN

Variable load in z F_{Qz10} = **400.0** kN

Column no.11 loads

Permanent load in z F_{Gz11} = **300.0** kN

Variable load in z F_{Qz11} = **250.0** kN

Column no.12 loads

Permanent load in z F_{Gz12} = **200.0** kN

Variable load in z F_{Qz12} = **200.0** kN

Column no.13 loads

Permanent load in z F_{Gz13} = **100.0** kN

Variable load in z F_{Qz13} = **100.0** kN

Column no.14 loads

Permanent load in z F_{Gz14} = **450.0** kN

Variable load in z F_{Qz14} = **400.0** kN

Column no.15 loads

Permanent load in z F_{Gz15} = **150.0** kN

Variable load in z F_{Qz15} = **200.0** kN

Column no.16 loads

Permanent load in z F_{Gz16} = **350.0** kN

Variable load in z F_{Qz16} = **300.0** kN

Column no.17 loads

Permanent load in z F_{Gz17} = **200.0** kN

Variable load in z F_{Qz17} = **100.0** kN

Column no.18 loads

Permanent load in z F_{Gz18} = **400.0** kN

Variable load in z F_{Qz18} = **300.0** kN

Column no.19 loads

Permanent load in z F_{Gz19} = **100.0** kN

Variable load in z F_{Qz19} = **50.0** kN

Column no.20 loads

Permanent load in z F_{Gz20} = **200.0** kN

Variable load in z F_{Qz20} = **150.0** kN

Column no.21 loads

Permanent load in z F_{Gz21} = **375.0** kN

Variable load in z F_{Qz21} = **300.0** kN

Column no.22 loads

Permanent load in z F_{Gz22} = **500.0** kN

Variable load in z F_{Qz22} = **300.0** kN

Column no.23 loads

Permanent load in z F_{Gz23} = **700.0** kN

Variable load in z F_{Qz23} = **450.0** kN

Column no.24 loads

Permanent load in z F_{Gz24} = **700.0** kN

Variable load in z F_{Qz24} = **450.0** kN

Column no.25 loads

Permanent load in z F_{Gz25} = **700.0** kN

Variable load in z F_{Qz25} = **450.0** kN

Bearing resistance (Section 6.5.2)

Forces on foundation

Force in z-axis

$F_{dz} = A \times (F_{swt} + F_{soil} + F_{Gsur} + F_{Qsur}) + F_{Gz1} + F_{Gz2} + F_{Gz3} + F_{Gz4} + F_{Gz5} + F_{Gz6} + F_{Gz7} + F_{Gz8} + F_{Gz9} + F_{Gz10} + F_{Gz11} + F_{Gz12} + F_{Gz13} + F_{Gz14} + F_{Gz15} + F_{Gz16} + F_{Gz17} + F_{Gz18} + F_{Gz19} + F_{Gz20} + F_{Gz21} + F_{Gz22} + F_{Gz23} + F_{Gz24} + F_{Gz25} + F_{Qz1} + F_{Qz2} + F_{Qz3} + F_{Qz4} + F_{Qz5} + F_{Qz6} + F_{Qz7} + F_{Qz8} + F_{Qz9} + F_{Qz10} + F_{Qz11} + F_{Qz12} + F_{Qz13} + F_{Qz14} + F_{Qz15} + F_{Qz16} + F_{Qz17} + F_{Qz18} + F_{Qz19} + F_{Qz20} + F_{Qz21} + F_{Qz22} + F_{Qz23} + F_{Qz24} + F_{Qz25} = \mathbf{34096.6}$ kN

Moments on foundation

Moment in x-axis

$M_{dx} = A \times (F_{swt} + F_{soil} + F_{Gsur} + F_{Qsur}) \times L_x / 2 + F_{Gz1} \times x_1 + F_{Gz2} \times x_2 + F_{Gz3} \times x_3 + F_{Gz4} \times x_4 + F_{Gz5} \times x_5 + F_{Gz6} \times x_6 + F_{Gz7} \times x_7 + F_{Gz8} \times x_8 + F_{Gz9} \times x_9 + F_{Gz10} \times x_{10} + F_{Gz11} \times x_{11} + F_{Gz12} \times x_{12} + F_{Gz13} \times x_{13} + F_{Gz14} \times x_{14} + F_{Gz15} \times x_{15} + F_{Gz16} \times x_{16} + F_{Gz17} \times x_{17} + F_{Gz18} \times x_{18} + F_{Gz19} \times x_{19} + F_{Gz20} \times x_{20} + F_{Gz21} \times x_{21} + F_{Gz22} \times x_{22} + F_{Gz23} \times x_{23} + F_{Gz24} \times x_{24} + F_{Gz25} \times x_{25} + F_{Qz1} \times x_1 + F_{Qz2} \times x_2 + F_{Qz3} \times x_3 + F_{Qz4} \times x_4 + F_{Qz5} \times x_5 + F_{Qz6} \times x_6 + F_{Qz7} \times x_7 + F_{Qz8} \times x_8 + F_{Qz9} \times x_9 + F_{Qz10} \times x_{10} + F_{Qz11} \times x_{11} + F_{Qz12} \times x_{12} + F_{Qz13} \times x_{13} + F_{Qz14} \times x_{14} + F_{Qz15} \times x_{15} + F_{Qz16} \times x_{16} + F_{Qz17} \times x_{17} + F_{Qz18} \times x_{18} + F_{Qz19} \times x_{19} + F_{Qz20} \times x_{20} + F_{Qz21} \times x_{21} + F_{Qz22} \times x_{22} + F_{Qz23} \times x_{23} + F_{Qz24} \times x_{24} + F_{Qz25} \times x_{25} = \mathbf{383215.3}$ kNm

Moment in y-axis

$M_{dy} = A \times (F_{swt} + F_{soil} + F_{Gsur} + F_{Qsur}) \times L_y / 2 + F_{Gz1} \times y_1 + F_{Gz2} \times y_2 + F_{Gz3} \times y_3 + F_{Gz4} \times y_4 + F_{Gz5} \times y_5 + F_{Gz6} \times y_6 + F_{Gz7} \times y_7 + F_{Gz8} \times y_8 + F_{Gz9} \times y_9 + F_{Gz10} \times y_{10} + F_{Gz11} \times y_{11} + F_{Gz12} \times y_{12} + F_{Gz13} \times y_{13} + F_{Gz14} \times y_{14} + F_{Gz15} \times y_{15} + F_{Gz16} \times y_{16} + F_{Gz17} \times y_{17} + F_{Gz18} \times y_{18} + F_{Gz19} \times y_{19} + F_{Gz20} \times y_{20} + F_{Gz21} \times y_{21} + F_{Gz22} \times y_{22} + F_{Gz23} \times y_{23} + F_{Gz24} \times y_{24} + F_{Gz25} \times y_{25} + F_{Qz1} \times y_1 + F_{Qz2} \times y_2 + F_{Qz3} \times y_3 + F_{Qz4} \times y_4 + F_{Qz5} \times y_5 + F_{Qz6} \times y_6 + F_{Qz7} \times y_7 + F_{Qz8} \times y_8 + F_{Qz9} \times y_9 + F_{Qz10} \times y_{10} + F_{Qz11} \times y_{11} + F_{Qz12} \times y_{12} + F_{Qz13} \times y_{13} + F_{Qz14} \times y_{14} + F_{Qz15} \times y_{15} + F_{Qz16} \times y_{16} + F_{Qz17} \times y_{17} + F_{Qz18} \times$

$y_{18} + F_{Qz19} \times y_{19} + F_{Qz20} \times y_{20} + F_{Qz21} \times y_{21} + F_{Qz22} \times y_{22} + F_{Qz23} \times y_{23} + F_{Qz24} \times y_{24} + F_{Qz25} \times y_{25} =$ **294529.0** kNm

Eccentricity of base reaction

Eccentricity of base reaction in x-axis $e_x = M_{dx} / F_{dz} - L_x / 2 =$ **-261** mm

Eccentricity of base reaction in y-axis $e_y = M_{dy} / F_{dz} - L_y / 2 =$ **-362** mm

Pad base pressures

$q_1 = F_{dz} \times (1 - 6 \times e_x / L_x - 6 \times e_y / L_y) / (L_x \times L_y) =$ **97.9** kN/m²

$q_2 = F_{dz} \times (1 - 6 \times e_x / L_x + 6 \times e_y / L_y) / (L_x \times L_y) =$ **78** kN/m²

$q_3 = F_{dz} \times (1 + 6 \times e_x / L_x - 6 \times e_y / L_y) / (L_x \times L_y) =$ **86.7** kN/m²

$q_4 = F_{dz} \times (1 + 6 \times e_x / L_x + 6 \times e_y / L_y) / (L_x \times L_y) =$ **66.8** kN/m²

Minimum base pressure $q_{min} = \min(q_1, q_2, q_3, q_4) =$ **66.8** kN/m²

Maximum base pressure $q_{max} = \max(q_1, q_2, q_3, q_4) =$ **97.9** kN/m²

Presumed bearing capacity

Presumed bearing capacity $P_{bearing} =$ **165.0** kN/m²

PASS - Presumed bearing capacity exceeds design base pressure

1.0 Introduction

Applied Geotechnical Engineering (AGE) has carried out an analysis of the predicted ground movement associated with the proposed basement construction at Nos 115-119 Goldhurst Terrace, London NW6, and on the basis of that assessment has carried out a Burland Damage Category Assessment on the neighbouring properties. This work is presented in AGE report Ref P4134Rev1 dated 21/6/16.

In the ground movement analysis the stiffness of the London Clay was treated as non-linear, and based upon recent high-quality case-history and published data.

On the insistence of Cambell Reith (CR) (the checker of that previous report) AGE has carried out a repeat analysis on the basis of London Clay stiffness data that CR view as more appropriate. The amendments required by CR are:-

i) That London Clay stiffness be quoted at a strain level of 0.1%, not 0.001%
ii) That the degradation curve be based upon 'accepted' data, namely the three curves (or tabulated data) presented in Appendix A of this addendum.

The methods of analysis are as described in the original AGE report, Section 5.

In the current addendum the two walls predicted to suffer the greatest damage in the original analysis have been re-analysed using the CR soil stiffness values.

The parts of the report text that have changed as a result of that re-analysis are given below, with the section numbering adopted from the original report for ease of reference. Similarly, the ground movement predictions relating to the critical walls are given in the attached figures, numbered as in the original report. These replacements can be taken to replace the original sections of the original report. In all other respects the original report remains unchanged.

2.0 Information Provided

i) SAS Borehole, Window Sampler and trial pit logs dated 14-16/3/2016.
ii) EW Drawing 2150657/SK01P2, annotations to Interlock Surveys topo drawing 150683, and sketches ref 2150657-01 and 02, giving proposed, existing, and construction loads.
iii) EW Sketches 2150657 SK/08P1, 09P1 and 11P1.
iv) KSR Architects Drawings 15033/P090(revised 10/6/16), P100, P210, P210, P212, P213, P310, P311
v) Interlock Surveys topographical survey drawings 150683 and 150683ELE.
vi) Cambell Reith London Clay stiffness data (see addendum Appendix A).
vii) Email correspondence SAS/ElliottWood - AGE dated 1/4/16 to 16/11/16.

5.2 Soil stiffness values

An equivalent-elastic analysis has been carried out using the program PDisp. The program takes no account of structural (building) stiffness.

The soil stiffness parameters are as given below.

The Made Ground lies above founding level and excavation level, and therefore will not influence the analysis.

The London Clay has been treated as a non-linear material. The undrained stiffness at 0.1% strain is taken as E_u = 630Su based upon an average of the stiffness data provided by Campbell Reith. Taking Poisson's ratio as 0.2 in the drained case, a drained stiffness (E') of 504Su is obtained at 0.1% strain.

Yielding :-

$E_{u\,0.1\%} = 28.3 + 4.4z_1$ (MPa) to 18mOD (20m below top of clay), then
$E_{u\,0.1\%} = 116 + 2.2z_2$ (MPa) to -22mOD (base of clay).

and:-

$E'_{0.1\%} = 22.7 + 3.5z_1$ (MPa) to 18mOD (20m below top of clay), then
$E'_{0.1\%} = 92.5 + 1.75z_2$ (MPa) to –22mOD (base of clay).

Where z_1 is the depth in metres below the top of the London Clay (taken to be 38mOD), and z_2 is the depth in metres below 18mOD.

A non-linear degradation curve relating stiffness to strain, based on the Campbell Reith data given in Appendix A has been used.

5.5 Predicted movement – Nos 111+113 Goldhurst Terrace, rear wall.

5.5.1 Vertical Movement

Profiles of short- and long-term vertical ground movement along the rear wall of Nos 111 and 113 Goldhurst Terrace have been calculated and plotted in Figure 6.

The wall is taken to be approximately 11.9m long and 9m high above ground level. It lies in the position shown on the plan in Figure 6.

The analysis indicates a maximum overall tilt of 4.8mm along the length of the wall. This equates to a whole-wall gradient of less than 1 in 2400. This is less than the 1:400 gradient recognised as requiring remedial action.

The maximum predicted wall distortion (Delta – as defined by Burland, Ref 2) is) 1.65mm within the length of the wall. The limit on tensile strain for 'very slight' damage is 0.075% (Ref 2), therefore the ratio of deflection ratio to limiting tensile strain is 0.185. By reference to Figure 4 (Ref 2 Figure 6) a horizontal strain/limiting tensile strain ratio of 0.83 is obtained, indicating that a horizontal strain of 0.062% is acceptable for a 'very slight' category of damage. The analysis does not take into account the stiffness of the wall and is conservative in this respect.

5.5.2 Lateral movement.

From Section 5.3 above, the greatest average horizontal ground strain adjacent to the proposed excavation at Nos 115-119 is predicted to be 0.064%. This is greater than the 0.062% limit for very slight damage calculated above, indicating that damage may lie at the lower end of the 'slight' category, which in this case extends from 0.062% to 0.137%.

However the maximum average horizontal strain is predicted only to extend 6.1m from the excavation, and beyond this distance the average horizontal strain reduces to 0.0375%. Furthermore, the analysis does not take into account the horizontal stiffness of the wall, or the fact that the predicted mode of distortion is sagging, which is less damaging than the hogging mode considered by Burland in his analysis. It is therefore considered that the predicted level of damage to this wall can be taken to lie close to the 'very slight'/'slight' boundary. Particular care in the propping of the excavation will be required at this location.

5.9 Predicted movement – Nos 121 to 125 Goldhurst Terrace, rear wall.

5.9.1 Vertical Movement

Profiles of short- and long-term vertical ground movement along the rear wall of Nos 121 to 125 Goldhurst Terrace have been calculated and plotted in Figure 10.

This wall is taken to be approximately 18.8m long and approximately 9m high, above ground level. It lies in the position shown on the plan in Figure 10.

The analysis indicates a maximum overall tilt of approximately 4.9mm along the length of this wall. This equates to a whole-wall gradient of less than 1 in 3800. This is less than the 1:400 gradient recognised as requiring remedial action.

Two modes of distortion are evident from Figure 10; sagging close to the excavation, and hogging over a greater part of the wall. Hogging is usually the more damaging mode, but from Section 5.3 above it is noted that maximum average horizontal strain only occurs within approximately 1.5 x wall depth of the excavation, in this case this is approximately 6.1m (to X= 23.3m in Figure 9). Therefore the local sagging is considered to be more critical.

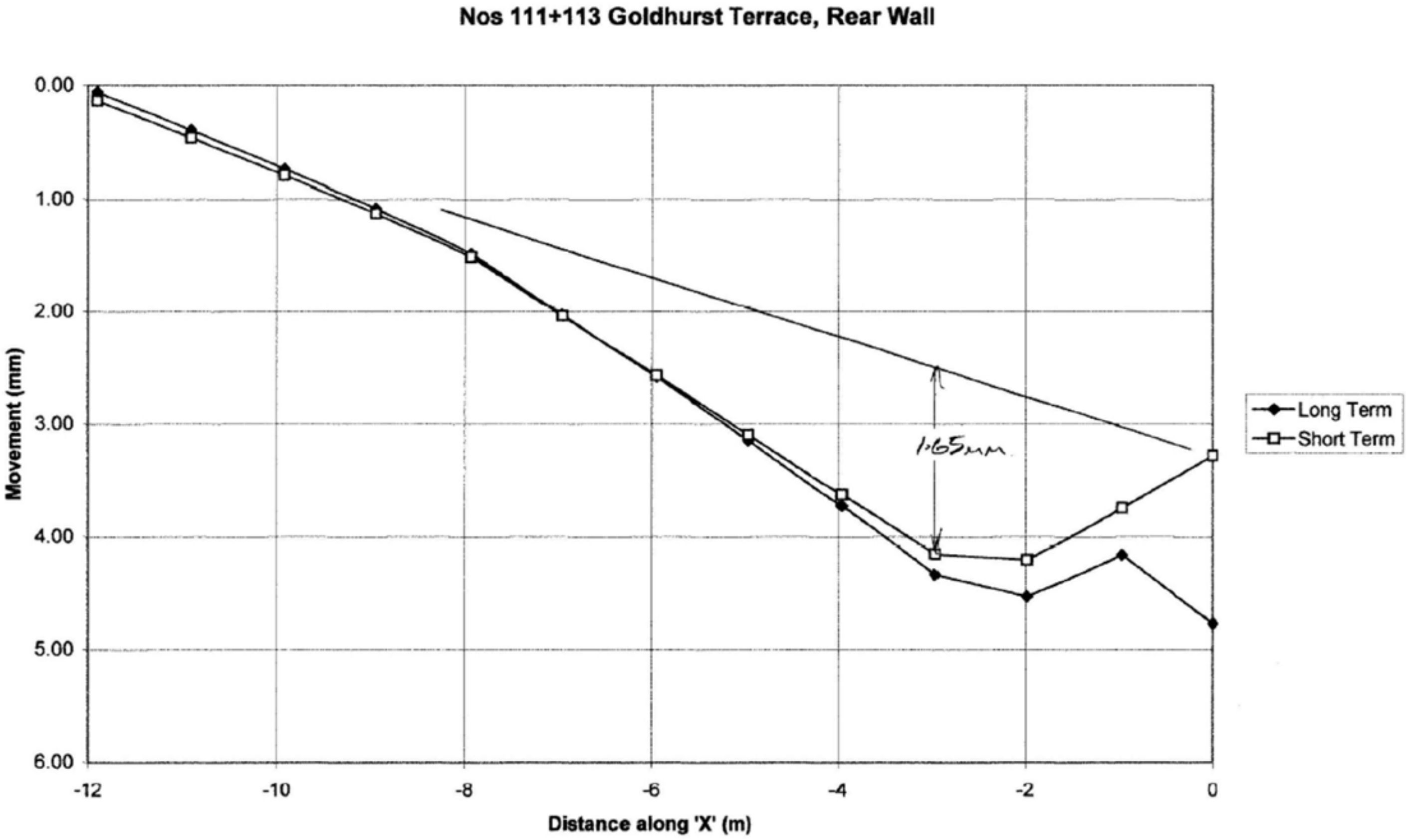
This predicted sagging wall distortion (Delta – as defined by Burland, Ref 2) is 1.6mm within a 10.4m length of the wall. The limit on tensile strain for 'very slight' damage is 0.075% (Ref 2), therefore the ratio of deflection ratio to limiting tensile strain is 0.20. By reference to Figure 4 (Ref 2 Figure 6) a horizontal strain/limiting tensile strain ratio of 0.83 is obtained, indicating that a horizontal strain of 0.062% is acceptable for a 'very slight' category of damage.

This result does not take into account the vertical stiffness of the wall, and is conservative in this respect.

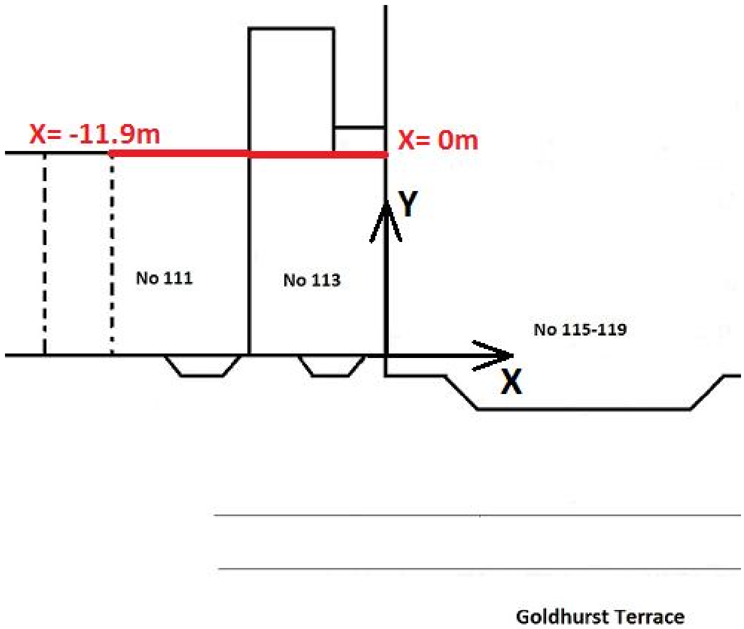
5.9.2 Lateral movement.

From Section 5.3 above, the greatest average horizontal ground strain adjacent to the proposed excavation at Nos 115-119 is predicted to be 0.064%. This is greater than the 0.062% limit for very slight damage calculated above, indicating that damage may lie at the lower end of the 'slight' category, which in this case extends from 0.062% to 0.137%.

However the maximum average horizontal strain is predicted only to extend 6.1m from the excavation, and beyond this distance the average horizontal strain reduces to 0.0375%. Furthermore, the analysis does not take into account the horizontal stiffness of the wall, or the fact that the predicted mode of distortion is sagging, which is less damaging than the hogging mode considered by Burland in his analysis. It is therefore considered that the predicted level of damage to this wall can be taken to lie close to the 'very slight'/'slight' boundary. Particular care in the propping of the excavation will be required at this location.

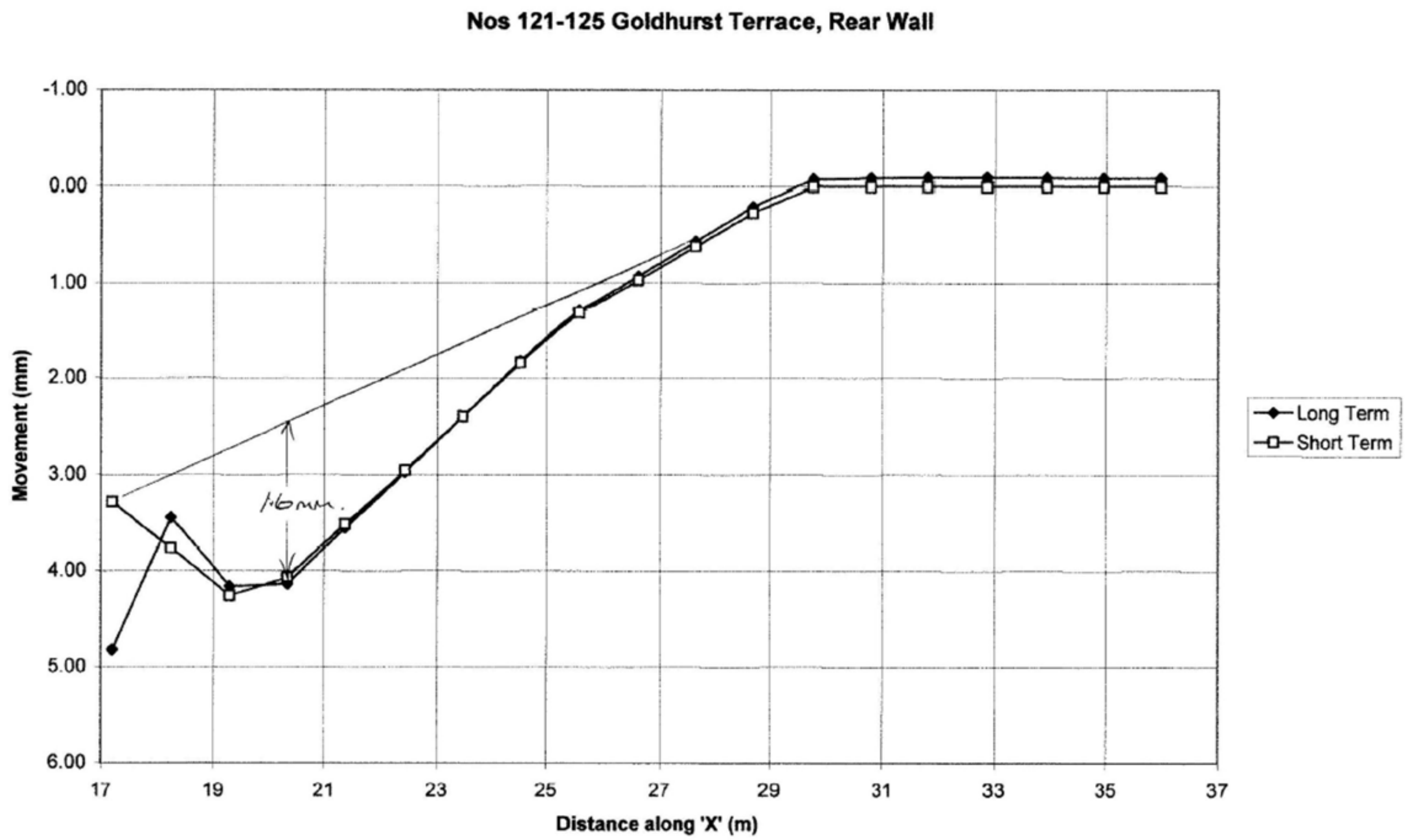


Nos 111+113 Goldhurst Terrace, Rear Wall
0.00
1.00
2.00
3.00
4.00
5.00
6.00
Movement (mm)
-12
-10
-8
-6
-4
-2
0
Distance along 'X' (m)
1.65mm
Long Term
Short Term

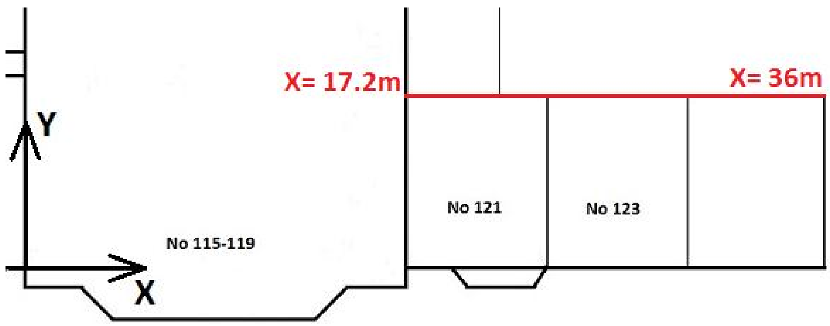


X= -11.9m
X= 0m
Y
X
No 111
No 113
No 115-119
Goldhurst Terrace

Figure 6



Nos 121-125 Goldhurst Terrace, Rear Wall
Movement (mm)
-1.00
0.00
1.00
2.00
3.00
4.00
5.00
6.00
17
19
21
23
25
27
29
31
33
35
37
Distance along 'X' (m)
Long Term
Short Term
1.6mm.



X= 17.2m
X= 36m
Y
X
No 115-119
No 121
No 123

Figure 10

Appendix A
Soil stiffness values proposed by Campbell Reith.

'From Borin'

Over-consolidated cohesionless soil
The Young's modulus of over-consolidated (compacted) cohesionless soil is approximately proportional to its SPT N value according to the equation:

Young's modulus (kN/m²) = F x SPT N value

where F is in the range 2000 to 6000 for retaining walls in sands and gravels.

6.9.2 Young's Modulus of Cohesive soil

Modulus of undrained cohesive soil
The following table may be used as a preliminary guide to values of Young's Modulus for cohesive soil:-

	Consistency		
	soft	firm	stiff
Undrained shear strength, kN/m²	20 - 40	40 - 75	75 - 150
Young's Modulus kN/m²	1600 - 6000	6000 - 20000	20000 - 75000

Table 6.3 Approximate values of Young's Modulus for cohesive soil

The stress-strain behaviour of clays is non-linear and so the value of E_u itself depends on the strain level at which the modulus is measured. The value of E_u entered in the data should therefore relate to the magnitude of the strains which occur during excavation. In the absence of direct measurements, E_u may be derived from published correlations between E_u and undrained shear strength, c_u. The relationship is of the form:-

$$E_u = M.c_u \qquad 6.8$$

where M depends on the strain at which E_u is measured. The following table is based on data from Burland et al. 1979:-

Strain level	E_u/c_u (=M)	Factor
2.0%	150	0.18
1.0%	250	0.33
0.4%	400	0.5
0.2%	600	0.75
0.1%	800	1

Table 6.4 Approximate relationship between Young's Modulus and undrained cohesion

Modulus of drained cohesive soil
The drained modulus, E' (at any particular strain level) will differ from the undrained modulus, E_u. Wroth (1972) has shown that they are related by the equation:-

$$E' = {}^2/_3\,(1+\nu)\ E_u \qquad 6.9$$

but for drained clay:-

$$\nu = 0.15 \text{ (approx.)}$$

therefore substituting in Eq.6.9 gives:

$$E' = 0.77\ E_u \qquad 6.10$$

Anisotropic modulus of drained modulus
Henkel (1972) has shown that for heavily overconsolidated clays the effects of anisotropy can be significant, with the drained horizontal Young's modulus, E'_h being greater than the drained vertical modulus E'_v. For example in London Clay:-

$$E'_h = 1.6\ E'_v = 1.23\ E_u \qquad 6.11$$

Since we are dealing with the **horizontal** response to horizontal changes of wall pressure we should use values of E'_h. However on account of the uncertainties inherent in measuring or estimating soil modulus, and the insensitivity of designs to modulus values the values given in Tables 6.3 and 6.4 may be used for both drained and undrained modulus.

'From Tomlinson'

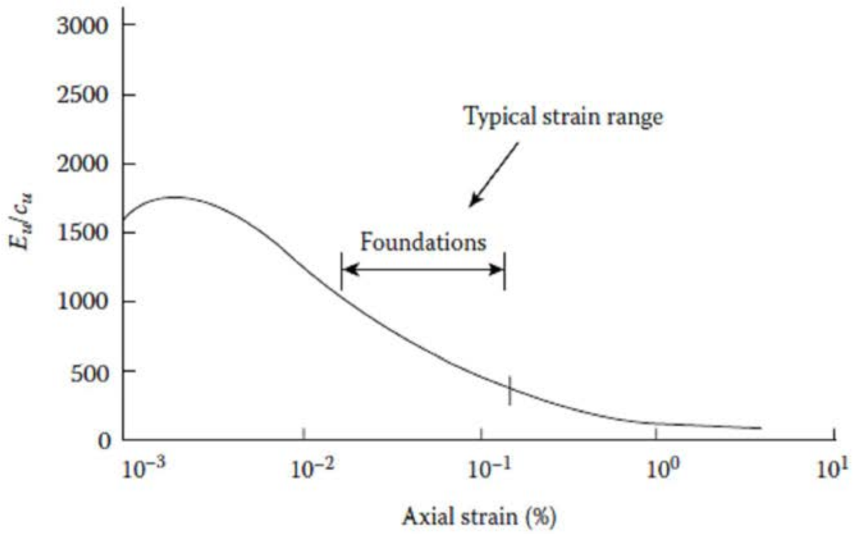


Figure 5.15 Relationship between E_u/c_u and axial strain. (After Jardine, R. et al., Field and laboratory measurements of soil stiffness, *Proceedings of the 11th International Conference on Soil Mechanics*, San Francisco, CA, Vol. 2, pp. 511–514, 1985.)

'From CIRIA C580'

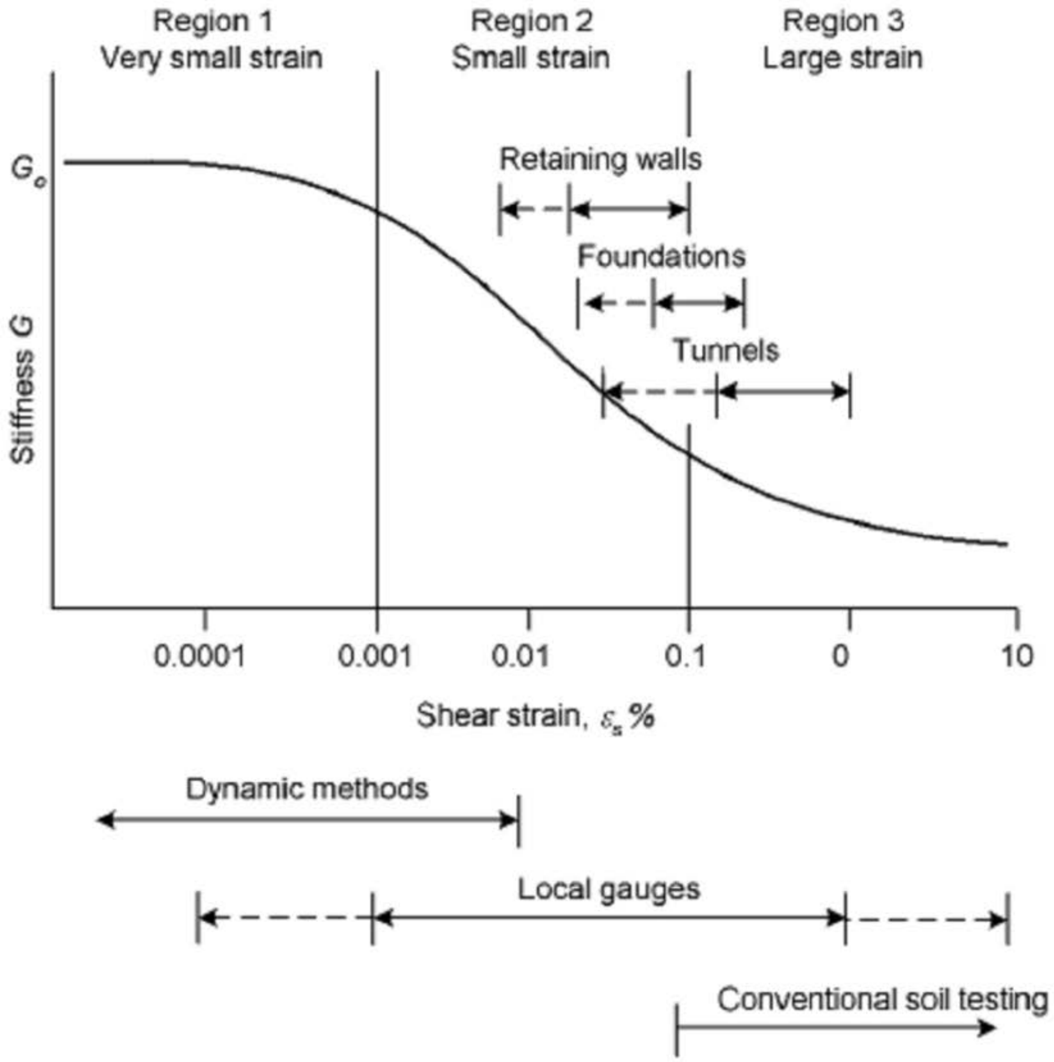


Figure 5.12 *Stiffness–strain behaviour of soil with typical strain ranges for laboratory tests and structures (after Atkinson and Sällfors, 1991 and Mair, 1993)*

Mayron House :Goldhurst Terrace
David Whittington
to:
'GrahamKite@campbellreith.com', 'HiepPham@campbellreith.com'
02/12/2016 15:52
Cc:
"'j.mill@elliottwood.co.uk'", Chris Ball
Hide Details
From: David Whittington <DWhittington@savills.com>
To: "'GrahamKite@campbellreith.com'" <GrahamKite@campbellreith.com>, "'HiepPham@campbellreith.com'" <HiepPham@campbellreith.com>
Cc: "'j.mill@elliottwood.co.uk'" <j.mill@elliottwood.co.uk>, Chris Ball <COBall@savills.com>

1 Attachment

160912 - Goldhurst Terrace raft calculations.pdf

Dear Graham

I understand from Jess Mill at Elliott Wood that you are missing 2 final pieces of information relating to the BIA.

1: Confirmation that qualifications of authors/reviewers of the Structural Engineering Report comply with requirements of CPG 4

- Please find confirmation of the relevant qualifications below:

Prepared by:	Jess Mill	MEng (Hons)
Checked by:	Justin Gathercole	MEng (Hons) CEng MIStructE
	Gemima Walker	MEng (Hons) CEng MICE
Approved by:	Henry Murray	MEng (Hons) CEng MIStructE

2. Checking the adequacy of the bearing stratum and calculations of basement raft slab.

- Please see calculations attached, which include the design of the basement raft to resist heave. We believe that this would be a better solution, but if it is not deemed acceptable then we will update to include heave protection as discussed with Campbell Reith last week.

Regards

David

David Whittington BA (Hons) DipTP
Director
Planning

Savills, 33 Margaret Street , London W1G 0JD
Tel :+44 (0) 20 7557 9997
Mobile :+44 (0) 7717 897 465
Email :DWhittington@savills.com
Website :www.savills.co.uk

London

Friars Bridge Court
41- 45 Blackfriars Road
London, SE1 8NZ

T: +44 (0)20 7340 1700
E: london@campbellreith.com

Birmingham

Chantry House
High Street, Coleshill
Birmingham B46 3BP

T: +44 (0)1675 467 484
E: birmingham@campbellreith.com

Surrey

Raven House
29 Linkfield Lane, Redhill
Surrey RH1 1SS

T: +44 (0)1737 784 500
E: surrey@campbellreith.com

Manchester

No. 1 Marsden Street
Manchester
M2 1HW

T: +44 (0)161 819 3060
E: manchester@campbellreith.com

Bristol

Wessex House
Pixash Lane, Keynsham
Bristol BS31 1TP

T: +44 (0)117 916 1066
E: bristol@campbellreith.com

UAE

Office 705, Warsan Building
Hessa Street (East)
PO Box 28064, Dubai, UAE

T: +971 4 453 4735
E: uae@campbellreith.com

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A list of Members is available at our Registered Office at: Friars Bridge Court, 41- 45 Blackfriars Road, London SE1 8NZ
VAT No 974 8892 43