CampbellReith
consulting engineers

66 Fitzjohn's Avenue, London, NW1 0AA

Basement Impact Assessment Audit

For

London Borough of Camden

Project Number: 12066-98
Revision: D2

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Project Name	66 Fitzjohn's Avenue
Planning Reference	2015/5847/P

Contents

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 66 Fitzjohn's Avenue, London NW3 5LT (planning reference 2015/5847/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 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. The BIA and Hydrology BIA were completed by competent consultants suitably qualified in accordance with CPG4.

1.5. The proposed works consist of the demolition of the existing above ground two storey building and the construction of a three storey building above ground with basement below.

1.6. The BIA has confirmed that the ground conditions comprise Made Ground over the Claygate Member and then London Clay. Monitoring of a single borehole has shown a groundwater level approximately 0.50m above the proposed top of floor slab level and additional groundwater monitoring is recommended.

1.7. No geotechnical laboratory tests, interpretation or proposed geotechnical parameters for design were provided in the BIA. It has been confirmed that no laboratory testing was undertaken. Whilst this is not the best practice, it is accepted that parameters for detailed design can be agreed with the party wall surveyor.

1.8. Nearby foundations have been assumed to be shallow strips and the presence of a semi-basement to No. 64 Fitzjohn's Avenue has been confirmed. Other properties are remote from the proposed basement.

1.9. It is accepted that the surrounding slopes to the development site are stable.

1.10. The proposed construction method for the basement is to be a propped bored pile, secant retaining wall. Indicative calculations for the retaining walls and floor slab have been submitted, together with an indicative construction sequence demonstrating the principles of design. Although, there are queries with respect to the assumptions made, it is accepted that they are sufficient for planning and detailed design may be agreed with the party wall surveyor.

1.11. It should be ensured that the boundary wall alongside No 64 Fitzjohn's Avenue can support the proposed loadings and vibration associated with construction and whether that area is underlain by a tunnel. Further investigation is required to determine whether a tunnel exists and, if so, suitable mitigation provided.

1.12. The ground movement assessment provided in July 2016 makes allowance for heave due to the overall basement excavation and justifies the assumptions made. It is accepted that, on the assumption of good control of workmanship, damage should be limited to category 0-1 for 64 Fitzjohn's Avenue.

1.13. It has been confirmed whether the removal of the Silver Birch tree will not affect existing and proposed foundations.

1.14. Proposals for monitoring have been provided. The detail and extent of condition surveys may be agreed with the party wall surveyor.

1.15. The flood risk assessment shows the only significant flood risk as blockage of private drainage connections.

1.16. The Historic Shepherds Hill conduit (water course) used to run within 20-40m to the west of the site. Based on this and the groundwater level identified in the borehole, mitigation measures are proposed. The BIA has stated that the development will not impact on the wider hydrogeology of the area, any other watercourses, springs or the Hampstead Heath Pond chain catchment area.

1.17. The proposed development increases the impermeable surface area. Supplementary information provides justification for proposed mitigation measures.

1.18. Queries and requests for clarification are discussed in Section 4 and summarised in Appendix 2.

2.0 INTRODUCTION

2.1. CampbellReith was instructed by London Borough of Camden (LBC) on 5th January 2016 to carry out a Category B Audit on the Basement Impact Assessment (BIA) submitted as part of the Planning Submission documentation for 66 Fitzjohn's Avenue, London NW3 5LT, Planning Reference 2015/5847/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 "Demolition of existing two houses and the erection of two new single family dwellings."

2.6. CampbellReith accessed LBC's Planning Portal on 9th February 2016 and gained access to the following relevant documents for audit purposes:

General Information

- arboricultural report.pdf
- BIA Audit Form.pdf
- BIA.pdf
- Construction Management Plan.pdf
- Design Access statement.pdf
- Hydrological BIA Report.pdf
- Location Plan.pdf
- Planning Application Form.pdf
- PLANNING CMP.pdf
- Planning Policy Statement.pdf

Drawings

- 1169.01.02-Exstng SP(2).pdf
- 1169.01.04-Exstng GF(2).pdf
- 1169.01.05-Exstng RP(2).pdf
- 1169.03.01-Exstng FE(2).pdf
- 1169.03.02-Exstng RE(2).pdf
- 1169.03.03-Exstng SE(2).pdf
- 1169.03.04-Exstng SE(2).pdf

- 1169.01.10(B)-Prpsd SP(2).pdf
- 1169.01.11(C)-Prpsd SP(2).pdf
- 1169.01.12(A)-Prpsd LGF(2).pdf
- 1169.01.13(B)-Prpsd GF(2).pdf
- 1169.01.14-Prpsd FF(2).pdf
- 1169.01.15-Prpsd SF(2).pdf

- 1169.01.16-Prpsd RP(2).pdf
- 1169.01.17-Prpsd CDM(2).pdf
- 1169.02.11-Prpsd AA(2).pdf
- 1169.03.11-Prpsd FE(2).pdf
- 1169.03.12-Prpsd RE(2).pdf
- 1169.03.13-Prpsd SE(2).pdf
- 1169.03.14-Prpsd SE(2).pdf

2.7. Subsequent to the issue of the initial audit report, further information was provided in July 2016 comprising

- Response to queries raised in CampbellReith’s BIA Audit, Michael Chester and Partners, July 2016
- Memorandum, SLR, 29 April 2016:

2.8. That further information is presented in Appendix 3 and considered in this revised audit report. Reference was made to revised drawings and additional consultation responses uploaded on to Camden’s planning website since the previous audit was issued.

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3.0 BASEMENT IMPACT ASSESSMENT AUDIT CHECK LIST

Item	Yes/No/NA	Comment
Are BIA Author(s) credentials satisfactory?	Yes	BIA was by a Chartered Engineer (CEng) who is a Member of the Institution of Structural Engineers. Hydrology BIA by a Chartered Geologist (CGeol). Other (unnamed) contributors have suitable qualifications.
Is data required by Cl.233 of the GSD presented?	Yes	Revised site plan on planning website shows boundary clearly defined. Development occupies almost the whole site apart from an access strip & no temporary land appears to be available for construction.
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 Construction Management Plan (CMP).
Are suitable plan/maps included?	Yes	See BIA, HBIA & Drawings.
Do the plans/maps show the whole of the relevant area of study and do they show it in sufficient detail?	Yes	See BIA, HBIA & Drawings.
Land Stability Screening: Have appropriate data sources been consulted? Is justification provided for 'No' answers?	Yes	See BIA – further assessment needed.
Hydrogeology Screening: Have appropriate data sources been consulted? Is justification provided for 'No' answers?	Yes	See HBIA – further assessment needed.
Hydrology Screening: Have appropriate data sources been consulted? Is justification provided for 'No' answers?	Yes	Appropriate data sources have been consulted but further assessment required.

Item	Yes/No/NA	Comment
Is a conceptual model presented?	Yes	Description is given in HBIA.
Land Stability Scoping Provided? Is scoping consistent with screening outcome?	Yes	Supplementary information considers heave due to excavation and the impact of tree removal, and provides proposals for monitoring.
Hydrogeology Scoping Provided? Is scoping consistent with screening outcome?	Yes	See HBIA. Some increase in ground water level may occur - a French drain & sump are proposed for mitigation. Proposed impermeable "roof" over the basement could result in a local increase in infiltration with potential risk of water emerging into the sunken Patio to No 62 – roof should be laid to fall towards the French drain and sump.
Hydrology Scoping Provided? Is scoping consistent with screening outcome?	Yes	Hydrology scoping is provided and is consistent.
Is factual ground investigation data provided?	No	See HBIA & BIA – laboratory data, ground descriptions not included.
Is monitoring data presented?	Yes	Standpipes - only one result provided. The BIA indicates that monitoring is to be ongoing and we would concur.
Is the ground investigation informed by a desk study?	Yes	See HBIA.
Has a site walkover been undertaken?	Yes	See HBIA & BIA.
Is the presence/absence of adjacent or nearby basements confirmed?	Yes	Supplementary information confirms a semi-basement to No. 64 Fitzjohn's Avenue. Other properties are remote.
Is a geotechnical interpretation presented?	No	Only part of the ground investigation is provided. No laboratory results, descriptions, proposed geotechnical parameters or interpretation are included.
Does the geotechnical interpretation include information on retaining wall design?	No	

Item	Yes/No/NA	Comment
Are reports on other investigations required by screening and scoping presented?	Yes	Additional groundwater monitoring required and provision of further factual and interpretive geotechnical information.
Are the baseline conditions described, based on the GSD?	Yes	See BIA & HBIA.
Do the base line conditions consider adjacent or nearby basements?	Yes	Supplementary information confirms a semi-basement to No. 64 Fitzjohn’s Avenue. Other buildings are remote.
Is an Impact Assessment provided?	Yes	But some issues need to be further reviewed.
Are estimates of ground movement and structural impact presented?	No	Supplementary information includes a comprehensive ground movement/building damage assessment. However, further investigation of a possible “tunnel” is required.
Is the Impact Assessment appropriate to the matters identified by screen and scoping?	No	Further investigation of possible tunnel required.
Has the need for mitigation been considered and are appropriate mitigation methods incorporated in the scheme?	No	It remains to be confirmed whether mitigation measures are required in respect to potential tunnel.
Has the need for monitoring during construction been considered?	Yes	Proposals for monitoring are presented in supplementary information.
Have the residual (after mitigation) impacts been clearly identified?	No	Consideration of possible tunnel required.
Has the scheme demonstrated that the structural stability of the building and neighbouring properties and infrastructure will be maintained?	No	Consideration of possible tunnel required.
Has the scheme avoided adversely affecting drainage and run-off or causing other damage to the water environment?	Yes	See HBIA. Further assessment of the need for a basal drainage layer to the basement and for attenuation of surface water infiltration presented with supplementary informaton.

Item	Yes/No/NA	Comment
Has the scheme avoided cumulative impacts upon structural stability or the water environment in the local area?	Yes	See HBIA and supplementary information.
Does report state that damage to surrounding buildings will be no worse than Burland Category 2?	Yes	Report says damage to No. 64 Fitzjohn’s Avenue will be no worse than Category 1. Other buildings are more remote.
Are non-technical summaries provided?	No	However, the BIA has generally been written in a way that is easy to understand without the use of excessive technical terms.

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4.0 DISCUSSION

4.1. The BIA was carried out by a local Consulting Engineering Practice, Michael Chester & Partners, and was authored by a Chartered Engineer (CEng) who is a Member of the Institution of Structural Engineers.

4.2. The accompanying Hydrology BIA Report (HBIA) by SLR Consulting, was authored by a Chartered Geologist (CGeol). It is stated that other (unnamed) staff involved in the preparation included two hydrogeologists with Chartered Geologist qualifications and one hydrologist who is a Chartered Civil Engineer and holds a Masters Degree in Hydrology. As requested in the initial audit report, these staff should be named together with their relevant qualifications.

4.3. The proposed works consist of the demolition of the existing above ground two storey building and the construction of a three storey building above ground with basement below.

4.4. The proposed above ground building measures approximately 7m x 16m on plan which is a generally similar size to the existing building. The below ground works for the basement, however, measure approximately 12.5 x 16.3m on plan which is almost double the plan area of the existing building. The excavation depth for the basement to the underside of basement slab is approximately 4.5m below the existing ground level. The new basement extends under almost the whole of the existing plot right up to the boundaries with the adjacent properties.

4.5. There is only a narrow access strip alongside No 64 Fitzjohn's Avenue and it has been reported by one of the local residents that there may be some form of tunnel under this strip. Supplementary information provided by the engineer states that a desk study has revealed no evidence on this tunnel and that a radar survey will be carried out prior to construction. A later consultation response on Camden's website includes a photograph which purports to show the tunnel. It is recommended that the results of a site reconnaissance are provided by the engineer.

4.6. The boundary wall supports the intended access route for construction traffic. Supplementary information includes calculation that show it is adequate to accommodate the construction traffic loadings.

4.7. The BIA has confirmed that general ground conditions at the site are a variable thickness of Made Ground (gravelly clay, sand and clayey gravel) of up to 3.8m, over the Claygate Member (soft becoming firm sandy clay) to 4.5m to 5.0m and then firm becoming stiff London Clay to the base of the borehole at 15m bgl. The BIA & HBIA have identified that in the middle of the proposed basement there is approximately 1m of Made Ground overlaying approximately 3.5m of fine, sandy clay, thus the basement will be founded in or just above the London Clay. The retaining walls will support a combination of Made Ground and materials from the Claygate Formation.

4.8. No geotechnical laboratory tests, interpretations or proposed geotechnical parameters fro design were provided in the BIA. Supplementary information states that laboratory testing was not carried out. Best practice in ground investigation is to rely on a combination of in situ and laboratory testing.

4.9. Monitoring of a single borehole has shown a groundwater level approximately 0.5m above the proposed top of floor slab level. However, this was in the summer and the hydrogeology BIA states that water levels could rise considerably in the winter months. Additional groundwater monitoring is recommended.

4.10. There are a number of existing trees adjacent to the boundary of, or on the site of, the proposed basement works. There is a Western Red Cedar immediately adjacent to the southern boundary and a large London Plane Tree, with its trunk just outside the boundary of the property. An aboricultural report concluded that damage would not be caused to the tree. In the BIA it is proposed that an existing Silver Birch on the site is to be felled.

4.11. The underlying clay formation is known to be of high plasticity so the removal of the Silver Birch could also result in some heave. The potential impact of ground movements for shrinking and/or swelling of clays in the context of the tree removal has been considered. It is accepted that existing and proposed foundations are below the depth of any likely desiccation.

4.12. Additional groundwater monitoring is recommended. This will further clarify any need for design against flotation. It is noted that proposed measures were described to deal with such a scenario i.e. basal drainage layer. The basement is to be tanked and a drained cavity system will be provided.

4.13. The proposed construction method for the basement is to:

- construct a bored pile, secant type, wall around the edge of the new basement;
- cast a concrete capping beam onto the piles;
- partially excavate within the piled perimeter to 1.0m;
- install temporary props;
- excavate to full depth;
- cast basement slab;
- remove lower props

- cast walls;
- cast ground floor slab; and
- remove upper props.

4.14. Indicative calculations and a basic sequence of construction have been provided. It is noted that the soil stiffness adopted in the retaining wall design are higher than those normally assumed in these circumstances and differ from those adopted in the accompanying ground movement assessment. This should be resolved in detailed design and agreed with the party wall surveyor.

4.15. The piles appear to be positioned directly under the existing boundary fences which will need to be removed to enable construction to proceed. The piling rig may also clash with the canopy of the London Plane Tree and Western Red Cedar and some lower branches may need to be removed. These matters should be addressed in the Construction Management Plan.

4.16. A detailed ground movement and building damage assessment based on CIRIA 580 was provided with the supplementary information and provides justification for the assumptions made. Heave is also considered. On this basis, it is accepted that damage to adjacent structures is predicted to be Category 0 to Category 1.

4.17. Supplementary information including proposals for monitoring of adjacent buildings are included in the BIA. This should be further developed with the party wall surveyor together with condition surveys.

4.18. The local topography is <7 degrees and slope stability is suggested not to be an issue.

4.19. Hydrogeology & Hydrology screening, scoping and mitigation measures have been included in the HBIA. The historic Shepherds Hill conduit (water course) used to run within 20-40m to the west of the site. It is acknowledged within the HBIA that the basement construction may increase below ground water levels and in view of this and the historic conduit, it proposes a drainage corridor, French drain and sump as mitigation measures.

4.20. A flood risk assessment was completed. The only significant flood risk identified was from blockage of private drainage connections.

4.21. Development increases the impermeable surface area. An assessment was undertaken in accordance with CIRIA Suds Manual C697 and concluded that there is no material impact from the increased surface area. However, it did state that attenuation could be provided if needed to ensure the existing condition is maintained and detailed drainage design could also include grassed filter strips. Further analyses and design were presented as supplementary information and would appear to confirm that the proposed mitigation measured are adequate.

4.22. The BIA has stated that the development will not impact on the wider hydrogeology of the area, any other watercourses, springs or the Hampstead Heath Pond chain catchment area.

5.0 CONCLUSIONS

5.1. The BIA and Hydrology BIA were completed by competent consultants suitably qualified in accordance with CPG4.

5.2. The proposed works consist of the demolition of the existing above ground two storey building and the construction of a three storey building above ground with basement below.

5.3. The BIA has confirmed that general ground conditions at the site comprise Made Ground to up to 3.8m, over the Claygate Member and then London Clay to the base of the borehole at 15m bgl. Monitoring of a single borehole has shown a groundwater level approximately 0.5m above the proposed top of floor slab level. Additional groundwater monitoring is recommended.

5.4. No geotechnical laboratory tests, interpretation or proposed geotechnical parameters for design were provided in the BIA. It has been confirmed no laboratory testing was undertaken. This does not conform with best practice.

5.5. Nearby foundations have been assumed to be shallow strips and the presence of a semi-basement to No. 64 Fitzjohn's Avenue has been confirmed. Other building are remote from the site.

5.6. The site and surrounding area are essentially flat (slope angles <7°). The proposed development will not alter this scenario. It is accepted that the surrounding slopes to the development site are stable.

5.7. The proposed construction method for the basement is to be a propped bored pile, secant retaining wall. Props will be removed after construction of the basement level and first floor level slabs. Indicative calculations for the retaining walls and floor slab have been provided, together with an indicative construction sequence demonstrating the principles of design. The need for dewatering has been considered. The soil stiffness values adopted for retaining wall design are considered too high, however, the final design may be agreed as part of the party wall award.

5.8. It is noted that, depending on ongoing groundwater monitoring, allowance has been made for anti-flotation mitigation comprising a basal drainage layer.

5.9. The Historic Shepherds Hill conduit (water course) used to run within 20-40m to the west of the site. Based on this, the groundwater level identified in the borehole and the increased impermeable area, mitigation measures are proposed in the BIA and HBIA. These include provision of a drainage corridor, French drain, sump and pump.

5.10. There may be some form of tunnel beneath the narrow access strip to 64 Fitzjohn's Avenue (reported by a local resident). The supplementary information provided by the applicant's team relies on desk study evidence and proposes a radar survey. It is recommended that a site reconnaissance is undertaken and the results reported.

5.11. The boundary wall to No. 64 Fitzjohn's Avenue supports the proposed access road (for construction traffic). Calculations have been provided to demonstrate that the wall can support the proposed loadings.

5.12. A detailed ground movement and building damage assessment based on the empirical method in CIRIA 580 assuming a piled retaining wall embedded in stiff clays and high support stiffness has been provided with justification for the assumptions made. Damage to neighbouring structures in predicted to be no worse than Burland Category 1. The predicted ground movements include a consideration of heave.

5.13. Outline proposals for the monitoring of adjacent buildings are included in the supplementary information. The final scheme and the extent of condition surveys may be agreed with the party wall surveyor.

5.14. The flood risk assessment shows the only significant flood risk as blockage of private drainage connections.

5.15. Development increases the impermeable surface area. It stated that attenuation could be provided if needed to ensure the existing condition is maintained and detailed drainage design could also include grassed filter strips. Further analyses and design indicate the proposed mitigation measures are adequate.

5.16. The BIA has stated that the development will not impact on the wider hydrogeology of the area, any other watercourses, springs or the Hampstead Heath Pond chain catchment area.

Appendix 1: Residents' Consultation Comments

Residents' Consultation Comments

Surname	Address	Date	Issue raised	Response
McGregor	Flat A, 64 Fitzjohn's Avenue	26/01/2016	Existing tunnel beneath main access road in the property is unsuitable for lorries and large vehicles. Tunnel also bears onto the walls of No. 64 Fitzjohns Avenue.	Item 4 - Audit Query Tracker
			The site access road is supported by the wall of No. 64 Fitzjohns Avenue. Vibrations caused by lorries will be considerable.	Item 5 - Audit Query Tracker
			Effects of short term de-watering during basement construction could be detrimental to stability of adjacent properties.	Item 9 - Audit Query Tracker
			Basement is below groundwater level which will be shallower in the winter than recorded in investigation undertaken. Diversion of groundwater will impact surrounding buildings.	Item 2 – Audit Query Tracker. Water diversion also addressed in current Hydrogeology BIA.
			Proposed basement is too close to suspected water courses.	Addressed in current HBIA
			Potential rise in groundwater level is unacceptable due to groundwater already being shallow.	Item 2 – Audit Query Tracker. Water diversion also addressed in current Hydrogeology BIA.
			Potential effects due to tree removal and installation of a contiguous piled wall.	Items 6 and 7 – Audit Query Tracker
Oldroyd	Flat D, 64 Fitzjohn's Avenue	26/01/2016	Slope stability and subterranean (groundwater) are development constraints.	Items 2, 3, 6, 7 & 10 – Audit Query Tracker

			Prediction of ground movements due to the works are difficult to predict accurately. This creates unknown future risks.	Item 6 – Audit Query Tracker
			Property likely to be on a 'raft' of clays that are that are subject to changes in groundwater level and best left undisturbed.	Items 1, 2 & 6 – Audit Query Tracker
Oldroyd	Flat D, 64 Fitzjohn's Avenue	02/02/2016	Objective is to keep damage to neighbouring properties within Burland category 2. However, Category 2 still requires repair works and therefore cost and inconvenience to neighbours.	Item 6 – Audit Query Tracker
			Risk of surface flow flooding after heavy rain.	Refer to paragraph 4.21
			Basement requires excavation close to neighbouring foundations. This triggers Party Wall Act of 1996 and a notice needs to be served to neighbours.	Agreed
Salprime Ltd	64 Fitzjohn's Avenue	18/07/16	Further statement of existence of tunnel beneath access road.	Item 4 – Audit Query Tracker
			Risk of surface water flow.	Item 2 – Audit Query Tracker
Casdagli	Flat B, 64 Fitzjohn's Avenue	03/08/16	Concerns on impact to foundations to No. 64 Fitzjohn's Avenue.	Item 6 – Audit Query Tracker
Green	Flat E, 64 Fitzjohn's Avenue	18/08/16	Concerns on impact to 64 Fitzjohn's Avenue.	Item 6 – Audit Query Tracker
			Presence of tunnel.	Item 4 – Audit Query Tracker

Appendix 2: Audit Query Tracker

Audit Query Tracker

Query No	Subject	Query	Status	Date closed out
1	Hydrogeology/Stability	All geotechnical data i.e laboratory testing, interpretations, derived geotechnical parameters for design etc. to be provided. Further ground monitoring to be carried out.	Closed – No laboratory testing undertaken. Design to be based on insitu testing. Final design and groundwater regime to be agreed with party wall surveyor.	August 2016
2	Hydrogeology/Hydrology	Further assessment of: · Attenuation requirements for water infiltration to ground to ensure current regime is maintained. · Need for basal drainage layer to basement.	Closed – Refer to Appendix 3.	August 2016
3	Stability	Are there any basements in adjacent properties and/or what are foundation types, depths etc?	Closed – Semi-basement to No. 64 Fitzjohn's Avenue confirmed. Other structures are remote.	August 2016
4	Stability	Is there a tunnel beneath the access strip adjacent to No.64 Fitzjohn's Avenue and will it be affected by the works or trafficking?	Open – To also consider storage of construction materials.	
5	Stability	Is site access road supported by the wall of No.64 Fitzjohn's Avenue? Is it structurally able to support proposed construction traffic loads?	Closed – Calculations demonstrate adequacy of wall and foundation.	August 2016
6	Land Stability	Further review of potential ground movement/building damage assessment needed, in particular heave due to the 4.5m excavation and installation of piles in form clay.	Closed – Detailed ground movement assessment provided. Final design of retaining wall to be agreed with party wall surveyor.	August 2016

7	Land Stability	Confirmation of impact of removal of Silver Birch tree required.	Closed – Confirmed no impact on foundations.	August 2016
8	Stability	A monitoring regime for adjacent buildings/infrastructure is required, including development of trigger and action levels.	Closed – Final details to be agreed with party wall surveyor.	August 2016
9	Stability	Indicative structural calculations and construction sequence required showing principles of design and propping, and consideration of dewatering.	Closed – Information provided shows secant wall which will avoid loss of soils due to dewatering. Final design to be agreed with party wall surveyor.	August 2016

Appendix 3: Supplementary Supporting Documents

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Our Ref: 15094 July 2016

66 FITZJOHN'S AVENUE, LONDON NW3

RESPONSE TO QUERIES RAISED IN CAMPBELL REITH'S BASEMENT IMPACT ASSESSMENT AUDIT

INTRODUCTION:

Michael Chester & Partners prepared a structural Basement Impact Assessment (BIA) to accompany a planning application for the above site by Webb Architects. The application included the demolition of an existing semi-detached property followed by the construction of a new semi-detached building with basement.

Campbell Reith act on behalf of London Borough of Camden and they have prepared an Audit Report of the BIA. The following addresses the queries raised by Campbell Reith in the Audit Tracker contained within Appendix 2 of their report. The queries are reproduced for ease of reference.

QUERIES RAISED IN AUDIT TRACKER REPORT:

1. **All geotechnical data i.e laboratory testing, interpretations, derived geotechnical parameters for design etc. to be provided. Further ground monitoring to be carried out.**

   No laboratory testing was carried out, only the insitu testing noted on the borehole logs included within the structural BIA. This is because the engineering properties of the Claygate Beds and London Clay are well known to piling contractors who regularly work within London. Also, our experience is that, on small project like this, piling contractors prefer insitu tests to determine pile design parameters because they find they more accurately reflect the ground conditions than do laboratory tests (samples are often poorly taken) plus the fact that there are inadequate economies of scale to make the savings on pile construction that laboratory tests might allow on much larger projects.

   Additional ground water monitoring has been carried out and the results are considered further in the response to the Audit Tracker by the Hydrological Engineer, SLR Consulting, contained under separate cover.

2. **Are there any basements in adjacent properties and/or what are foundation types, depths etc?**

   There is a half depth basement at No.64 Fitzjohn's Avenue. Foundations details are not known but the building is a traditionally built Victorian structure so they have conservatively been assumed to be shallow corbelled brickwork. The next closest property is 12m distant from the site. It is not known whether this building has a basement but it is sufficiently far away that it is, in any case, not relevant to this development in purely structural terms.

3. **Is there a tunnel beneath the access strip adjacent to No.64 Fitzjohn's Avenue and will it be affected by the works or trafficking?**

   Desk studies have revealed no evidence of a tunnel or culvert running across the strip of land adjacent to No.64 though some sources do indicate an old upper tributary of the Tyburn to the east of No.64.

   Before work commences on site the contractor will be required to carry out a ground radar survey to investigate this further. They will also be required to provide a temporary road base that will span over any anticipated soft spots. At this stage this is assumed to take the form of a thick reinforced concrete slab built off a DoT subbase.

4. Is site access road supported by the wall of No.64 Fitzjohn's Avenue? Is it structurally able to support proposed construction traffic loads?

As above, there is a half depth basement to the full footprint of No.64 Fitzjohn's Avenue so, yes, the flank wall will be required to support some traffic loads from the access road. The access road is narrow, however, being only 2.6m wide at its pinch point, so vehicular access will be limited. Material deliveries during construction will, therefore, in any case, have to be made in small loads.

The road is currently used by cars to access the properties at the rear and there is no evidence that this is having or has had a detrimental effect on the wall. The wall in question is 450mm thick at its base and it is preloaded at the very least by 13m of brickwork. MCP have carried out some preliminary calculations to assess the strength of the wall and these are contained within Appendix A. They concur with the visual evidence and show that the wall and its foundations are capable of withstanding a surcharge of 2.5kN/m^2 whilst maintaining reactions within the middle third of the foundation (factor of safety against overturning is, therefore, in excess of 3) and without excessive brick bearing stresses.

As above, the contractor will in any case be required to provide a road base that will span over possible soft spots. This will have the benefit of distributing wheel and axial loads more evenly along the length of the wall and across the width of the access road and will help to mitigate any adverse effects of the traffic.

5. Further review of potential ground movement/building damage assessment needed, in particular heave due to the 4.5m excavation and installation of piles in form clay.

Pile calculations have been received from Southern Geotechnical Design Ltd and a geotechnical report on the heave aspects has been received from Donaldson Associates. Both are contained within Appendices B & C below and both concur with the original BIA, confirming that if ground movements occur beyond the site boundary anticipated damage would fall within categories 0 or 1, negligible to very slight.

Southern Geotechnical Design's calculations consider temporary propping during the works at just below existing ground level to allow the capping beam to be formed along the heads of the piles and permanent props at new basement slab and ground floor slab levels. The sequence of construction assumes that the temporary prop will be in place before bulk excavation commences and that the basement slab will be formed as soon as excavation reaches the appropriate depth. The calculations predict that the maximum settlement depth will be 4mm at 3m from the face of the new piled wall, tailing off to zero at 14m distance from the piled wall. No.64 Fitzjohn's Avenue is approximately 3m from the piled wall; the possible movement gives a strain of 0.036% corresponding to a damage assessment of category 0. No.14 Arkenside Road is 10m from the piled wall; predicted settlements at this distance are in the order of 1.5mm with a similar overall strain anticipated.

Donaldson Associates have considered the above along with the heave movements due to the release of overburden following the excavation. They have predicted vertical movements of between 4mm and 7mm at the face of No.64 Fitzjohn's Avenue and horizontal movements of between 6mm and 9mm resulting in a strain of 0.05%. This corresponds to a damage assessment on the border between category 0 and category 1. They have also predicted vertical movements of between 0mm and 4mm and horizontal movements of between 1mm and 5mm for No.14 Arkenside Road resulting again in a strain of 0.005%. Because of its distance from the excavation Donaldson Associates have concluded that there is a very low risk of damage to No.14 Arkenside Road and propose no further assessment but they recommend monitoring of No.62/64 Fitzjohn's Avenue.

6. Confirmation of impact of removal of Silver Birch tree required.

Silver Birches are classed by the National House Building Council's (NHBC) guidelines for building near trees as low water demand trees. The height of the Silver Birch in question is between about 10m and 12m and it is 3.4m from the face of No.64 Fitzjohn's Avenue. Based on this, the NHBC Standards Part 4.2 Chart 1 indicates that foundations deeper than 1.35m will be beyond the zone of influence of the roots. The difference in ground levels between where the Silver Birch is growing and the basement is 1.6m. The foundations are, therefore, clearly deeper than required by the NHBC guidelines so the removal of this tree will not affect No.64 Fitzjohn's Avenue. There are no other buildings within the zone of influence of the tree.

7. A monitoring regime for adjacent buildings/infrastructure is required, including development of trigger and action levels.

Donaldson Associates have recommended monitoring of No.62/64 Fitzjohn's Avenue during the course of the works. Given the very small movements anticipated consideration is to be given to the use of an "intelligent" data logging system which will provide greater accuracy than traditional tell-tales or demountable gauges and will provide more detailed information around particular movement "events" if they occur. A green, amber, red traffic light system of trigger and action levels will be developed in conjunction with the Party Wall Surveyors.

8. Indicative structural calculations and construction sequence required showing principles of design and propping, and consideration of dewatering.

Drawing number 15094/SK02revA by MCP (Appendix D) and pile calculations by Southern Geotechnical Design Ltd (Appendix B) describe the sequence of construction and principles of the design and propping. In summary this is as follows –

a) Erect a hoarding around the site and demolish the existing building.
b) Install a secant piled wall around the perimeter of the proposed basement, sealed in to the London Clay.
c) Pump ground water out from within the footprint of the proposed basement.
d) Construct a capping beam to tie the heads of the piles and install horizontal props to restrain the head of the piled wall.
e) Excavate within the piles to new basement level. Cast new basement slab and the new permanent retaining walls all round the excavation.
f) Cast the ground floor level slab.
g) Remove temporary props when ground floor level slab is fully cured.
h) Complete construction of superstructure.

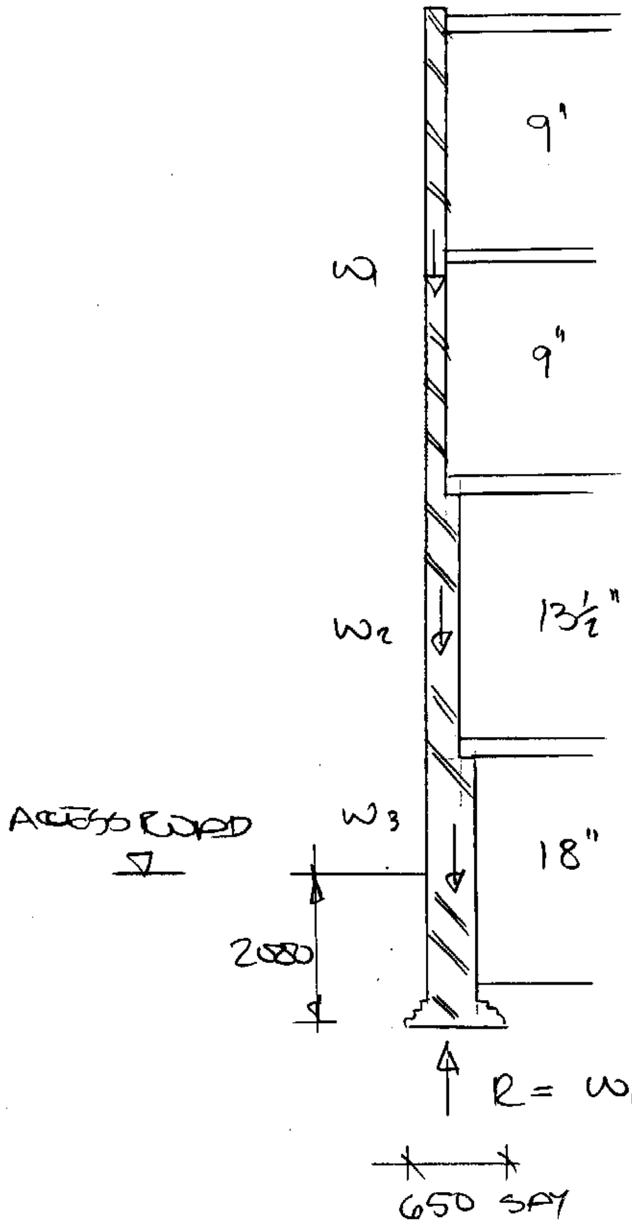
In terms of dewatering, as set out in the original BIA, it is proposed to install a secant piled wall sealed off in to the London Clay. This will prevent water entering the excavation from the side through the piled wall and from below, thus allowing the water within the basement footprint to be pumped out completely prior to excavation. As no water is able to enter the excavation during the work, no fines are lost from the soils beyond the piled perimeter of the site thus eliminating the associated effects of soil consolidation on the surrounding ground and buildings.

APPENDIX A

66 FITZJOHN'S AVENUE, LONDON NW3

PRELIMINARY CALCULATIONS FOR FLANK WALL OF No.64 FITZJOHN'S AVENUE

FLANK WALL OF No 64 FITZJOHN'S ROAD



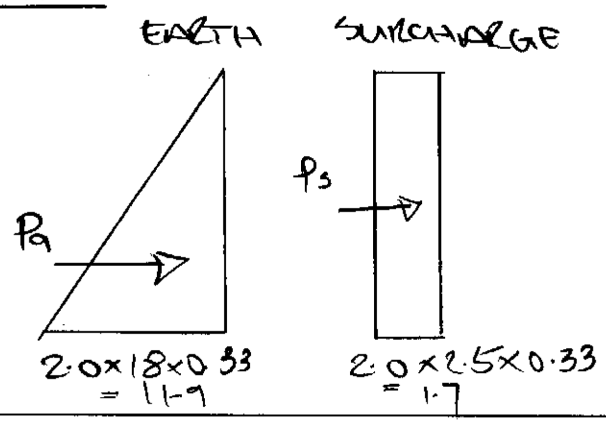
$W_1 = 6.5 \times 5.0 = 32.5\text{kN}$

$W_2 = 3.5 \times 7.6 = 26.6\text{kN}$

$W_3 = 3.5 \times 10.2 = 35.7\text{kN}.$

$R = W_1 + W_2 + W_3 = 95\text{kN}.$

OVERTURNING.



$P_a = \frac{1}{2} \times 2.0 \times 11.9 = 11.9\text{ kN}$

$P_s = 2.0 \times 1.7 = 3.4\text{ kN}$

MICHAEL CHESTER & PARTNERS Consulting Civil & Structural Engineers	Date JUN 16 Eng TOM	Job No 15094	Sheet No RW02	Rev
8 Hale Lane London NW7 3NX tel 020 8959 9119 fax 020 8959 9662	Project 66 FITZJOHN'S AVENUE LONDON NW3			

OVERTURNING MOMENT -

$$M_{ov} = (11.9 \times 2.0/3) + (3.4 \times 2.0/2) = 11.3 \text{ kNm}$$

ECCENTRICITY OF LOADING -

$$95\bar{y} = (32.5 \times 443) + (26.6 \times 385) + (35.7 \times 325) - 11.3 \times 10^3$$

$$\bar{y} = 263$$

$$e = 325 - 263 = 62 \quad \therefore \text{ WITHIN MIDDLE THIRD}$$

THEREFORE FACTOR OF SAFETY AGAINST OVERTURNING GREATER THAN 3

INCREASES IN BEARING PRESSURES ARE VERY SHORT TERM AND WILL HAVE NO APPRECIABLE EFFECT ON THE FOUNDATION.

BRICK STRESSES AT BASE OF WALL -

$$\sigma_b = \frac{95 \times 10^3}{1000 \times 450} \pm \frac{6 \times 95 \times 10^3 \times 62}{1000 \times 450^2}$$

$$0.21 \pm 0.18$$

$$\left.\begin{aligned} \text{MAX} &= 0.39 \text{ N/mm}^2 \\ \text{MIN} &= 0.03 \text{ N/mm}^2 \end{aligned}\right\} \underline{\text{OK}}.$$

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APPENDIX B

66 FITZJOHN'S AVENUE, LONDON NW3

PILING CALCULATIONS BY SOUTHERN GEOTECHNICAL DESIGNS LTD

STRUCTURAL ENGINEERING CALCULATIONS

PRELIMINARY DESIGN OF SECANT BORED PILE RETAINING WALL

AT

66, FITZJOHNS AVENUE

LONDON

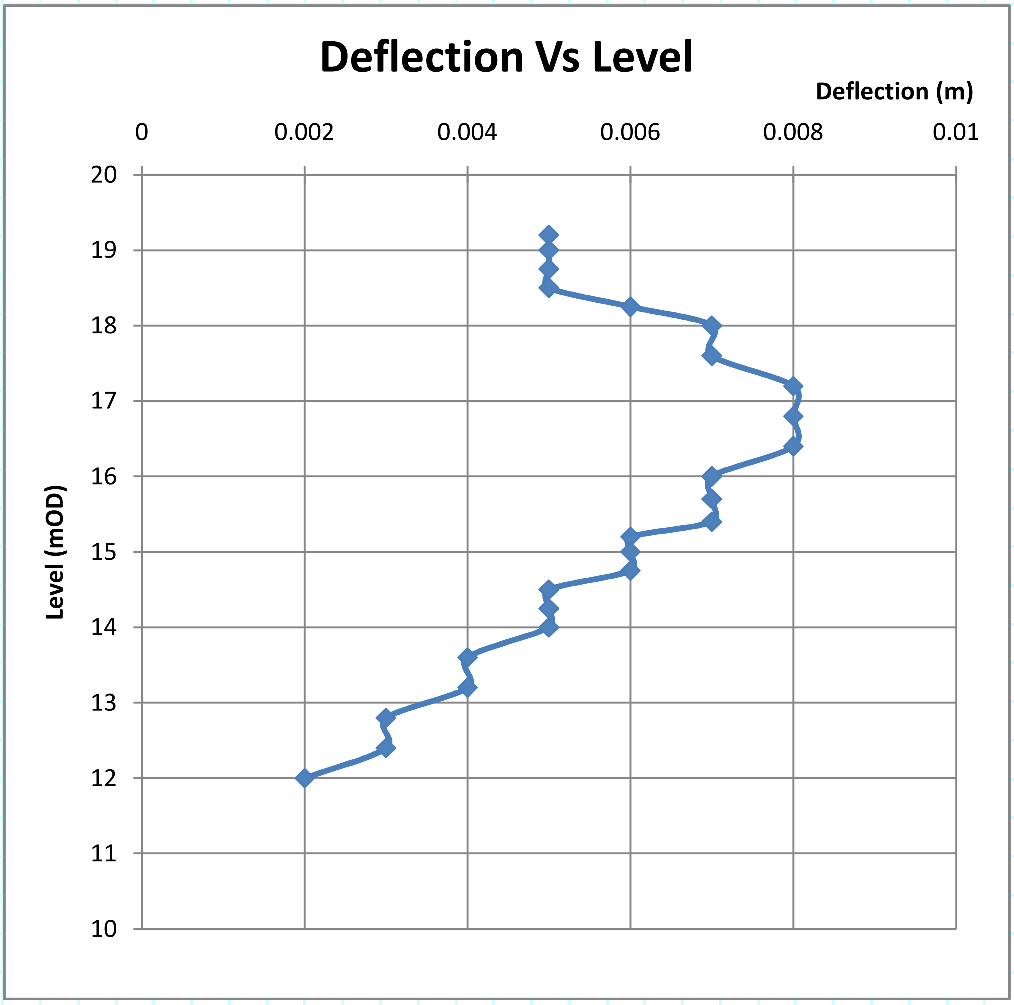
NW3 5LT

ISSUE	DATE	STATUS	REVISION DESCRIPTION	PAGES
00	21 May 2016		Initial Design	

Executive Summary

Based on the wall being 350 mm diameter piles at 550 mm centres

The anticipated deflected shape of the wall is:



Based on figure 2.16 in CIRIA C580

For this wall	$\delta 0$ =	5 mm	Thus anticipated settlment at wall =	2.5	mm
	δmax =	8 mm	Thus maximum ground settlement =	4.0	mm
		at 3m depth	at 3m from wall		

Maximum distance from wall of any vertical movement is likely to be 14.0m

Contents

1 Introduction

CP Plus Ltd has commissioned Southern Geotechnical Design Limited to carry out the preliminary design for the secant pile walls that are required for to retain the ground at 66, Fitzjohns Avenue, London NW3 5LT. The wall will be constructed as a propped bottom up excavation. Ground level is at 19.3mOD formation is at 15.0mOD (allowing for 485mm of slab construction).

1.1 Reference Documents

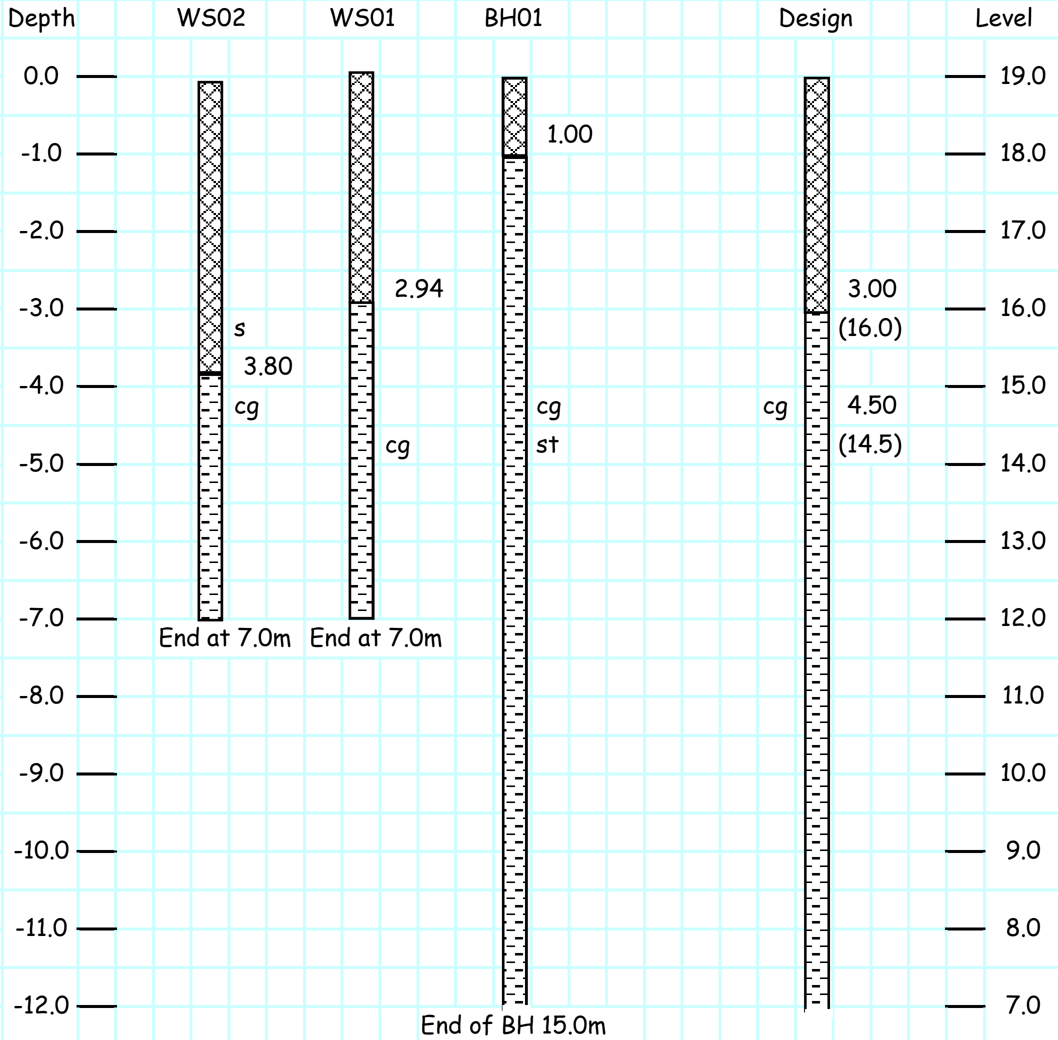
Specification	ICE SPERW
Site Investigation	SLR Hydrology Report For Basement Impact Assessment
Drawings	Numbered 15094/SK01 "Typical Section Through Basement"
Michael Chester	Numbered 15094/SK02 revision A "Assumed Sequence of Construction".
	Numbered 15094/SK03 "Section Indicating Surcharge Loading"
Web Architects	Numbered 1169.01.01(-) "Location Plan".
	Numbered 1169.01.11(-) "Proposed Front Elevation".
	Numbered 1169.01.12(-) "Proposed Basement Plan".
	Numbered 1169.01.13(-) "Proposed Ground Floor Plan".
	Numbered 1169.01.14(-) "Proposed First Floor Plan".
	Numbered 1169.01.15(-) "Proposed Second Floor Plan".
	Numbered 1169.01.14(-) "Proposed Roof Plan".
MJH Surveyors	Numbered 0160 03 "Front Elevation No 66".
	Numbered 0163 01 "Site Plan".
	Numbered 0163 01 "Roof Plan".
	Numbered 0163 04 "Rear Elevation No 66".
	Numbered 0163 05 "Side Elevation No 66".
	Numbered 0163 06 "Side Elevation No 66".
	Numbered 0163 07 "Side Elevation".
	Numbered 0163 08 "Rear Elevation No 12".
Codes, Standards & References:	
	BS EN 1997-1: 2004 Eurocode 7: Geotechnical Design - Part 1: General Rules
	UK National Annex to Eurocode 7: Geotechnical Design - Part 1: General Rules.
	CIRIA C580 London 2003 Embedded Retaining Walls - Guidance For Economic Design
	BS EN 1992-1-1: 2004 Eurocode 2: Design of Concrete Structures - Part 1-1: General Rules and Rules for Buildings
	UK National Annex to Eurocode 2: Design of Concrete Structures - Part 1-1: General Rules and Rules for Buildings
	"Pile design and construction practice", M J Tomlinson, 4th ed, 1994.

Charts for the design of circular columns to Eurocodes, IstructE Manual for the design of concrete building structures to Eurocode 2.

2 Ground Conditions

2.1 Strata Profile

The ground investigation shows the following strata:



Key: Made Ground   Stiff slighty sandy London Clay

The 'design' borehole is therefore taken as:

From	PPL	to	16.00	mOD	Made Ground
From	16.00	to	14.50	mOD	Claygate Beds - sandy Clay
From	14.50	to	Toe	mOD	Stiff slighty sandy London Clay

Ground water was struck in BH1 at 5.0m depth, however the level given on drawing 15094 SK01 of 16.4mOD is taken as more realistic

2.2 Soil Parameters

Moderately conservative soil parameters are required for the wall calculations. Some of these parameters will be factored for various of the analyses as detailed below:

	On Su	c'	tan φ'	E
For SLS analysis	1.0	1.0	1.0	1.0
For Com 2 analysis	1.40	1.25	1.25	1.0

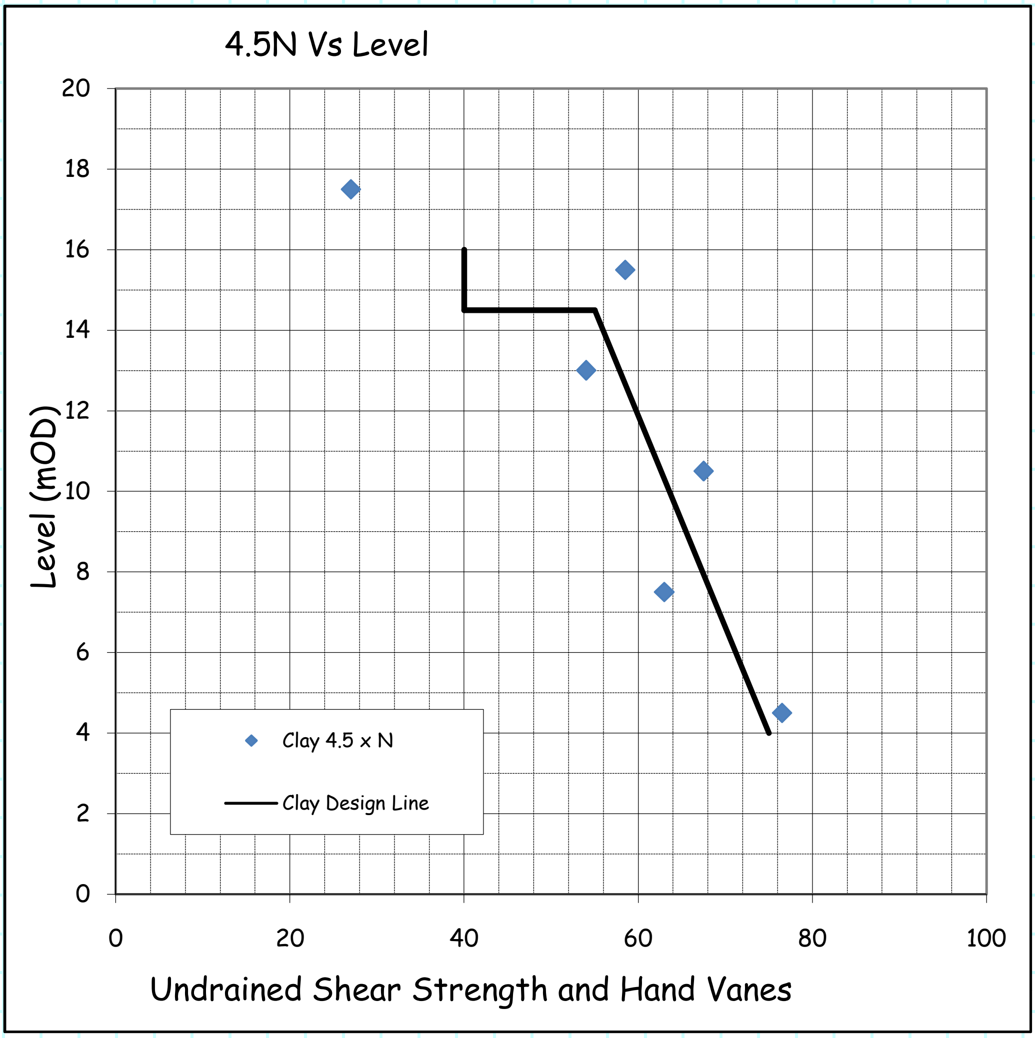
WALL FRICTION

For the bored pile walls there will be friction between the soil and the piles, this acts to reduce the active limit of soil pressure and increase the passive.

However when axial compression load is applied to the wall, it settles slightly, in this case since the piles are moving in the same direction as the active wedge, the active wall friction is taken as zero. The passive wall friction however remains the same.

The active wall friction will not be set to zero since there is no vertical load applied to the piled walls.

The undrained shear strength (triaxials and hand vane) plot versus depth is presented below.



MADE GROUND / OVERBURDEN

Bulk density,	γb =	18	kN/m^3
Soil type	Cohesionless		
Angle of friction,	φ' =	30	°
Effective cohesion	c' =	0	kN/m^2
Earth pressure coefficients,	at rest, ko =	0.50	
Elastic Modulus	E =	10000	$kN/m^2/m$

CLAYGATE BEDS

From 16mOD to 14.5mOD - Firm sandy clay - Claygate.

Bulk density, γb = 20 kN/m^3

Soil type Cohesive Undrained

Undrained Shear Strength Cu = 40 kN/m^2

allow softening 20 % Cud = 32 kN/m^2

Elastic Modulus, Eu / Cu = 800 based on cantilever and large strain

Eu = 32 MN/m^2

Drained parameters

Drained Shear Strength c' = 0 kN/m^2

Angle of friction, φ' = 22 °

Earth pressure coefficients, ko = 0.625

Elastic Modulus, E' = 0.7 Eu

E' = 22.4 MN/m^2

LONDON CLAY

From 14.5mOD to Toe - Stiff slightly sandy Clay - London Clay.

Bulk density, γb = 20 kN/m^3

Soil type Cohesive Undrained

Undrained Shear Strength Cu = 55 + 1.9 z kN/m^2

allow softening 20 % Cud = 44 + 1.52 z kN/m^2

Elastic Modulus, Eu / Cu = 800 based on cantilever and large strain

Eu = 44 + 1.52 z MN/m^2

Drained parameters

Drained Shear Strength c' = 0 kN/m^2

Angle of friction, φ' = 24 °

Earth pressure coefficients, ko = 0.593

Elastic Modulus, E' = 0.7 Eu

E' = 30.8 + 1.07 z MN/m^2

2.3 Groundwater

Ground water was struck in BH1 at 5.0m depth, however the level given on drawing 15094 SK01 of 16.4mOD is taken as more realistic

3 Basis of Design

3.1 General

The design is based on the following:

- The soil and properties used are correct for the whole site.
- The strata used are correct for the whole site.
- The retained geometry and construction sequences are as given in Appendix A
- Surcharge on the retained soil is as detailed in section 6.3.

3.2 Bearing Capacity

There are no vertical loads on the wall, other than those exerted by the ties.

3.3 Lateral Loads

The forces induced in the piled wall and the props will be calculated using the Wallap computer programme.

3.4 Structural Parameters

3.4.1 Wall Piles

Type	CFA		
Grout fcu	35	N/mm^2	28 day characteristic cube strength
fck	28	N/mm^2	28 day characteristic cylinder strength
Reinforcement fy	500	N/mm^2	

Young's Modulus of pile: instantaneous Ecm = $22 \times \{(fck + 8) / 10\} / 1.5\}^{0.3}$

= 32.3 GN/m^2

short term Ecs = 0.7 Eci

= 22.6 GN/m^2

long term Ecs = 0.5 Eci

= 16.2 GN/m^2

Diameter	350	mm
Spacing	550	mm
Second Moment of area		$= \pi d^4 / 64s$
	1.34E-03	m^4 / m

Thus wall stiffness is:

Short term	30290	kN/m^2
Long term	21640	kN/m^2

3.4.2 Struts

Strut details are given below:

Type	Centre Level (mOD)	Spacing (m)	Cross Section (m^2)	Young's Modulus (kN/m^2)	Free Length (m)	Angle (°)	Pre-Stress (kN)	Tension Allowed?
Temp	18.5	5	0.02	2.0E+08	5	0	0.0	No
Perm GF	19.2	1	0.4	2.0E+07	5	0	0.0	No
Perm B1	15.2	1	0.4	2.0E+07	5	0	0.0	No

4 Factors of Safety

4.1 Axial Load - Compression

The factor of safety for the vertical load will be 3.0

4.2 Axial Load - Tension

The factor if safety for the tensile tie load will be 3.0

4.3 Lateral and Moment Loads

The forces within the piles in the wall have been calculated using EC7. An SLS and a single ULS (Design approach 1 Combination 2) analyses have been carried out with the soil parameters factored as detailed in section 2.2.
The forces generated by these analyses will be further factored for the structural analysis of the pile. These factors are as follows:

For ULS use factor of	1.00
For SLS use factor of	1.35

The pile structural analysis will be carried out with the maxima from these two results.

4.4 Strut Loads

The strut loads are taken from the ULS and SLS wallap analyses and are then factored for the strut design using the same factors detailed for structural analysis above.

5 Basis of Design

5.1 Negative Skin Friction

Given the site details it is extremely unlikely that the soil will induce negative skin friction on the piles, therefore no allowance is made for any.

5.2 Heave Forces

There is no likelihood of heave being induced in the wall piles.

5.3 Pile Spacing

The piles will be designed on the basis of the nominal 550mm spacing.

5.4 Pile Tolerances

The piles will be installed to the standard piling tolerances, that is 1:75 verticality and +or- 75mm position. (Note that the positional tolerance increases if cut off level is below platform level at 1:75).

6 Analysis

6.1 Wall Description

The retaining wall is required to allow for the short term excavation and construction of the new structure. In the long term there will be a lining wall which will eventually be required to carry the hydrostatic pressures, not-with-standing this the piled wall is also designed for these loads.

The Engineer's sketch indicates that the piled wall will be installed, and, following a minimal excavation (say 1.0m) will be trimmed and a capping beam cast. A temporary strut will be installed and excavation continued to formation level. The base slab and roof slabs will then be cast and the temporary strut removed.

6.2 Construction Sequence for wall

The construction sequence is detailed below.

1. Prepare platform at 19.0mOD (estimated).
2. Apply existing surcharges
3. Re zero walls to represent as is situation.
4. Apply general surcharges.
5. Excavate 18.0mOD to allow cap to be built.
6. Install temporary prop at 18.5mOD
7. Excavate to 15.0mOD (Allow 0.35m unplanned excavation in ULS Com 2)
8. Fill to 15.48mOD on excavated side.
9. Install B1 slab to prop wall at 15.2mOD
10. Remove temporary prop at 18.5mOD
11. Allow soil and wall to relax to long term parameters
12. Allow long term flood conditions

6.3 Surcharge Loads

The surcharges used are detailed below:

1	Building dead	allow	115	kN/m	applied at	18.0	mOD	
		over	1.0	m width	at	4.0	m from wall	
2	Building live	allow	17	kN/m²	applied at	18.0	mOD	
		over	1.0	m width	at	4.0	m from wall	
3	General	allow	10	kN/m²	applied at	19.0	mOD	
		at	0.0	m from the wall over		4.0	m width	

6.4 Lateral and Moment Loads

There are no lateral or moment loads.

6.5 Wallap output

The Wallap input and output is presented in Appendix B

Wall		Moment (kNm/m)	Shear (kN/m)	Toe (mOD)	Defl (mm)	Struts (kN/m) Temp 18.5	Perm 19.2	Perm 15.2
All	Com 1	50	70			40	35	85
	Com 2	65	74	12.0		55	45	100
	SLS	50	50		8	40	35	40
	Des	68	95	12.0	8	55	47	115

Note Strut loads are kN per m run of wall at 90 degrees to wall

6.6 Reinforcement

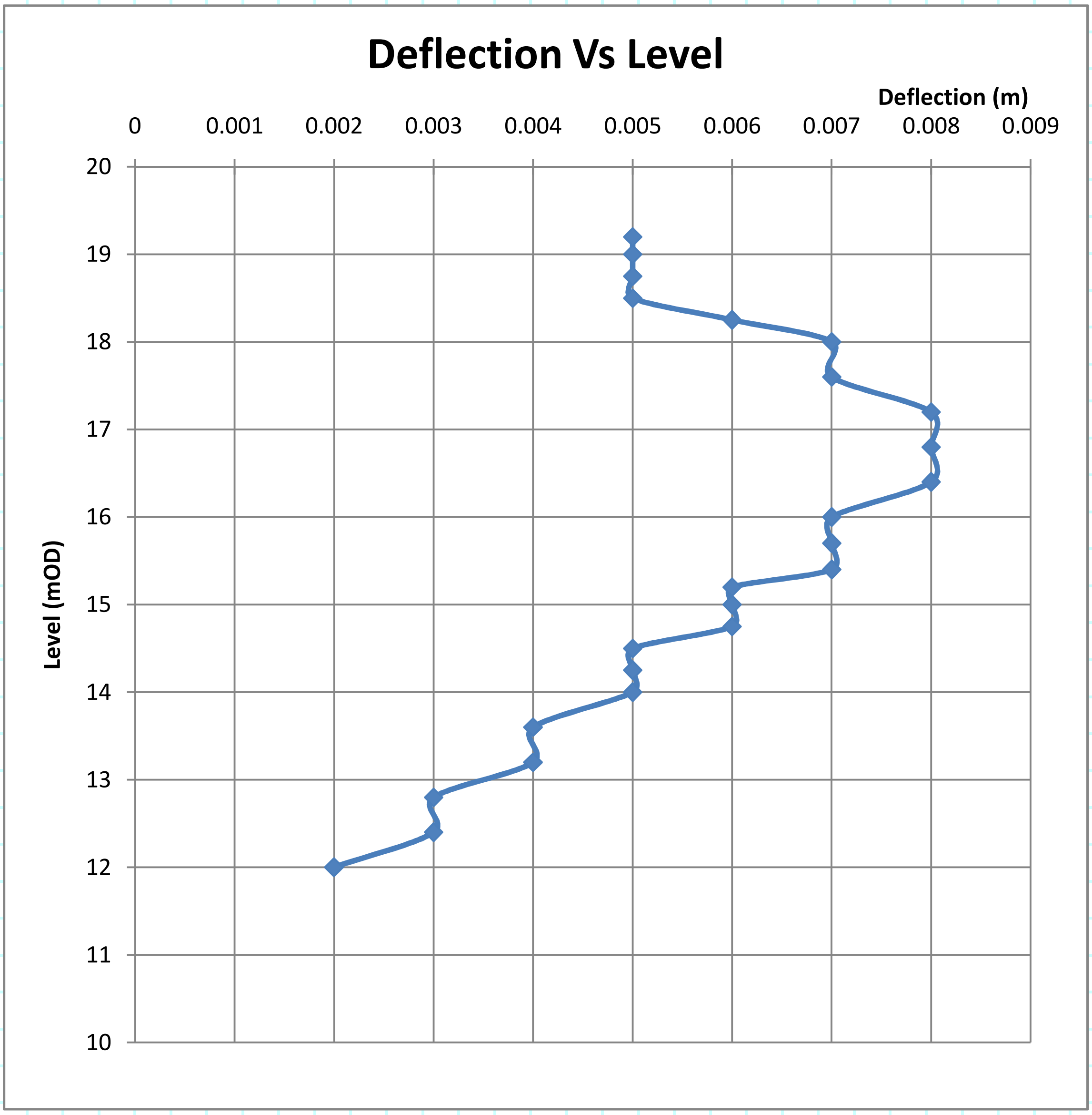
Reinforcement is designed using the I Struct E circular column design charts, conservatively based on the uncased pile section.
Standard sheets are used, these are presented in Appendix C.

6.7 Wall Vertical Capacity

No loads 54.70 mOD

6.8 Defelction

The anticipated maximum deflection is given in section 6.5 above.
The anticipated deflected shape of the wall is presented in graphical format overleaf.



Based on figure 2.16 in CIRIA C580

For this wall	δ0 = 5 mm		Thus anticipated settlment at wall =	2.5 mm
	δmax = 8 mm		Thus maximum ground settlement =	4.0 mm
	at 3m depth		at 3m from wall	

Maximum distance from wall of any vertical movement is likely to be 14.0m

APPENDIX A - Construction Sequences

Refer to Michael Chester Drawing 15094/SK02 revision A

Stages

1	Prepare platform at 19.0mOD (estimated).
2	Apply existing surcharges
3	Re zero walls to represent as is situation.
4	Apply general surcharges. 0
5	Excavate 18.0mOD to allow cap to be built.
6	Install temporary prop at 18.5mOD
7	Excavate to 15.0mOD (Allow 0.35m unplanned excavation in ULS Com 2)
8	Fill to 15.48mOD on excavated side.
9	Install B1 slab to prop wall at 15.2mOD
10	Remove temporary prop at 18.5mOD
11	Allow soil and wall to relax to long term parameters
12	Allow long term flood conditions

APPENDIX B – WALLAP INPUT / OUTPUT – COM 1

```
SOUTHERN GEOTECHNICAL DESIGN                                   | Sheet No.
Program: WALLAP  Version 6.05  Revision A45.B58.R49            | Job No.   C0745
                             Licensed from GEOSOLVE            | Made by :    MP
Data filename/Run ID: Com 1                                    |
66 Fitzjohns Avenue, London NW3 5LT                            | Date:23-05-2016
Com 1                                                          | Checked :
-----------------------------------------------------------------------------
                                                       Units: kN,m
```

INPUT DATA

SOIL PROFILE

```
Stratum   Elevation of    ------------------ Soil types -------------------
  no.    top of stratum   Active side               Passive side
   1          19.00       1  Made Ground            1  Made Ground
   2          16.00       2  Claygate Undr          2  Claygate Undr
   3          15.00       2  Claygate Undr          3  Claygate To soft
   4          14.50       4  London Clay Undr       4  London Clay Undr
```

SOIL PROPERTIES (Unfactored SLS soil strengths)

```
                          Bulk    Young's   At rest  Consol  Active  Passive
-- Soil type --         density  Modulus    coeff.  state.  limit    limit    Cohesion
No. Description          kN/m3  Eh,kN/m2     Ko     NC/OC    Ka       Kp       kN/m2
  (Datum elev.)                (dEh/dy ) (dKo/dy) (  Nu ) ( Kac ) (  Kpc ) ( dc/dy )
 1  Made Ground         18.00     10000    0.500     OC    0.333    4.369
                                                 (0.200) (0.000) ( 0.000)
 2  Claygate            20.00     32000    1.000     OC    1.000    1.000     32.00u
    Undr                                         (0.490) (2.000) ( 2.000)
 3  Claygate To          20.00     32000    1.000     OC    1.000    1.000     32.00u
    soft                                         (0.490) (2.000) ( 2.000)
 4  London Cl..         20.00     44000    1.000     OC    1.000    1.000     44.00u
    (   14.50 )               (   1520)              (0.490) (2.000) ( 2.390) (  1.520)
 5  Claygate ..         20.00         1    1.000     OC    1.000    1.000     1.000u
    (   15.00 )               (  64000)              (0.490) (2.000) ( 2.000) (  64.00)
 6  Claygate Dr         20.00     22400    1.000     OC    0.455    2.198       0.0d
                                                 (0.150) (1.349) ( 2.965)
 7  London Cl..         20.00     30800    1.000     OC    0.422    3.077       0.0d
    (   14.50 )               (   1070)              (0.150) (1.299) ( 4.665)
```

Additional soil parameters associated with Ka and Kp

```
                          --- parameters for Ka ---  --- parameters for Kp ---
                            Soil      Wall    Back-    Soil      Wall    Back-
------- Soil type ------- friction  adhesion  fill   friction  adhesion  fill
No. Description             angle    coeff.   angle    angle    coeff.   angle
 1  Made Ground             30.00    0.000    0.00     30.00    0.500    0.00
 2  Claygate Undr            0.00    0.000    0.00      0.00    0.000    0.00
 3  Claygate To soft         0.00    0.000    0.00      0.00    0.000    0.00
 4  London Clay Undr         0.00    0.000    0.00      0.00    0.500    0.00
 5  Claygate Soft            0.00    0.000    0.00      0.00    0.000    0.00
 6  Claygate Dr             22.00    0.000    0.00     22.00    0.000    0.00
 7  London Clay LT          24.00    0.000    0.00     24.00    0.500    0.00
```

GROUND WATER CONDITIONS

Density of water = 10.00 kN/m3

	Active side	Passive side
Initial water table elevation	16.40	16.40

Automatic water pressure balancing at toe of wall : No

Water press. profile no.	Active side Point no.	Elev. m	Piezo elev. m	Water press. kN/m2	Passive side Point no.	Elev. m	Piezo elev. m	Water press. kN/m2	
1	1	16.40	16.40	0.0	1	15.00	15.00	0.0	MC
					2	13.00	16.40	34.0	
2	1	16.40	16.40	0.0	1	14.65	14.65	0.0	WC
					2	12.60	16.40	38.0	
3	1	16.40	16.40	0.0	1	15.00	15.00	0.0	MC+WC
					2	14.90	16.40	15.0	
4	1	18.00	18.00	0.0	1	15.00	15.00	0.0	WC
					2	14.90	16.40	15.0	

WALL PROPERTIES

Type of structure = Fully Embedded Wall
Elevation of toe of wall = 12.00
Maximum finite element length = 0.40 m
Youngs modulus of wall E = 2.2600E+07 kN/m2
Moment of inertia of wall I = 1.3400E-03 m4/m run
E.I = 30284 kN.m2/m run
Yield Moment of wall = Not defined

STRUTS and ANCHORS

Strut/anchor no.	Elev.	Strut spacing m	X-section area of strut sq.m	Youngs modulus kN/m2	Free length m	Inclin -ation (degs)	Pre-stress /strut kN	Tension allowed
1	18.50	5.00	0.020000	2.000E+08	5.00	0.00	0	No
2	19.20	1.00	0.400000	2.000E+07	5.00	0.00	0	No
3	15.20	1.00	0.400000	2.000E+07	5.00	0.00	0	No

SURCHARGE LOADS

Surch -arge no.	Elev.	Distance from wall	Length parallel to wall	Width perpend. to wall	Surcharge kN/m2 Near edge	Far edge	Equiv. soil type	Partial factor/	Category
1	18.00	4.00(A)	100.00	1.00	115.00	=	N/A	1.00	-
2	18.00	4.00(A)	100.00	1.00	17.00	=	N/A	1.00	-
3	19.00	0.00(A)	100.00	4.00	10.00	=	N/A	1.00	-
4	15.20	-0.00(P)	100.00	100.00	20.00	=	N/A	1.00	-

Note: A = Active side, P = Passive side
Limit State Categories P/U = Permanent Unfavourable
P/F = Permanent Favourable
Var = Variable (unfavourable)

CONSTRUCTION STAGES

```
Construction   Stage description
  stage no.    -------------------------------------------------------
     1         Change EI of wall to 100.00 kN.m2/m run
               100.00 kN.m2/m run
               No adjustments to wall displacements
     2         Apply surcharge no.1 at elevation 18.00
     3         Change EI of wall to 30284 kN.m2/m run
               30284 kN.m2/m run
               Reset wall displacements to zero at this stage
     4         Apply surcharge no.2 at elevation 18.00
     5         Apply surcharge no.3 at elevation 19.00
     6         Excavate to elevation 18.00 on PASSIVE side
     7         Install strut or anchor no.1 at elevation 18.50
     8         Apply water pressure profile no.2  ( Worst Cred. )
     9         Excavate to elevation 14.65 on PASSIVE side
    10         Change properties of soil type 3 to soil type 5
               Ko pressures will not be reset
    11         Fill to elevation 15.40 on PASSIVE side with soil type 1
    12         Install strut or anchor no.3 at elevation 15.20
    13         Install strut or anchor no.2 at elevation 19.20
    14         Remove strut or anchor no.1 at elevation 18.50
    15         Apply surcharge no.4 at elevation 15.20
    16         Apply water pressure profile no.3  ( Worst Cred. )
    17         Change properties of soil type 2 to soil type 6
               Ko pressures will not be reset
    18         Change properties of soil type 5 to soil type 6
               Ko pressures will not be reset
    19         Change properties of soil type 4 to soil type 7
               Ko pressures will not be reset
    20         Change EI of wall to 21640 kN.m2/m run
               Yield moment not defined
               Allow wall to relax with new modulus value
    21         Apply water pressure profile no.4  ( Worst Cred. )
```

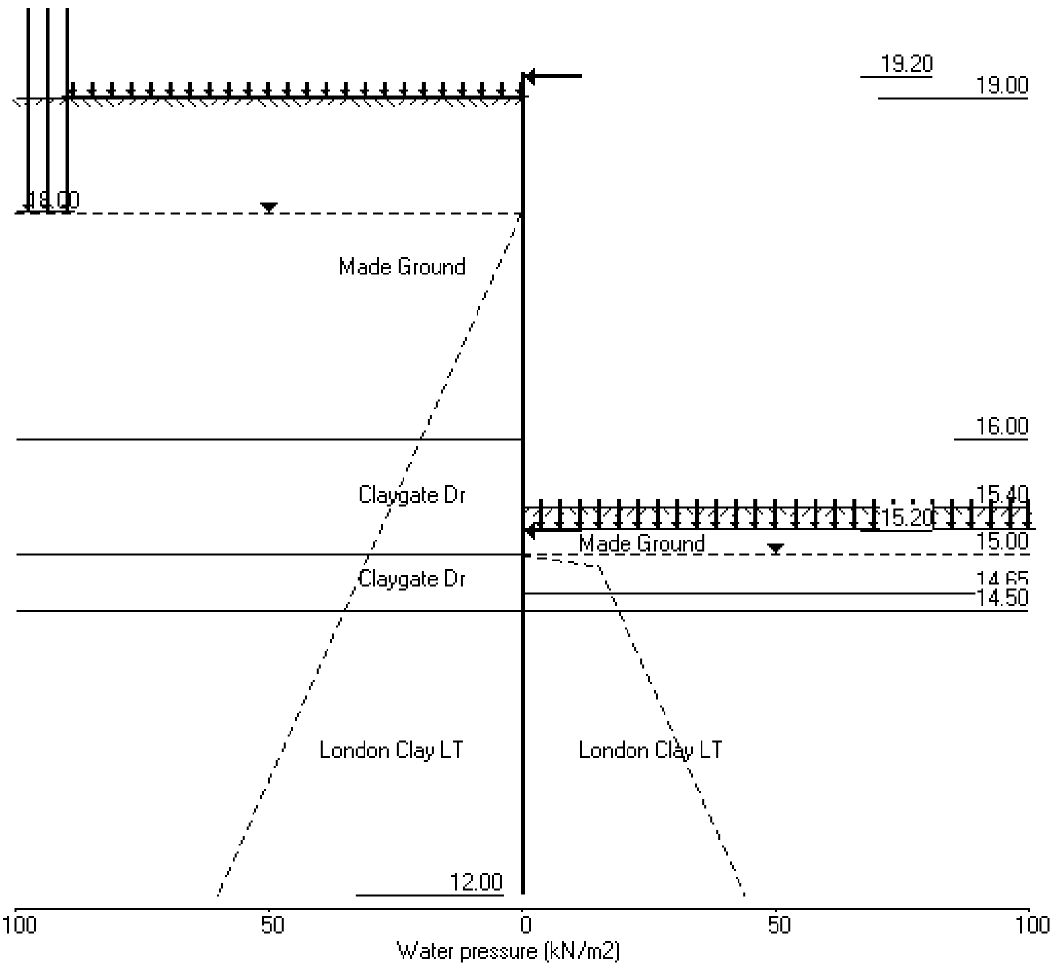
FACTORS OF SAFETY and ANALYSIS OPTIONS

```
   Limit State options: ULS DA1 Combination 1
      Water pressures : Worst Credible
      Partial factor on   C'         = 1.000
      Partial factor on Phi'         = 1.000
      Partial factor on   Cu         = 1.000
      Partial factor on Soil Modulus = 1.000
      Partial factor on Permanent Unfavourable loads = 1.000
      Partial factor on Permanent Favourable loads   = 1.000
      Partial factor on Permanent Variable loads     = 1.100
      Design factor on calculated Bending Moments    = 1.350


   Parameters for undrained strata:
      Minimum equivalent fluid density             =   5.00 kN/m3
      Maximum depth of water filled tension crack  =   0.00 m

   Bending moment and displacement calculation:
      Method  -  Subgrade reaction model using Influence Coefficients
      Open Tension Crack analysis? - No
      Non-linear Modulus Parameter (L) = 0 m

   Boundary conditions:
      Length of wall (normal to plane of analysis) = 1000.00 m
```

```
Width of excavation on active  side of wall  = 20.00 m
Width of excavation on passive side of wall  = 20.00 m

Distance to rigid boundary on active side  = 20.00 m
Distance to rigid boundary on passive side = 20.00 m
```

Units: kN,m

Stage No.21 Apply water pressure profile no.4 (Worst Cred.)



Units: kN,m

Summary of results

BENDING MOMENT and DISPLACEMENT ANALYSIS of Fully Embedded Wall

Analysis options

```
Length of wall perpendicular to section = 1000.00m
Subgrade reaction model  -  Boussinesq Influence coefficients
Soil deformations are elastic until the active or passive limit is reached
Open Tension Crack analysis - No
```

Rigid boundaries: Active side 20.00 from wall
Passive side 20.00 from wall

Limit State: ULS DA1 Combination 1

Calculated Bending Moments and Strut Forces have been multiplied by a factor of 1.35 to obtain values for structural design.

Bending moment, shear force and displacement envelopes

Node no.	Y coord	Displacement max. m	Displacement min. m	Bending moment Calculated max. kN.m/m	Bending moment Calculated min. kN.m/m	Bending moment Factored max. kN.m/m	Bending moment Factored min. kN.m/m	Shear force Calculated max. kN/m	Shear force Calculated min. kN/m	Shear force Factored max. kN/m	Shear force Factored min. kN/m
1	19.20	0.005	-0.000	0	-0	0	-0	0	-31	0	-42
2	19.00	0.005	-0.000	0	-6	0	-8	0	-31	0	-42
3	18.75	0.005	-0.000	1	-13	1	-18	4	-27	6	-36
4	18.50	0.006	-0.000	2	-20	3	-27	7	-31	9	-42
5	18.25	0.006	0.000	1	-26	2	-35	4	-30	6	-40
6	18.00	0.007	0.000	3	-31	4	-42	6	-27	9	-37
7	17.60	0.008	0.000	6	-38	8	-52	7	-23	10	-31
8	17.20	0.008	0.000	8	-44	11	-60	5	-18	7	-24
9	16.80	0.008	0.000	10	-48	13	-64	3	-12	4	-16
10	16.40	0.008	0.000	11	-48	14	-65	12	-4	16	-6
11	16.00	0.008	0.000	11	-46	15	-62	26	-0	34	-0
12	15.70	0.007	0.000	11	-41	14	-55	40	-2	53	-3
13	15.40	0.007	0.000	10	-34	13	-46	55	-4	74	-5
14	15.20	0.007	0.000	9	-30	12	-40	66	-16	89	-21
15	15.00	0.007	0.000	8	-25	11	-34	36	-6	49	-8
16	14.65	0.006	0.000	7	-13	9	-17	49	-3	66	-4
17	14.50	0.006	0.000	6	-6	8	-9	46	-2	62	-3
18	14.25	0.005	0.000	11	-0	15	-0	28	-3	38	-4
19	14.00	0.005	0.000	16	-0	22	-0	16	-4	22	-5
20	13.60	0.004	0.000	17	-0	23	-0	4	-3	6	-5
21	13.20	0.004	0.000	14	-0	18	-0	0	-12	0	-16
22	12.80	0.003	0.000	8	-0	11	-0	0	-14	0	-19
23	12.40	0.003	0.000	3	-0	3	-0	0	-10	0	-13
24	12.00	0.002	0.000	0	-0	0	-0	0	-0	0	-0

Calculated Bending Moments and Strut Forces have been multiplied by a factor of 1.35 to obtain values for structural design.

Maximum and minimum bending moment and shear force at each stage

Stage no.	Bending moment Calculated max. kN.m/m	elev.	Bending moment Calculated min. kN.m/m	elev.	Bending moment Factored max. kN.m/m	Bending moment Factored min. kN.m/m	Shear force Calculated max. kN/m	elev.	Shear force Calculated min. kN/m	elev.	Shear force Factored max. kN/m	Shear force Factored min. kN/m
1	0	13.60	-0	14.25	0	-0	0	19.20	0	19.20	0	0
2	0	14.25	-0	15.00	0	-0	0	14.50	-0	15.20	0	-0
3	No calculation at this stage											
4	0	17.20	-0	14.65	0	-0	0	14.50	-0	15.20	0	-0
5	1	14.25	-0	19.00	1	-0	1	18.50	-1	13.20	1	-1
6	11	16.00	-0	19.20	15	-0	7	17.60	-4	15.20	10	-6
7	No calculation at this stage											
8	11	16.00	-0	19.20	14	-0	7	17.60	-4	15.20	9	-5
9	10	13.60	-39	16.00	14	-53	43	14.65	-31	18.50	58	-42
10	11	13.60	-39	16.00	14	-53	43	14.65	-31	18.50	58	-42
11	12	13.60	-40	16.00	16	-54	45	14.65	-31	18.50	61	-42
12	No calculation at this stage											
13	No calculation at this stage											
14	13	13.60	-42	16.80	18	-57	40	14.65	-29	19.20	54	-39
15	16	13.60	-46	16.40	22	-62	49	14.65	-30	19.20	66	-41
16	16	13.60	-45	16.40	22	-61	47	14.65	-30	19.20	64	-40
17	17	13.60	-48	16.40	23	-65	48	14.65	-31	19.20	65	-42

18	17	13.60	-48	16.40	23	-65	46	14.50	-31	19.20	62	-42
19	10	13.60	-44	16.80	14	-59	43	15.20	-30	19.20	59	-40
20	12	13.60	-39	16.80	16	-52	45	15.20	-27	19.20	61	-36
21	10	13.60	-43	16.80	14	-58	66	15.20	-29	19.20	89	-39

Maximum and minimum displacement at each stage

Stage no.	Displacement maximum m	elev.	minimum m	elev.	Stage description
1	0.000	14.00	-0.000	17.60	Change EI of wall to 100.00kN.m2/m run
2	0.000	12.00	-0.000	18.25	Apply surcharge no.1 at elev. 18.00
3	Wall displacements reset to zero				Change EI of wall to 30284kN.m2/m run
4	0.000	12.00	-0.000	19.20	Apply surcharge no.2 at elev. 18.00
5	0.001	19.20	0.000	19.20	Apply surcharge no.3 at elev. 19.00
6	0.005	19.20	0.000	19.20	Excav. to elev. 18.00 on PASSIVE side
7	No calculation at this stage				Install strut no.1 at elev. 18.50
8	0.005	19.20	0.000	19.20	Apply water pressure profile no.2
9	0.007	16.40	0.000	19.20	Excav. to elev. 14.65 on PASSIVE side
10	0.007	16.40	0.000	19.20	Change soil type 3 to soil type 5
11	0.007	16.40	0.000	19.20	Fill to elev. 15.40 on PASSIVE side
12	No calculation at this stage				Install strut no.3 at elev. 15.20
13	No calculation at this stage				Install strut no.2 at elev. 19.20
14	0.008	16.40	0.000	19.20	Remove strut no.1 at elev. 18.50
15	0.008	16.40	0.000	19.20	Apply surcharge no.4 at elev. 15.20
16	0.008	16.40	0.000	19.20	Apply water pressure profile no.3
17	0.008	16.40	0.000	19.20	Change soil type 2 to soil type 6
18	0.008	16.40	0.000	19.20	Change soil type 5 to soil type 6
19	0.008	16.40	0.000	19.20	Change soil type 4 to soil type 7
20	0.008	16.80	0.000	19.20	Change EI of wall to 21640kN.m2/m run
21	0.008	16.80	0.000	19.20	Apply water pressure profile no.4

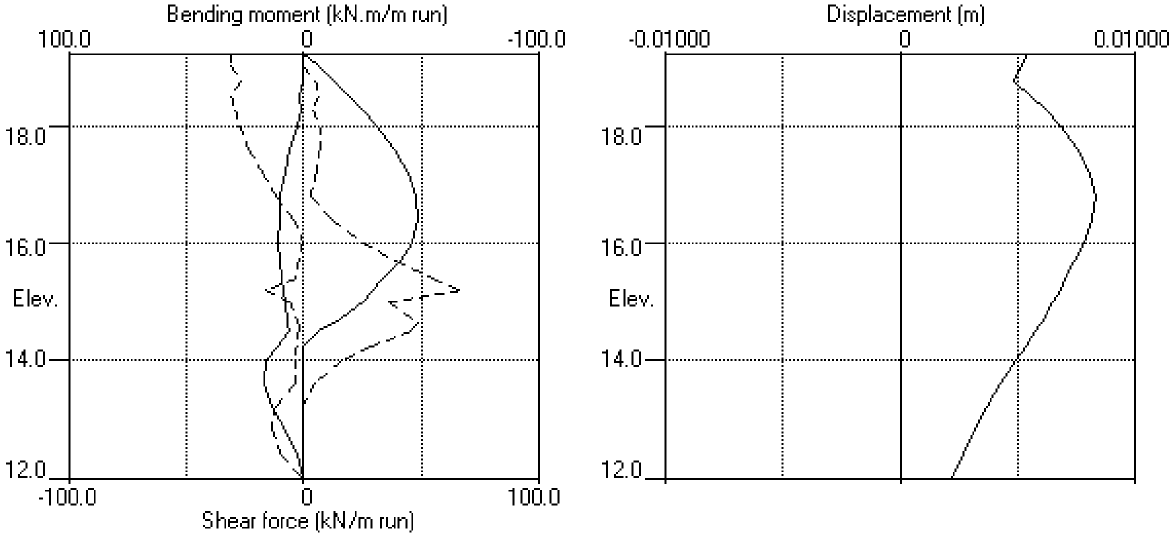
Calculated Bending Moments and Strut Forces have been multiplied by a factor of 1.35 to obtain values for structural design.

Strut forces at each stage (horizontal components)

Stage no.	Strut no. 1 at elev. 18.50 --Calculated-- kN per m run	kN per strut	Factored kN per strut	Strut no. 2 at elev. 19.20 --Calculated-- kN per m run	kN per strut	Factored kN per strut	Strut no. 3 at elev. 15.20 --Calculated-- kN per m run	kN per strut	Factored kN per strut
8	0	1	1	---	---	---	---	---	---
9	37	187	252	---	---	---	---	---	---
10	37	187	253	---	---	---	---	---	---
11	38	189	255	---	---	---	---	---	---
14	---	---	---	29	29	39	12	12	17
15	---	---	---	30	30	41	slack	slack	slack
16	---	---	---	30	30	40	slack	slack	slack
17	---	---	---	31	31	42	12	12	16
18	---	---	---	31	31	42	15	15	21
19	---	---	---	30	30	40	35	35	47
20	---	---	---	27	27	36	43	43	57
21	---	---	---	29	29	39	82	82	111

* Indicates that the total force shown is the sum of the force in the strut plus a force applied at the same elevation which may represent temperature load or other forces which are part of the strut load. Force components are listed in the detailed results for individual stages.

Bending moment, shear force, displacement envelopes



APPENDIX B – WALLAP INPUT / OUTPUT – COM 2

```
SOUTHERN GEOTECHNICAL DESIGN                                | Sheet No.
Program: WALLAP  Version 6.05  Revision A45.B58.R49         | Job No.   C0745
                             Licensed from GEOSOLVE         | Made by :    MP
Data filename/Run ID: Com 2                                 |
66 Fitzjohns Avenue, London NW3 5LT                         | Date:23-05-2016
Com 2                                                       | Checked :
-----------------------------------------------------------------------------
                                                       Units: kN,m
```

INPUT DATA

SOIL PROFILE

```
Stratum   Elevation of    ------------------ Soil types -------------------
  no.    top of stratum   Active side                Passive side
   1          19.00       1  Made Ground             1  Made Ground
   2          16.00       2  Claygate Undr           2  Claygate Undr
   3          15.00       2  Claygate Undr           3  Claygate To soft
   4          14.50       4  London Clay Undr        4  London Clay Undr
```

SOIL PROPERTIES (Unfactored SLS soil strengths)

```
                  Bulk    Young's   At rest  Consol  Active  Passive
-- Soil type --  density  Modulus    coeff.  state.  limit    limit   Cohesion
No. Description   kN/m3  Eh,kN/m2     Ko     NC/OC    Ka       Kp      kN/m2
  (Datum elev.)          (dEh/dy ) (dKo/dy) (  Nu ) ( Kac ) (  Kpc ) ( dc/dy )
 1  Made Ground   18.00     10000    0.500     OC    0.333    4.369
                                           (0.200) (0.000) ( 0.000)
 2  Claygate      20.00     32000    1.000     OC    1.000    1.000     32.00u
    Undr                                   (0.490) (2.000) ( 2.000)
 3  Claygate To    20.00     32000    1.000     OC    1.000    1.000     32.00u
    soft                                   (0.490) (2.000) ( 2.000)
 4  London Cl..   20.00     44000    1.000     OC    1.000    1.000     44.00u
    (   14.50 )          (   1520)         (0.490) (2.000) ( 2.390) (  1.520)
 5  Claygate ..   20.00         1    1.000     OC    1.000    1.000     1.000u
    (   15.00 )          (  64000)         (0.490) (2.000) ( 2.000) (  64.00)
 6  Claygate Dr   20.00     22400    1.000     OC    0.455    2.198       0.0d
                                           (0.150) (1.349) ( 2.965)
 7  London Cl..   20.00     30800    1.000     OC    0.422    3.077       0.0d
    (   14.50 )          (   1070)         (0.150) (1.299) ( 4.665)
```

Additional soil parameters associated with Ka and Kp

```
                         --- parameters for Ka ---  --- parameters for Kp ---
                           Soil     Wall    Back-    Soil     Wall    Back-
------- Soil type ------- friction adhesion fill   friction adhesion fill
No. Description            angle    coeff.  angle    angle    coeff.  angle
 1  Made Ground            30.00    0.000   0.00     30.00    0.500   0.00
 2  Claygate Undr           0.00    0.000   0.00      0.00    0.000   0.00
 3  Claygate To soft        0.00    0.000   0.00      0.00    0.000   0.00
 4  London Clay Undr        0.00    0.000   0.00      0.00    0.500   0.00
 5  Claygate Soft           0.00    0.000   0.00      0.00    0.000   0.00
 6  Claygate Dr            22.00    0.000   0.00     22.00    0.000   0.00
 7  London Clay LT         24.00    0.000   0.00     24.00    0.500   0.00
```

GROUND WATER CONDITIONS

Density of water = 10.00 kN/m3

	Active side	Passive side
Initial water table elevation	16.40	16.40

Automatic water pressure balancing at toe of wall : No

Water press. profile no.	Active side Point no.	Elev. m	Piezo elev. m	Water press. kN/m2	Passive side Point no.	Elev. m	Piezo elev. m	Water press. kN/m2	
1	1	16.40	16.40	0.0	1	15.00	15.00	0.0	MC
					2	13.00	16.40	34.0	
2	1	16.40	16.40	0.0	1	14.65	14.65	0.0	WC
					2	12.60	16.40	38.0	
3	1	16.40	16.40	0.0	1	15.00	15.00	0.0	MC+WC
					2	14.90	16.40	15.0	
4	1	18.00	18.00	0.0	1	15.00	15.00	0.0	WC
					2	14.90	16.40	15.0	

WALL PROPERTIES

Type of structure = Fully Embedded Wall
Elevation of toe of wall = 12.00
Maximum finite element length = 0.40 m
Youngs modulus of wall E = 2.2600E+07 kN/m2
Moment of inertia of wall I = 1.3400E-03 m4/m run
E.I = 30284 kN.m2/m run
Yield Moment of wall = Not defined

STRUTS and ANCHORS

Strut/ anchor no.	Elev.	Strut spacing m	X-section area of strut sq.m	Youngs modulus kN/m2	Free length m	Inclin -ation (degs)	Pre-stress /strut kN	Tension allowed
1	18.50	5.00	0.020000	2.000E+08	5.00	0.00	0	No
2	19.20	1.00	0.400000	2.000E+07	5.00	0.00	0	No
3	15.20	1.00	0.400000	2.000E+07	5.00	0.00	0	No

SURCHARGE LOADS

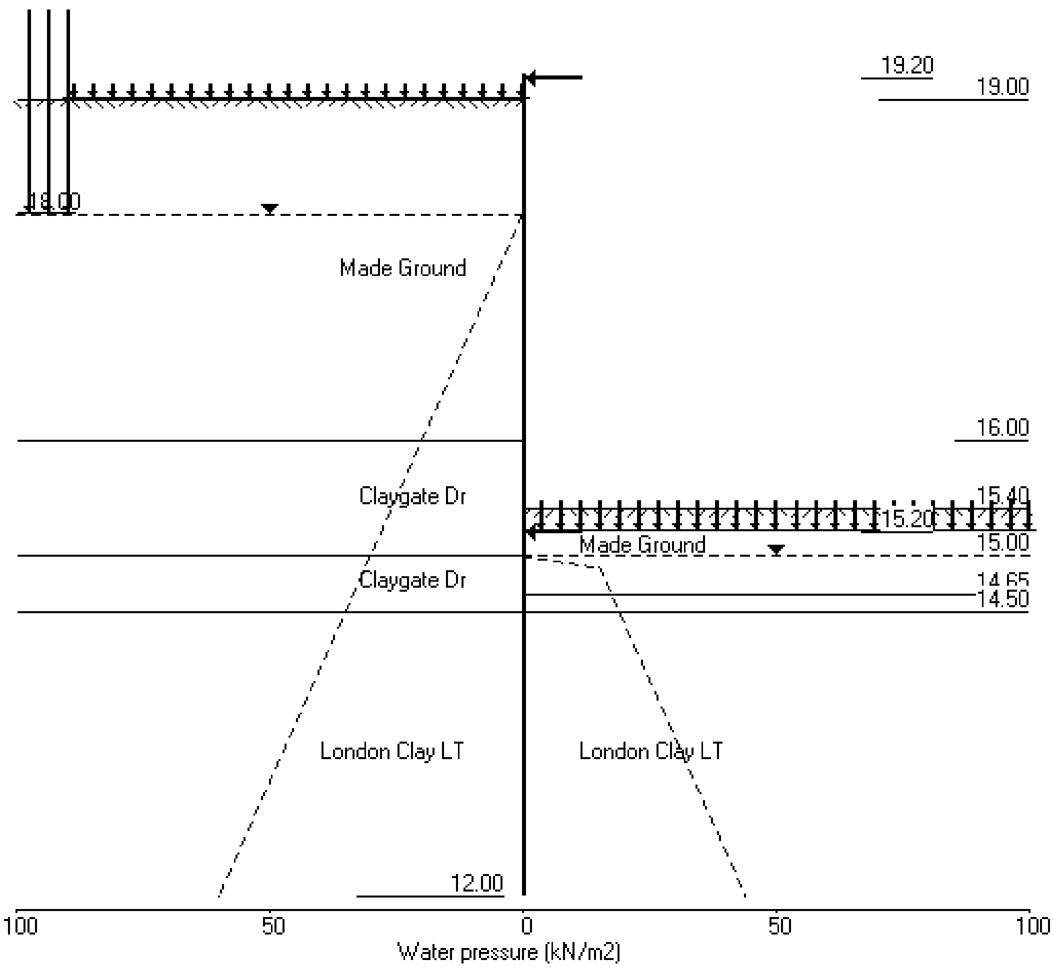
Surch -arge no.	Elev.	Distance from wall	Length parallel to wall	Width perpend. to wall	Surcharge kN/m2 Near edge	Surcharge kN/m2 Far edge	Equiv. soil type	Partial factor/ Category
1	18.00	4.00(A)	100.00	1.00	115.00	=	N/A	1.00 -
2	18.00	4.00(A)	100.00	1.00	17.00	=	N/A	1.00 -
3	19.00	0.00(A)	100.00	4.00	10.00	=	N/A	1.00 -
4	15.20	-0.00(P)	100.00	100.00	20.00	=	N/A	1.00 -

Note: A = Active side, P = Passive side
Limit State Categories P/U = Permanent Unfavourable
P/F = Permanent Favourable
Var = Variable (unfavourable)

```
CONSTRUCTION STAGES
Construction   Stage description
  stage no.    -------------------------------------------------------
     1         Change EI of wall to 100.00 kN.m2/m run
               100.00 kN.m2/m run
               No adjustments to wall displacements
     2         Apply surcharge no.1 at elevation 18.00
     3         Change EI of wall to 30284 kN.m2/m run
               30284 kN.m2/m run
               Reset wall displacements to zero at this stage
     4         Apply surcharge no.2 at elevation 18.00
     5         Apply surcharge no.3 at elevation 19.00
     6         Excavate to elevation 18.00 on PASSIVE side
     7         Install strut or anchor no.1 at elevation 18.50
     8         Apply water pressure profile no.2  ( Worst Cred. )
     9         Excavate to elevation 14.65 on PASSIVE side
    10         Change properties of soil type 3 to soil type 5
               Ko pressures will not be reset
    11         Fill to elevation 15.40 on PASSIVE side with soil type 1
    12         Install strut or anchor no.3 at elevation 15.20
    13         Install strut or anchor no.2 at elevation 19.20
    14         Remove strut or anchor no.1 at elevation 18.50
    15         Apply surcharge no.4 at elevation 15.20
    16         Apply water pressure profile no.3  ( Worst Cred. )
    17         Change properties of soil type 2 to soil type 6
               Ko pressures will not be reset
    18         Change properties of soil type 5 to soil type 6
               Ko pressures will not be reset
    19         Change properties of soil type 4 to soil type 7
               Ko pressures will not be reset
    20         Change EI of wall to 21640 kN.m2/m run
               Yield moment not defined
               Allow wall to relax with new modulus value
    21         Apply water pressure profile no.4  ( Worst Cred. )

FACTORS OF SAFETY and ANALYSIS OPTIONS
   Limit State options: ULS DA1 Combination 2
      Water pressures : Worst Credible
      Partial factor on   C'         = 1.250
      Partial factor on Phi'         = 1.250
      Partial factor on   Cu         = 1.400
      Partial factor on Soil Modulus = 1.000
      Partial factor on Permanent Unfavourable loads = 1.000
      Partial factor on Permanent Favourable loads   = 1.000
      Partial factor on Permanent Variable loads     = 1.300

   Stability analysis:
      Method of analysis  -  Strength Factor method
      Overall factor on soil strength for calculating wall depth = 1.20

   Parameters for undrained strata:
      Minimum equivalent fluid density             =   5.00 kN/m3
      Maximum depth of water filled tension crack  =   0.00 m

   Bending moment and displacement calculation:
      Method  -  Subgrade reaction model using Influence Coefficients
      Open Tension Crack analysis? - No
      Non-linear Modulus Parameter (L) = 0 m

   Boundary conditions:
```

```
Length of wall (normal to plane of analysis) = 1000.00 m

Width of excavation on active  side of wall  = 20.00 m
Width of excavation on passive side of wall  = 20.00 m

Distance to rigid boundary on active side  = 20.00 m
Distance to rigid boundary on passive side = 20.00 m
```

Stage No.21 Apply water pressure profile no.4 (Worst Cred.)



Summary of results

LIMIT STATE PARAMETERS

```
   Limit State: ULS DA1 Combination 2
   Water pressures : Worst Credible
   Partial factor on   C'         = 1.250
   Partial factor on Phi'         = 1.250
   Partial factor on   Cu         = 1.400
   Partial factor on Soil Modulus = 1.000
   Partial factor on Permanent Unfavourable loads = 1.000
   Partial factor on Permanent Favourable loads   = 1.000
```

Partial factor on Permanent Variable loads = 1.300

STABILITY ANALYSIS of Fully Embedded Wall according to Strength Factor method

Factor of safety on soil strength

Stage No.	G.L. Act.	G.L. Pass.	Strut Elev.	Overall FoS for toe elev. = 12.00: Factor of Safety	Moment equilib. at elev.	Toe elev. for FoS = 1.200: Toe elev.	Wall Penetr -ation
1	19.00	19.00	Cant.	Conditions not suitable for FoS calc.			
2	19.00	19.00	Cant.	Conditions not suitable for FoS calc.			
3	19.00	19.00		No analysis at this stage			
4	19.00	19.00	Cant.	Conditions not suitable for FoS calc.			
5	19.00	19.00	Cant.	6.595	12.30	18.70	0.30
6	19.00	18.00	Cant.	2.242	12.56	15.82	2.18
7	19.00	18.00		No analysis at this stage			
8	19.00	18.00	18.50	3.691	n/a	17.43	0.57
9	19.00	14.65	18.50	1.219	n/a	12.18	2.47
10	19.00	14.65	18.50	1.217	n/a	12.16	2.49
11	19.00	15.40	18.50	1.391	n/a	13.15	2.25
12	19.00	15.40		No analysis at this stage			

All remaining stages have more than one strut - FoS calculation n/a

BENDING MOMENT and DISPLACEMENT ANALYSIS of Fully Embedded Wall

Analysis options

Length of wall perpendicular to section = 1000.00m
Subgrade reaction model - Boussinesq Influence coefficients
Soil deformations are elastic until the active or passive limit is reached
Open Tension Crack analysis - No

Rigid boundaries: Active side 20.00 from wall
Passive side 20.00 from wall

Limit State: ULS DA1 Combination 2

Bending moment, shear force and displacement envelopes

Node no.	Y coord	Displacement maximum m	Displacement minimum m	Bending moment maximum kN.m/m	Bending moment minimum kN.m/m	Shear force maximum kN/m	Shear force minimum kN/m
1	19.20	0.007	-0.000	0.0	-0.0	0.0	-40.3
2	19.00	0.007	-0.000	0.0	-8.1	0.0	-40.3
3	18.75	0.006	-0.000	1.0	-17.2	5.8	-34.9
4	18.50	0.007	-0.000	3.0	-25.4	8.8	-41.6
5	18.25	0.008	0.000	1.7	-33.2	5.1	-39.4
6	18.00	0.009	0.000	3.3	-40.3	7.8	-36.8
7	17.60	0.010	0.000	7.1	-49.9	8.9	-31.6
8	17.20	0.010	0.000	10.2	-57.6	6.9	-25.2
9	16.80	0.011	0.000	12.6	-62.4	5.3	-17.5
10	16.40	0.011	0.000	14.5	-64.0	13.9	-8.6
11	16.00	0.010	0.000	16.0	-61.8	29.1	-0.0
12	15.70	0.010	0.000	16.1	-56.8	44.7	-1.4
13	15.40	0.009	0.000	15.1	-51.3	61.8	-4.4
14	15.20	0.009	0.000	14.1	-46.9	73.8	-26.1
15	15.00	0.009	0.000	12.9	-41.2	41.6	-15.4
16	14.65	0.008	0.000	10.8	-27.2	53.1	-5.5
17	14.50	0.008	0.000	10.0	-19.8	51.6	-4.9
18	14.25	0.007	0.000	8.7	-10.4	39.6	-5.7

19	14.00	0.006	0.000	8.9	-5.6	28.6	-5.9
20	13.60	0.005	0.000	15.9	-0.0	11.9	-5.5
21	13.20	0.005	0.000	15.0	-0.0	3.6	-8.1
22	12.80	0.004	0.000	11.0	-0.0	0.0	-15.9
23	12.40	0.003	0.000	3.9	-0.0	0.0	-13.8
24	12.00	0.002	-0.000	0.0	-0.0	0.0	-0.0

Maximum and minimum bending moment and shear force at each stage

Stage no.	Bending moment maximum kN.m/m	elev.	minimum kN.m/m	elev.	Shear force maximum kN/m	elev.	minimum kN/m	elev.
1	0.0	13.60	-0.0	14.25	0.0	19.20	0.0	19.20
2	0.0	14.25	-0.0	15.00	0.2	14.50	-0.1	15.20
3	No calculation at this stage							
4	0.0	17.20	-0.1	14.65	0.2	14.50	-0.1	15.20
5	2.1	16.00	-0.0	19.00	1.1	18.00	-0.8	13.60
6	16.1	15.70	0.0	19.20	8.9	17.60	-6.0	15.00
7	No calculation at this stage							
8	15.8	15.70	0.0	19.20	8.7	17.60	-5.9	15.00
9	10.6	13.20	-55.9	16.00	49.4	14.50	-41.0	18.50
10	10.6	13.20	-56.4	16.00	50.1	14.50	-41.2	18.50
11	11.5	13.20	-57.0	16.00	50.9	14.50	-41.6	18.50
12	No calculation at this stage							
13	No calculation at this stage							
14	12.3	13.20	-58.1	16.40	44.9	14.50	-38.3	19.20
15	15.3	13.60	-62.2	16.40	53.1	14.65	-39.7	19.20
16	15.6	13.60	-62.4	16.40	52.8	14.65	-39.8	19.20
17	15.9	13.60	-64.0	16.40	51.8	14.65	-40.3	19.20
18	15.7	13.60	-63.8	16.40	51.3	14.50	-40.3	19.20
19	5.5	12.80	-54.6	16.80	50.7	15.20	-37.5	19.20
20	6.0	12.80	-47.6	16.80	52.5	15.20	-33.3	19.20
21	4.3	12.80	-51.6	16.80	73.8	15.20	-35.7	19.20

Maximum and minimum displacement at each stage

Stage no.	Displacement maximum m	elev.	minimum m	elev.	Stage description
1	0.000	14.00	-0.000	17.60	Change EI of wall to 100.00kN.m2/m run
2	0.000	12.00	-0.000	18.25	Apply surcharge no.1 at elev. 18.00
3	Wall displacements reset to zero				Change EI of wall to 30284kN.m2/m run
4	0.000	12.00	-0.000	19.20	Apply surcharge no.2 at elev. 18.00
5	0.001	19.20	0.000	19.20	Apply surcharge no.3 at elev. 19.00
6	0.007	19.20	0.000	19.20	Excav. to elev. 18.00 on PASSIVE side
7	No calculation at this stage				Install strut no.1 at elev. 18.50
8	0.007	19.20	0.000	19.20	Apply water pressure profile no.2
9	0.010	16.00	0.000	19.20	Excav. to elev. 14.65 on PASSIVE side
10	0.010	16.00	0.000	19.20	Change soil type 3 to soil type 5
11	0.010	16.40	0.000	19.20	Fill to elev. 15.40 on PASSIVE side
12	No calculation at this stage				Install strut no.3 at elev. 15.20
13	No calculation at this stage				Install strut no.2 at elev. 19.20
14	0.010	16.40	0.000	19.20	Remove strut no.1 at elev. 18.50
15	0.010	16.40	0.000	19.20	Apply surcharge no.4 at elev. 15.20
16	0.010	16.40	-0.000	12.00	Apply water pressure profile no.3
17	0.010	16.40	-0.000	12.00	Change soil type 2 to soil type 6
18	0.010	16.40	-0.000	12.00	Change soil type 5 to soil type 6
19	0.010	16.40	0.000	19.20	Change soil type 4 to soil type 7
20	0.010	16.40	0.000	19.20	Change EI of wall to 21640kN.m2/m run
21	0.011	16.80	0.000	19.20	Apply water pressure profile no.4

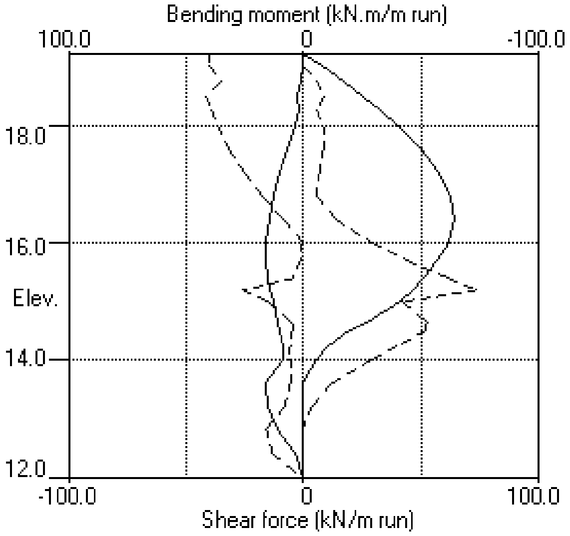
Strut forces at each stage (horizontal components)

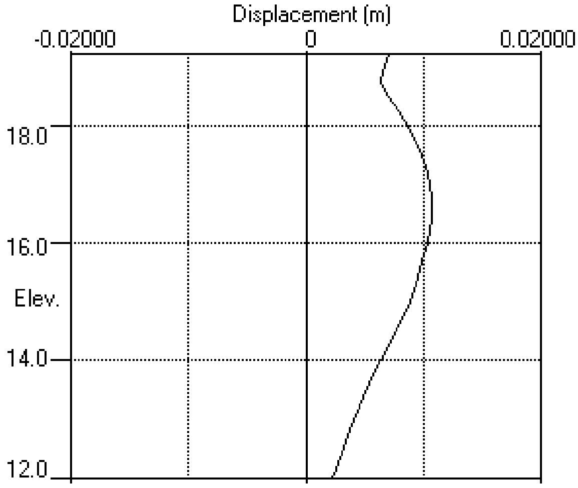
Stage no.	--- Strut no. 1 --- at elev. 18.50 kN/m run	kN/strut	--- Strut no. 2 --- at elev. 19.20 kN/m run	kN/strut	--- Strut no. 3 --- at elev. 15.20 kN/m run	kN/strut
8	0.21	1.07	---	---	---	---
9	49.75	248.75	---	---	---	---
10	50.02	250.09	---	---	---	---
11	50.26	251.29	---	---	---	---
14	---	---	38.29	38.29	16.66	16.66
15	---	---	39.72	39.72	1.99	1.99
16	---	---	39.78	39.78	slack	slack
17	---	---	40.35	40.35	14.65	14.65
18	---	---	40.27	40.27	16.78	16.78
19	---	---	37.53	37.53	50.97	50.97
20	---	---	33.31	33.31	59.62	59.62
21	---	---	35.68	35.68	99.87	99.87

* Indicates that the total force shown is the sum of the force in the strut plus a force applied at the same elevation which may represent temperature load or other forces which are part of the strut load. Force components are listed in the detailed results for individual stages.

Units: kN,m

Bending moment, shear force, displacement envelopes





APPENDIX B – WALLAP INPUT / OUTPUT – SLS

```
SOUTHERN GEOTECHNICAL DESIGN                                   | Sheet No.
Program: WALLAP  Version 6.05  Revision A45.B58.R49            | Job No.   C0745
                           Licensed from GEOSOLVE              | Made by :    MP
Data filename/Run ID: SLS                                      |
66 Fitzjohns Avenue, London NW3 5LT                            | Date:23-05-2016
SLS                                                            | Checked :
------------------------------------------------------------------------------
                                                       Units: kN,m
```

INPUT DATA

SOIL PROFILE

Stratum no.	Elevation of top of stratum	Soil types: Active side	Soil types: Passive side
1	19.00	1 Made Ground	1 Made Ground
2	16.00	2 Claygate Undr	2 Claygate Undr
3	15.00	2 Claygate Undr	3 Claygate To soft
4	14.50	4 London Clay Undr	4 London Clay Undr

SOIL PROPERTIES

No.	-- Soil type -- Description (Datum elev.)	Bulk density kN/m3	Young's Modulus Eh,kN/m2 (dEh/dy)	At rest coeff. Ko (dKo/dy)	Consol state. NC/OC (Nu)	Active limit Ka (Kac)	Passive limit Kp (Kpc)	Cohesion kN/m2 (dc/dy)
1	Made Ground	18.00	10000	0.500	OC	0.333	4.369	
					(0.200)	(0.000)	(0.000)	
2	Claygate	20.00	32000	1.000	OC	1.000	1.000	32.00u
	Undr				(0.490)	(2.000)	(2.000)	
3	Claygate To	20.00	32000	1.000	OC	1.000	1.000	32.00u
	soft				(0.490)	(2.000)	(2.000)	
4	London Cl..	20.00	44000	1.000	OC	1.000	1.000	44.00u
	(14.50)		(1520)		(0.490)	(2.000)	(2.390)	(1.520)
5	Claygate ..	20.00	1	1.000	OC	1.000	1.000	1.000u
	(15.00)		(64000)		(0.490)	(2.000)	(2.000)	(64.00)
6	Claygate Dr	20.00	22400	1.000	OC	0.455	2.198	0.0d
					(0.150)	(1.349)	(2.965)	
7	London Cl..	20.00	30800	1.000	OC	0.422	3.077	0.0d
	(14.50)		(1070)		(0.150)	(1.299)	(4.665)	

Additional soil parameters associated with Ka and Kp

No.	------- Soil type ------- Description	Ka: Soil friction angle	Ka: Wall adhesion coeff.	Ka: Back-fill angle	Kp: Soil friction angle	Kp: Wall adhesion coeff.	Kp: Back-fill angle
1	Made Ground	30.00	0.000	0.00	30.00	0.500	0.00
2	Claygate Undr	0.00	0.000	0.00	0.00	0.000	0.00
3	Claygate To soft	0.00	0.000	0.00	0.00	0.000	0.00
4	London Clay Undr	0.00	0.000	0.00	0.00	0.500	0.00
5	Claygate Soft	0.00	0.000	0.00	0.00	0.000	0.00
6	Claygate Dr	22.00	0.000	0.00	22.00	0.000	0.00
7	London Clay LT	24.00	0.000	0.00	24.00	0.500	0.00

GROUND WATER CONDITIONS

Density of water = 10.00 kN/m3

	Active side	Passive side
Initial water table elevation	16.40	16.40

Automatic water pressure balancing at toe of wall : No

Water press. profile no.	Active side Point no.	Elev. m	Piezo elev. m	Water press. kN/m2	Passive side Point no.	Elev. m	Piezo elev. m	Water press. kN/m2	
1	1	16.40	16.40	0.0	1	15.00	15.00	0.0	MC
					2	13.00	16.40	34.0	
2	1	16.40	16.40	0.0	1	14.65	14.65	0.0	WC
					2	12.60	16.40	38.0	
3	1	16.40	16.40	0.0	1	15.00	15.00	0.0	MC+WC
					2	14.90	16.40	15.0	
4	1	18.00	18.00	0.0	1	15.00	15.00	0.0	WC
					2	14.90	16.40	15.0	

WALL PROPERTIES

```
                 Type of structure = Fully Embedded Wall
             Elevation of toe of wall = 12.00
        Maximum finite element length =  0.40 m
             Youngs modulus of wall E = 2.2600E+07 kN/m2
           Moment of inertia of wall I = 1.3400E-03 m4/m run
                                  E.I = 30284 kN.m2/m run
                  Yield Moment of wall = Not defined
```

STRUTS and ANCHORS

Strut/ anchor no.	Elev.	Strut spacing m	X-section area of strut sq.m	Youngs modulus kN/m2	Free length m	Inclin -ation (degs)	Pre-stress /strut kN	Tension allowed
1	18.50	5.00	0.020000	2.000E+08	5.00	0.00	0	No
2	19.20	1.00	0.400000	2.000E+07	5.00	0.00	0	No
3	15.20	1.00	0.400000	2.000E+07	5.00	0.00	0	No

SURCHARGE LOADS

Surch -arge no.	Elev.	Distance from wall	Length parallel to wall	Width perpend. to wall	Surcharge kN/m2 Near edge	Surcharge kN/m2 Far edge	Equiv. soil type	Partial factor/ Category
1	18.00	4.00(A)	100.00	1.00	115.00	=	N/A	1.00 -
2	18.00	4.00(A)	100.00	1.00	17.00	=	N/A	1.00 -
3	19.00	0.00(A)	100.00	4.00	10.00	=	N/A	1.00 -
4	15.20	-0.00(P)	100.00	100.00	20.00	=	N/A	1.00 -

Note: A = Active side, P = Passive side
Limit State Categories P/U = Permanent Unfavourable
P/F = Permanent Favourable
Var = Variable (unfavourable)

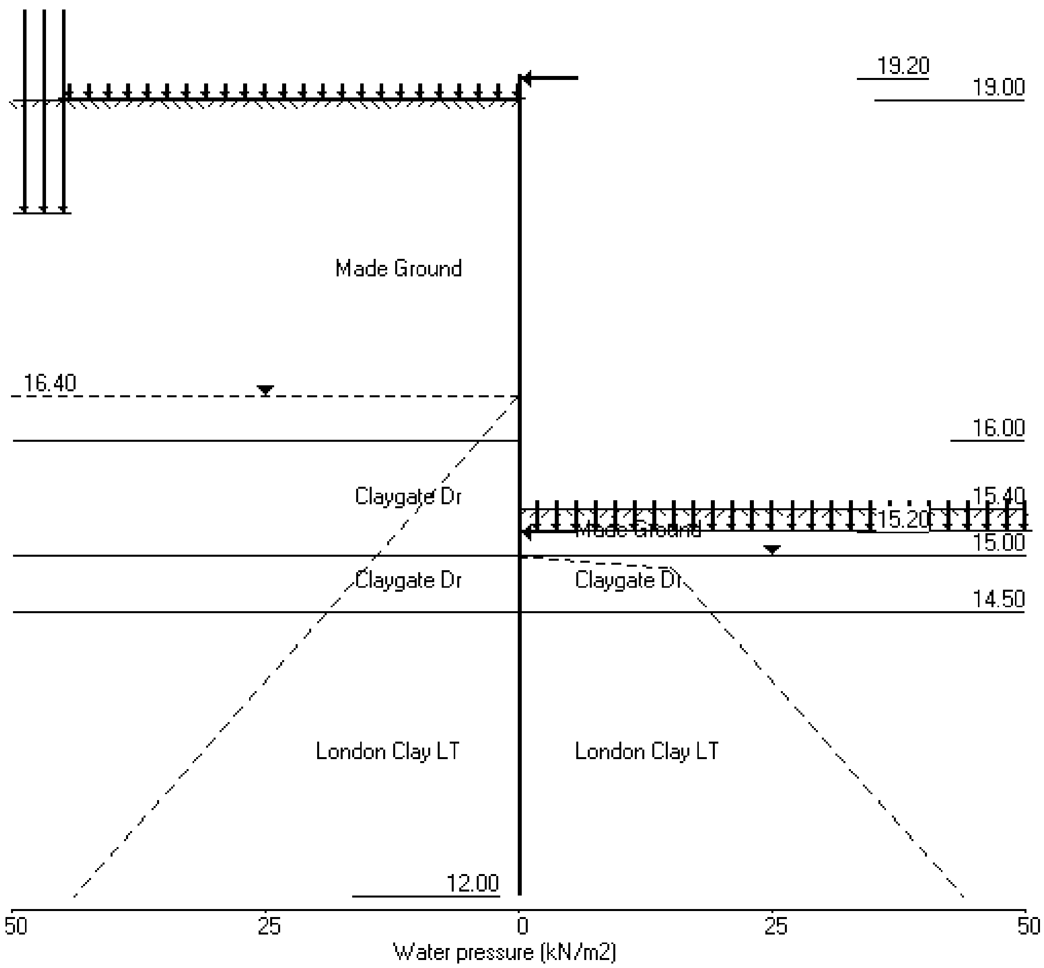
CONSTRUCTION STAGES

```
Construction   Stage description
  stage no.    -------------------------------------------------------
      1        Change EI of wall to 100.00 kN.m2/m run
               100.00 kN.m2/m run
               No adjustments to wall displacements
      2        Apply surcharge no.1 at elevation 18.00
      3        Change EI of wall to 30284 kN.m2/m run
               30284 kN.m2/m run
               Reset wall displacements to zero at this stage
      4        Apply surcharge no.2 at elevation 18.00
      5        Apply surcharge no.3 at elevation 19.00
      6        Excavate to elevation 18.00 on PASSIVE side
      7        Install strut or anchor no.1 at elevation 18.50
      8        Apply water pressure profile no.1  ( Mod. Conserv. )
      9        Excavate to elevation 15.00 on PASSIVE side
     10        Change properties of soil type 3 to soil type 5
               Ko pressures will not be reset
     11        Fill to elevation 15.40 on PASSIVE side with soil type 1
     12        Install strut or anchor no.3 at elevation 15.20
     13        Install strut or anchor no.2 at elevation 19.20
     14        Remove strut or anchor no.1 at elevation 18.50
     15        Apply surcharge no.4 at elevation 15.20
     16        Apply water pressure profile no.3  ( Mod. Conserv. )
     17        Change properties of soil type 2 to soil type 6
               Ko pressures will not be reset
     18        Change properties of soil type 5 to soil type 6
               Ko pressures will not be reset
     19        Change properties of soil type 4 to soil type 7
               Ko pressures will not be reset
     20        Change EI of wall to 21640 kN.m2/m run
               Yield moment not defined
               Allow wall to relax with new modulus value
     21        Apply water pressure profile no.3  ( Mod. Conserv. )
```

FACTORS OF SAFETY and ANALYSIS OPTIONS

```
   Limit State options: Serviceability Limit State
      All loads and soil strengths are unfactored

   Stability analysis:
      Method of analysis  -  Strength Factor method
      Factor on soil strength for calculating wall depth = 1.00

   Parameters for undrained strata:
      Minimum equivalent fluid density             =   5.00 kN/m3
      Maximum depth of water filled tension crack  =   0.00 m

   Bending moment and displacement calculation:
      Method  -  Subgrade reaction model using Influence Coefficients
      Open Tension Crack analysis? - No
      Non-linear Modulus Parameter (L) = 0 m

   Boundary conditions:
      Length of wall (normal to plane of analysis) = 1000.00 m

      Width of excavation on active  side of wall  = 20.00 m
      Width of excavation on passive side of wall  = 20.00 m

      Distance to rigid boundary on active side  = 20.00 m
      Distance to rigid boundary on passive side = 20.00 m
```

Stage No.21 Apply water pressure profile no.3 (Mod. Conserv.)



Summary of results

LIMIT STATE PARAMETERS

Limit State: Serviceability Limit State

All loads and soil strengths are unfactored

STABILITY ANALYSIS of Fully Embedded Wall according to Strength Factor method

Factor of safety on soil strength

				FoS for toe elev. = 12.00		Toe elev. for FoS = 1.000	
Stage No.	--- G.L. --- Act.	Pass.	Strut Elev.	Factor of Safety	Moment equilib. at elev.	Toe elev.	Wall Penetr -ation
1	19.00	19.00	Cant.	Conditions not suitable for FoS calc.			
2	19.00	19.00	Cant.	Conditions not suitable for FoS calc.			
3	19.00	19.00		No analysis at this stage			

4	19.00	19.00	Cant.	Conditions not suitable for FoS calc.			
5	19.00	19.00	Cant.	Conditions not suitable for FoS calc.			
6	19.00	18.00	Cant.	2.965	12.60	16.87	1.13
7	19.00	18.00		No analysis at this stage			
8	19.00	18.00	18.50	5.121	n/a	17.89	0.11
9	19.00	15.00	18.50	1.849	n/a	14.42	0.58
10	19.00	15.00	18.50	1.816	n/a	14.28	0.72
11	19.00	15.40	18.50	1.951	n/a	14.38	1.02
12	19.00	15.40		No analysis at this stage			

All remaining stages have more than one strut - FoS calculation n/a

BENDING MOMENT and DISPLACEMENT ANALYSIS of Fully Embedded Wall

Analysis options

Length of wall perpendicular to section = 1000.00m
Subgrade reaction model - Boussinesq Influence coefficients
Soil deformations are elastic until the active or passive limit is reached
Open Tension Crack analysis - No

Rigid boundaries: Active side 20.00 from wall
Passive side 20.00 from wall

Limit State: Serviceability Limit State

Calculated Bending Moments and Strut Forces have been multiplied by a factor of 1.35 to obtain values for structural design.

Bending moment, shear force and displacement envelopes

Node no.	Y coord	Displacement max. (m)	Displacement min. (m)	Bending moment Calculated max. (kN.m/m)	Bending moment Calculated min. (kN.m/m)	Bending moment Factored max. (kN.m/m)	Bending moment Factored min. (kN.m/m)	Shear force Calculated max. (kN/m)	Shear force Calculated min. (kN/m)	Shear force Factored max. (kN/m)	Shear force Factored min. (kN/m)
1	19.20	0.005	-0.000	0	-0	0	-0	0	-30	0	-41
2	19.00	0.005	-0.000	0	-6	0	-8	0	-30	0	-41
3	18.75	0.005	-0.000	1	-13	1	-17	4	-26	5	-35
4	18.50	0.005	-0.000	2	-19	3	-26	6	-30	9	-41
5	18.25	0.006	0.000	1	-25	2	-34	4	-28	6	-38
6	18.00	0.007	0.000	3	-30	4	-41	6	-26	9	-35
7	17.60	0.007	0.000	6	-37	8	-50	7	-22	10	-30
8	17.20	0.008	0.000	8	-43	11	-58	5	-17	7	-23
9	16.80	0.008	0.000	10	-46	13	-62	3	-11	4	-14
10	16.40	0.008	0.000	11	-46	14	-62	6	-3	8	-5
11	16.00	0.007	0.000	11	-44	15	-59	15	-0	20	-0
12	15.70	0.007	0.000	11	-38	14	-52	25	-2	34	-3
13	15.40	0.007	0.000	10	-31	13	-41	37	-4	50	-5
14	15.20	0.006	0.000	9	-27	12	-36	45	-4	61	-6
15	15.00	0.006	0.000	8	-21	11	-29	41	-4	56	-6
16	14.75	0.006	0.000	7	-12	10	-16	42	-4	57	-5
17	14.50	0.005	0.000	6	-3	8	-4	39	-2	53	-3
18	14.25	0.005	0.000	13	-0	18	-0	23	-3	32	-4
19	14.00	0.005	0.000	17	-0	23	-0	13	-4	18	-5
20	13.60	0.004	0.000	17	-0	23	-0	2	-4	3	-6
21	13.20	0.004	0.000	14	-0	18	-0	0	-12	0	-16
22	12.80	0.003	0.000	8	-0	11	-0	0	-13	0	-18
23	12.40	0.003	0.000	3	-0	4	-0	0	-10	0	-14
24	12.00	0.002	0.000	0	-0	0	-0	0	-0	0	-0

Calculated Bending Moments and Strut Forces have been multiplied by a factor of 1.35 to obtain values for structural design.

Maximum and minimum bending moment and shear force at each stage

Stage no.	BM Calculated max. kN.m/m	elev.	min. kN.m/m	elev.	BM Factored max. kN.m/m	min.	SF Calculated max. kN/m	elev.	min. kN/m	elev.	SF Factored max. kN/m	min. kN/m
1	0	13.60	-0	14.00	0	-0	0	19.20	0	19.20	0	0
2	0	14.25	-0	14.75	0	-0	0	14.50	-0	15.20	0	-0
3	No calculation at this stage											
4	0	17.20	-0	14.75	0	-0	0	14.50	-0	15.20	0	-0
5	1	14.25	-0	19.20	1	-0	1	18.50	-1	13.20	1	-1
6	11	16.00	-0	19.20	15	-0	7	17.60	-4	15.20	10	-6
7	No calculation at this stage											
8	11	16.00	-0	19.20	14	-0	7	17.60	-4	15.20	9	-5
9	10	13.60	-35	16.40	13	-47	35	15.00	-29	18.50	47	-39
10	11	13.60	-38	16.40	15	-51	37	14.75	-30	18.50	50	-41
11	12	13.60	-38	16.40	16	-51	37	14.75	-30	18.50	50	-41
12	No calculation at this stage											
13	No calculation at this stage											
14	13	13.60	-40	16.80	17	-54	32	14.75	-28	19.20	44	-37
15	16	13.60	-43	16.40	21	-57	40	14.75	-29	19.20	54	-39
16	16	13.60	-43	16.80	21	-57	40	14.75	-29	19.20	54	-39
17	17	13.60	-46	16.40	23	-62	42	14.75	-30	19.20	57	-41
18	17	13.60	-46	16.40	22	-61	42	15.20	-30	19.20	56	-40
19	13	13.60	-43	16.80	17	-58	43	15.20	-29	19.20	59	-39
20	13	13.60	-38	16.80	18	-52	45	15.20	-26	19.20	61	-36
21	13	13.60	-38	16.80	18	-52	45	15.20	-26	19.20	61	-36

Maximum and minimum displacement at each stage

Stage no.	Displacement maximum m	elev.	minimum m	elev.	Stage description
1	0.000	14.00	-0.000	17.60	Change EI of wall to 100.00kN.m2/m run
2	0.000	12.00	-0.000	18.25	Apply surcharge no.1 at elev. 18.00
3	Wall displacements reset to zero				Change EI of wall to 30284kN.m2/m run
4	0.000	12.00	-0.000	19.20	Apply surcharge no.2 at elev. 18.00
5	0.001	19.20	0.000	19.20	Apply surcharge no.3 at elev. 19.00
6	0.005	19.20	0.000	19.20	Excav. to elev. 18.00 on PASSIVE side
7	No calculation at this stage				Install strut no.1 at elev. 18.50
8	0.005	19.20	0.000	19.20	Apply water pressure profile no.1
9	0.007	16.40	0.000	19.20	Excav. to elev. 15.00 on PASSIVE side
10	0.007	16.40	0.000	19.20	Change soil type 3 to soil type 5
11	0.007	16.40	0.000	19.20	Fill to elev. 15.40 on PASSIVE side
12	No calculation at this stage				Install strut no.3 at elev. 15.20
13	No calculation at this stage				Install strut no.2 at elev. 19.20
14	0.007	16.40	0.000	19.20	Remove strut no.1 at elev. 18.50
15	0.007	16.80	0.000	19.20	Apply surcharge no.4 at elev. 15.20
16	0.007	16.80	0.000	19.20	Apply water pressure profile no.3
17	0.008	16.80	0.000	19.20	Change soil type 2 to soil type 6
18	0.008	16.80	0.000	19.20	Change soil type 5 to soil type 6
19	0.007	16.80	0.000	19.20	Change soil type 4 to soil type 7
20	0.008	16.80	0.000	19.20	Change EI of wall to 21640kN.m2/m run
21	0.008	16.80	0.000	19.20	Apply water pressure profile no.3

Calculated Bending Moments and Strut Forces have been multiplied by a factor of 1.35 to obtain values for structural design.

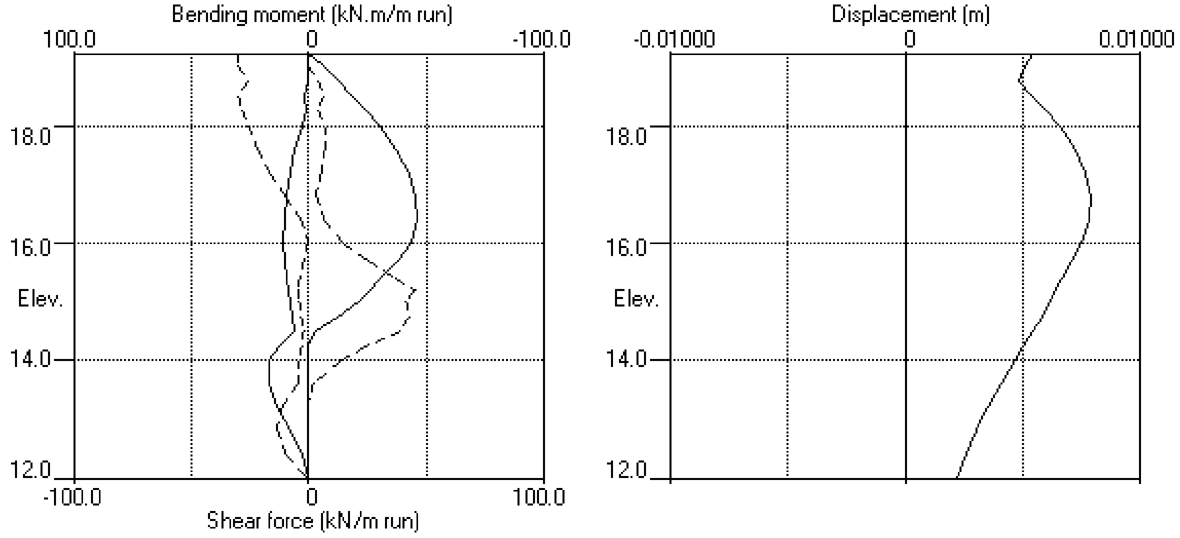
Strut forces at each stage (horizontal components)

Stage no.	Strut no. 1 at elev. 18.50 --Calculated-- kN per m run	kN per strut	Factored kN per strut	Strut no. 2 at elev. 19.20 --Calculated-- kN per m run	kN per strut	Factored kN per strut	Strut no. 3 at elev. 15.20 --Calculated-- kN per m run	kN per strut	Factored kN per strut
8	0	1	1	---	---	---	---	---	---
9	35	174	235	---	---	---	---	---	---
10	36	182	246	---	---	---	---	---	---
11	36	182	245	---	---	---	---	---	---
14	---	---	---	28	28	37	12	12	16
15	---	---	---	29	29	39	slack	slack	slack
16	---	---	---	29	29	39	slack	slack	slack
17	---	---	---	30	30	41	7	7	10
18	---	---	---	30	30	40	14	14	18
19	---	---	---	29	29	39	27	27	37
20	---	---	---	26	26	36	35	35	47
21	---	---	---	26	26	36	35	35	47

* Indicates that the total force shown is the sum of the force in the strut plus a force applied at the same elevation which may represent temperature load or other forces which are part of the strut load. Force components are listed in the detailed results for individual stages.

Units: kN,m

Bending moment, shear force, displacement envelopes



APPENDIX D REINFORCEMENT CALCULATIONS

REFERENCE
EC2

Rev:

Bending and Axial Force to EN 1992-1-1:2004 (EC2) - Secant Wall Circular Sections (Cast In-situ)

Pile section

	pile diameter	=	350	mm	
	Pile spacing	=	550	mm	
	design pile diameter h	=	330	mm	
	Ac	=	85530	mm^2	
4.4.1.3(4)	cover[1] c_{nom}	=	40	mm	k_2 = 50 mm
	cage diameter d	=	218	mm	
	ratio d/h	=	0.66		γ_c = 1.5 (This is adjusted by K_f=1.1 [2.4.2.5 (2)] to give 1.65)
	f_{ck}	=	28	MPa	γ_c = 1.65 α_{cc} = 1.0 [NA 3.1.6 (1)}
	f_{yk}	=	500	MPa	γ_s = 1.15 Already included in charts

Design Actions on pile

Actions N	=		kN	
Factored Actions N	=		kN	
Wallap shear	=	95	kN/m	
Shear V_{Ed}	=	52.25	kN	BM/SF factor 1.0
Ult Shear V_{Ed}	=	52.25	kN	
Wallap moment	=	68	kNm/m	
Induced Moment M_i	=	37.4	kNm	
Applied Moment M_{Ed}	=	0	kN	
Σ Moments M	=	37	kN	
Factored Ult M	=	37	kN	

Using IstructE design charts for circular columns:-

$M/h^3 f_{ck}$ = 0.04 (also checked for M/h3=0.0 for zero vertical load)

Actions N $N/h^2 f_{ck}$ = 0.00

Factored Actions N $N/h^2 f_{ck}$ = 0.00

therefore from charts;

$\rho\, f_{yk} / f_{ck}$ = 0.15 **From charts**

ρ = $4A_{st} / \pi.h^2$

therefore, adopt greater of:

Area of main steel Ast	=	718	mm^2	**or**	Minimum steel area to BS EN 1536: 481 mm^2 for 350 mm dia. pile	
main bar dia	=	16	mm		20	mm
no. main bars	=	6	no.		8	no.
helical dia	=	8	mm		8	mm
Area of main steel, A_{st}	=	1206	mm^2.			mm^2
Bar spacing (face to face)	=	98	mm			mm

REFERENCE | Rev:

EC2 **Shear to EN 1992-1-1:2004 (EC2) - Secant Wall** Circular Sections (Cast In-situ) using helical reinforcement

Pile section

	pile dia	=	350	mm			
	Pile spacing	=	550	mm			
	pile diameter dnom	=	330	mm			
	Ac	=	85530	mm^2			
4.4.1.3(4)	cover c_{nom}	=	40	mm	k_2 =	50 mm	[NA.1 4.4.1.3 (4)]
	main bar dia	=	16	mm			
	no. main bars	=	6	no.			
	helical dia	=	8	mm			
	d	=	229	mm	γ_c = 1.5 (This is adjusted by K_f=1.1 [2.4.2.5 (2)] to give 1.65)		
	f_{ck}	=	28	MPa	γ_c = 1.65	α_{cc} = 0.85	[NA.1 3.1.6 (1)}
	f_{yk}	=	500	MPa	γ_s = 1.15		
	Wallap shear	=	95	kN/m			
		=	52.25	kN	SF factor	1.0	
	Ult V_{Ed}	=	52.25	kN			
	Actions N	=		kN			
	factored actions N_{Ed}	=		kN			

6.2.2 Check requirement for shear reinforcement

$V_{Rd,c}$	=	$[C_{Rd,c}k(100\rho_1 f_{ck})^{1/3}+k_1\sigma_{cp}]b_w d$	CRd,c	=	0.18 / γc	0.11
with minimum	=	$(v_{min}+k_1\sigma_{cp})b_w d$	k	=	$1+(200/d)^{1/2}$	1.93 <=2.0
			ρ_1	=	$A_{sl}/b_w d$	0.01 <=0.02
v_{min}	=	$0.035k^{3/2}f_{ck}^{1/2}$	σ_{cp}	=	N_{ed}/A_c	0 < $0.2f_{cd}$
		0.49809039	k_1	=	0.15	[NA.1 6.2.2(1)]

$V_{Rd,c}$ = **38** kN

Is $V_{Rd,c} > V_{Ed}$ **=>** **NO** Action: ***Design of shear reinforcement required***

6.2.3 Design Shear Reinforcement

Check concrete strut capacity at Cot θ = 2.5 :-

				cot θ	=	2.5	
				tan θ	=	0.4	
6.2.3 (3) exp 6.9	$V_{Rd,max}$	=	$\alpha_{cw}.b_w.z.v_1.f_{cd}$ / (Cotθ+tanθ) (6.9)	α_{cw}	=	1	[NA.1 6.2.3(3)]
				z	=	0.9d	206 mm
	$V_{Rd,max}$	=	**180** **kN**	v_1	=	0.6 (1-(fck/250)	0.53 [6.6N]

Is $V_{Rd,c} > V_{Ed}$ **=>** **YES** Action: ***Calculate link spacing***

Calculation for strut inclination:-

θ = $0.5.\sin^{-1}[(6.54*V_{Ed})/(b_w.d.(1-f_{ck}/250).f_{ck})$

θ = NA rad cot θ = 2.5 > 1.0

Calculate shear reinforcement spacing after Turmo et al (2008);-

$V_{Rd,s}$ = $z.\cot\theta.(A_\Phi/0.5s).f_{ywd}.0.85$ A_{sw} = 50.3 mm^2

s = $2.([z.\cot\theta.A_\Phi.fywd.0.85]/V_{Rd,s})$ f_{ywd} = 435 MPa

= **367** mm

Check maximum shear link spacing:-

is $s_{l,max}$ > 0.75d **YES**

Provide	8	mm helical at nominal pitch	170	mm

Turo, J, *et al*. Shear truss analogy for concrete members of solid and hollow circular cross section. **Eng. Struc.** (2008)

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APPENDIX C

66 FITZJOHN'S AVENUE, LONDON NW3

GEOTECHNICAL REPORT BY DONALDSON ASSOCIATES

Duncan Mercer
Michael Chester & Partners LLP
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EC3A 7JB

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DATE 3 June 2016

REF HISK
PROJECT NO EL426

Dear Duncan,

66 Fitzjohn's Avenue

This report assesses the potential ground movement and building damage, due to construction of a basement at the site.

The site is on Fitzjohn's Avenue, south of Lyndhurst Road, and covers the plot of land behind No.64. There is currently a two storey semi-detached building on the site (with no basement) and this is to be demolished and replaced with a new three storey building with a single storey basement.

A site investigation has been carried out and consisted of one 15m deep cable percussion borehole and two window samples. The ground consists of the Claygate Beds (clayey) over London Clay with the Claygate member extending to about 3m depth. Standpipes installed in September showed the water level at the time to be about 900mm above structural slab level.

The basement will be formed of a propped secant piled wall to form a cutoff so that the water within the excavation of 4.5m can be pumped out.

62/64 Fitzjohn's Avenue is around 3m and 14 Akenside Road is around 10m from the excavation.

Secant wall installation

Very little movement is to be expected when installing a secant wall in clay using modern plant. Limited data has been published in CIRIA C580[1] from prior to the 1990's and is available to provide an initial estimate. This is based on

[1] CIRIA C580 *"Embedded retaining walls – guidance for economic design", London 2003, see figures*

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TRL Reports PR23[2] and R172[3]. Of particular interest is that 4 out of 5 of these piled walls were installed into a London geology sequence of made ground, claygate beds/head (firm clay) or terrace gravels over London clay.

Assuming a wall depth of around 7m, movements based on C580 of 2-3mm vertically and 2-4mm horizontally may be expected at the façades of 62/64 Fitzjohn's Avenue. At 14 Akenside Road up to 2mm vertically and horizontally may be expected at the facade.

Settlement due to basement excavation

Ground movement curves have been published in CIRIA C580[4] based on empirical correlations of case history field measurements. The ground movement curves are shown in the figures. These ground movements have been derived from monitored surface movements due to the excavation in front of bored piles, diaphragm and sheet pile walls wholly embedded in stiff clay. In 16 of 17 case studies walls were installed into a London geology sequence of made ground, claygate beds/head (firm clay) or terrace gravels over London clay and so are relevant to the current site. The ground movements are expressed in terms of percentage of maximum excavation depth, here 4.5m.

62/64 Fitzjohn's Avenue is around 3m from the excavation. Movements based on this of 2-4mm vertically and 4-5mm horizontally may be expected at the facades.

14 Akenside Road is around 10m from the excavation. Movements based on this of up to 2mm vertically and 1-3mm horizontally may be expected at the facades.

Heave due to overburden removal

Settlements calculated by reference to C580 include an element caused by excavation heave. Using an adjusted elasticity method (BSEN 1997:2005 Geotechnical Design Part 1 General Rules Appendix F) and conservatively taking cu=65kPa as the soil strength over the heave bulb. Following the C580 recommendation, Eu=65 x 425kPa, the initial heave at the centre of the base

---

[2] TRL PR23 "*Behaviour during construction of a propped contiguous bored pile wall in stiff clay at Rayleigh Weir*", 1994
[3] TRL R172, *"Ground movements caused by different embedded retaining wall construction techniques"*, 1995
[4] CIRIA C580 *"Embedded retaining walls – guidance for economic design",* London 2003, see figures

of the excavation can be estimated by treating the excavation as a negative load. The base of the excavation will heave around 20mm as overburden is removed. The effect of heave movements on adjacent buildings during construction will be limited by the wall depth, stiffness and propping. In the longer term, slab construction and the re-imposition of building loading will limit heave to negligible levels.

Building damage assessment

An initial assessment of building damage can be made using C580 empirical estimates of ground movement.

BUILDING	v (mm)	h (mm)	Deflection ratio, M (%)	Horizontal strain, εh (%)	DAMAGE CATEGORY
62/64 FA	4-7	6-9	~0	0.05	0/1
14 AR	0-4	1-5	~0	0.05	0/1

Conclusion

Basement construction has the potential to cause ground movements during wall installation, excavation and in the longer term. Longer term ground movements will be limited by wall and basement design.

Ground movements during wall installation and excavation have been empirically derived based on the construction methodology in the BIA and indicated category 0/1 damage.

14 Akenside Road is around 10m from the excavation and at low risk of damage. No further assessment is proposed.

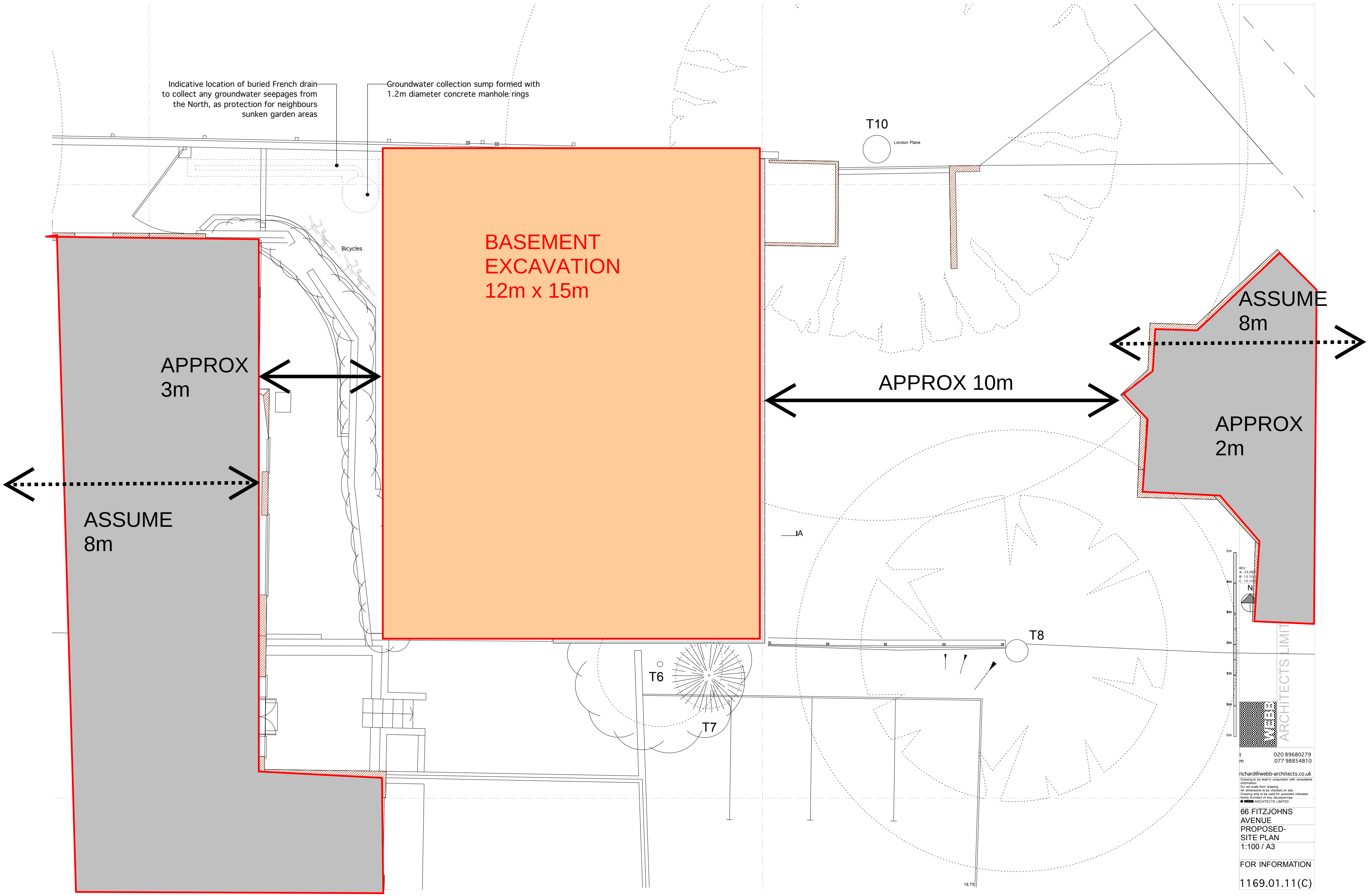
62/64 Fitzjohn's Avenue is around 3m from the excavation and the initial screening suggests a low risk of damage. However, given its proximity to the excavation, it is suggested that the BIA construction methodology used for the assessment is confirmed to still be the case when basement design and sequencing is finalised. It is likely that a condition survey and some façade monitoring will be required.

I hope that this report answers the questions raised by the BIA review.

Yours sincerely,

Hilary Skinner

SITE PLAN


Indicative location of buried French drain to collect any groundwater seepages from the North, as protection for neighbours sunken garden areas
Groundwater collection sump formed with 1.2m diameter concrete manhole rings
Bicycles
T10
London Plane
BASEMENT EXCAVATION 12m x 15m
APPROX 3m
APPROX 10m
ASSUME 8m
ASSUME 8m
APPROX 2m
T6
T7
T8
N
WEBB ARCHITECTS LIMITED
t 020 89680279
m 077 98854810
richard@webb-architects.co.uk
Drawing to be read in conjunction with consultants information.
Do not scale from drawing.
All dimensions to be checked on site.
Drawing only to be used for purposes indicated.
Notify Architect of any discrepencies.
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66 FITZJOHNS AVENUE
PROPOSED- SITE PLAN
1:100 / A3
FOR INFORMATION
1169.01.11(C)

Figures from CIRIA C580 "Embedded retaining walls – guidance for economic design", London 2003

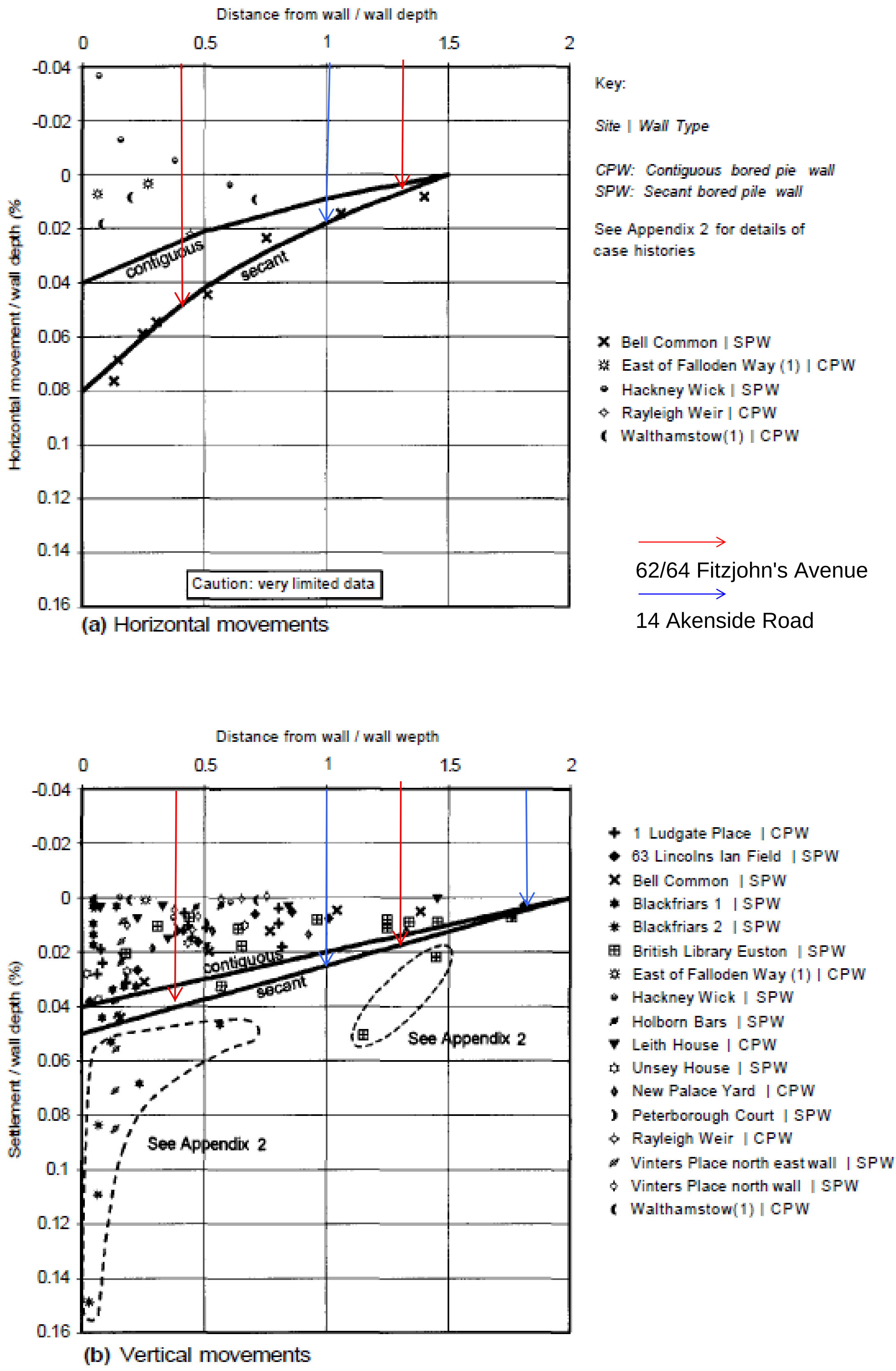


Figure 2.8 *Ground surface movements due to bored pile wall installation in stiff clay*

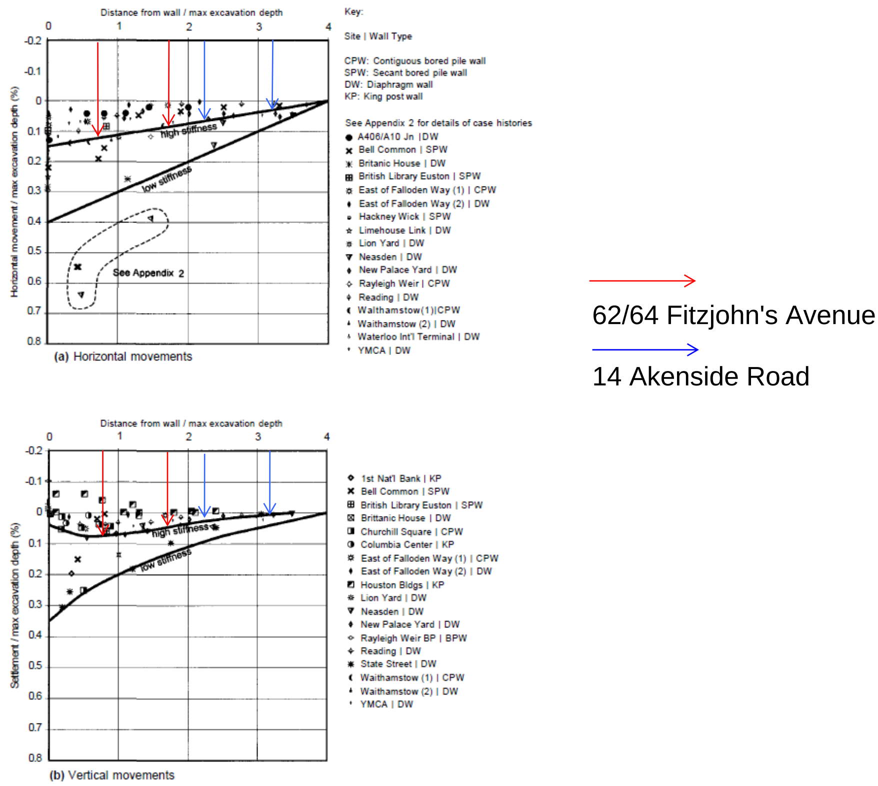
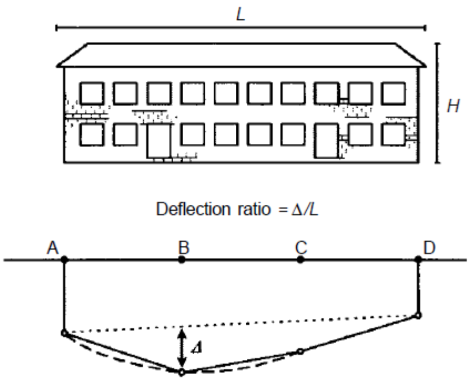
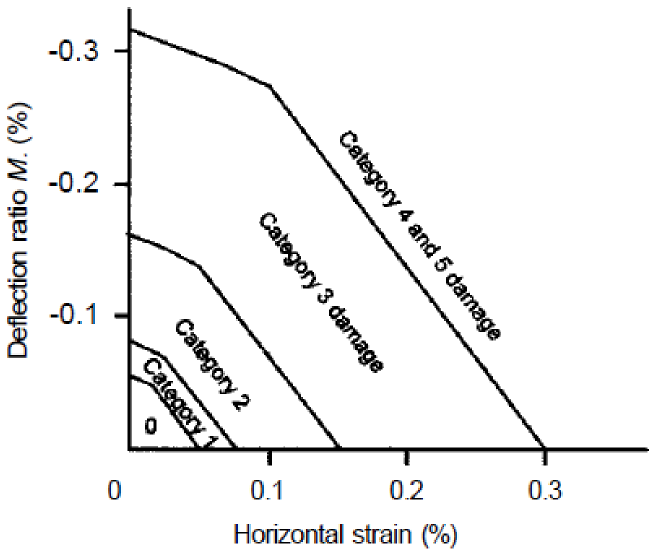


Figure 2.11 *Ground surface movements due to excavation in front of wall in stiff clay*



(a) Definition of deflection ratio.



(c) Relationship between damage category and deflection ratio and horizontal tensile strain for hogging for (L/H) = 1.0 (after Burland, 2001)

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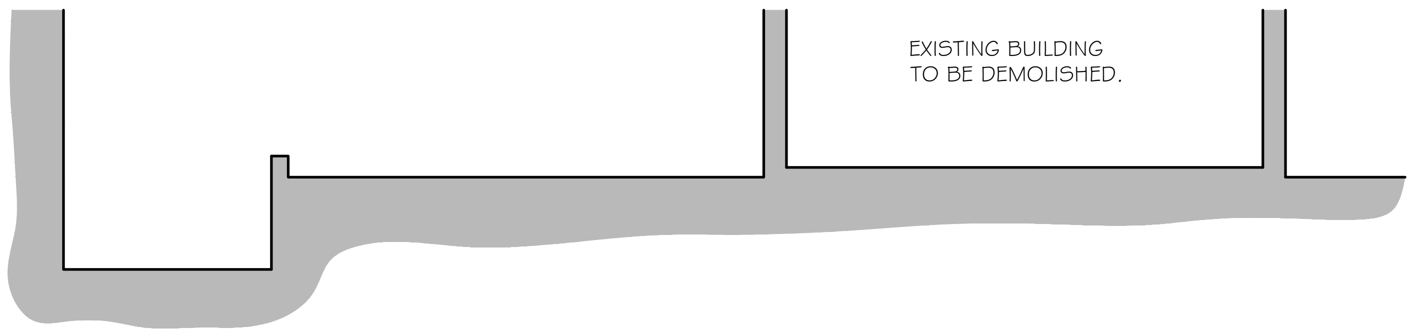
APPENDIX D

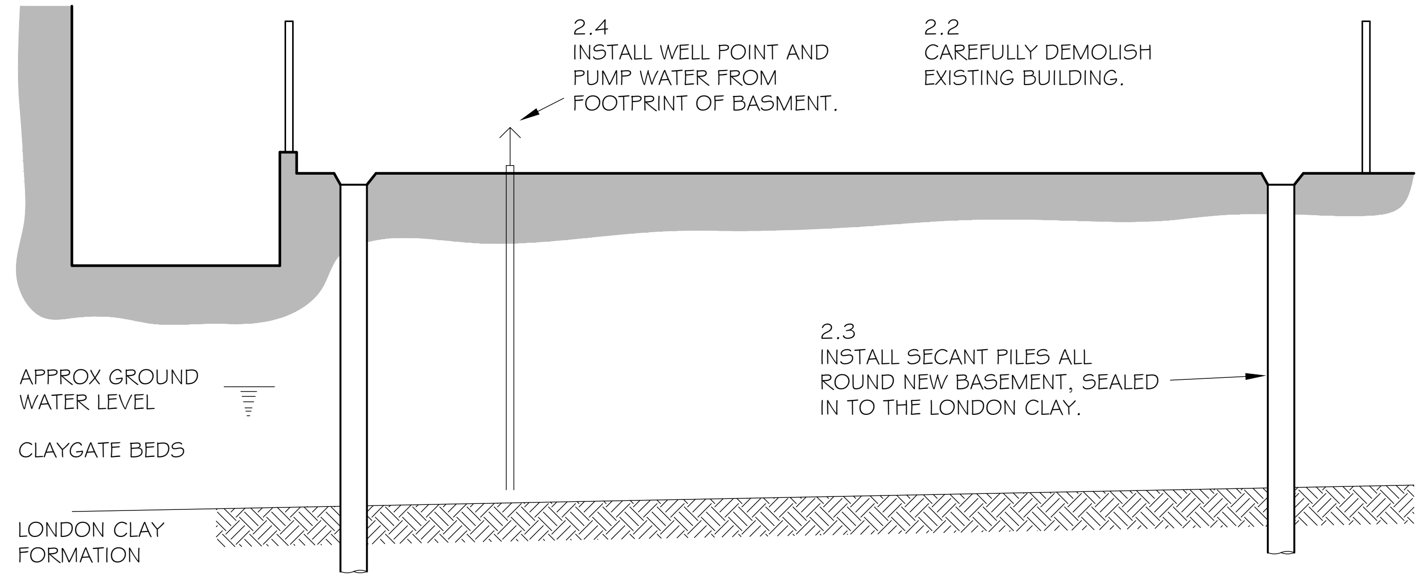
66 FITZJOHN'S AVENUE, LONDON NW3

DRAWING NUMBER 15094/SK02revA BY MICHAEL CHESTER & PARTNERS

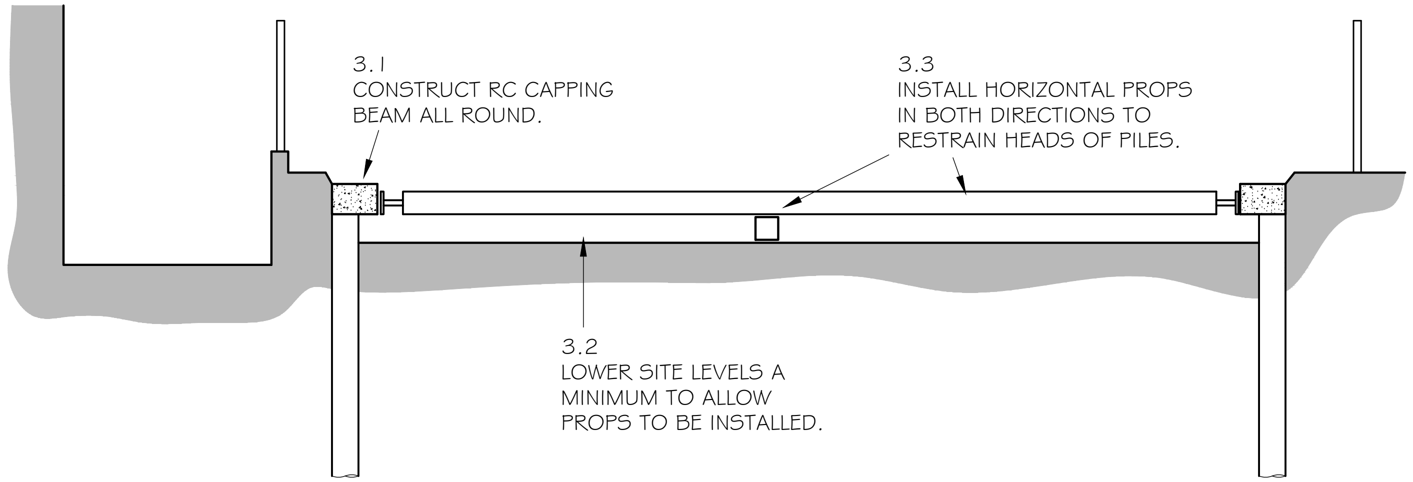
STAGE 1 EXISTING.



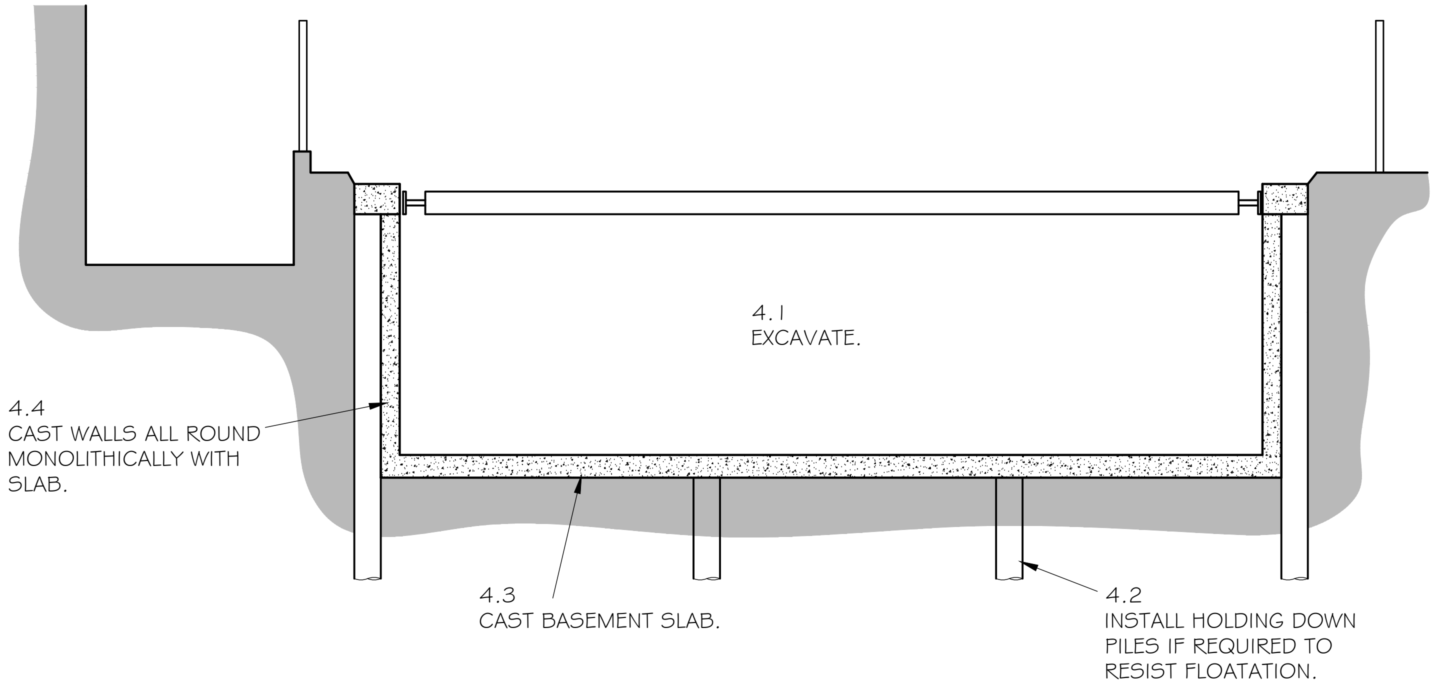
STAGE 2



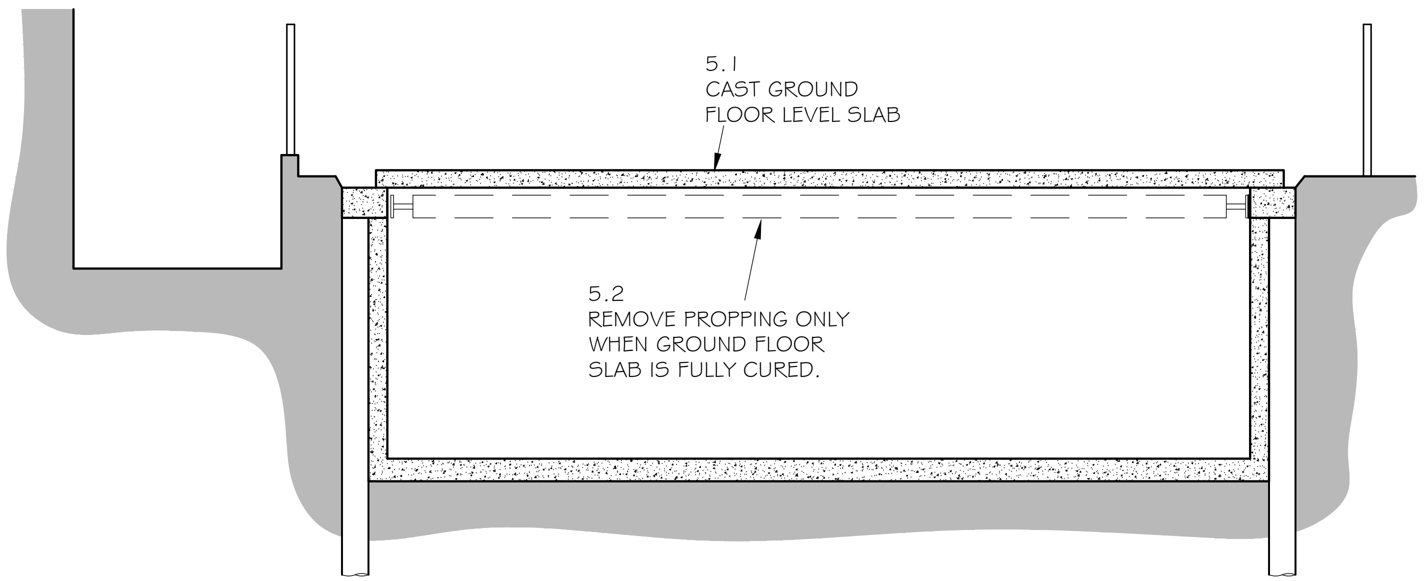
STAGE 3



STAGE 4



STAGE 5



Rev	Date	Alteration
A	SEPT 15	NOTES ADDED.

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Date JULY 15
Drawn DM
Scale nts
Drg No 15094/SK02
Rev A

Project
66 FITZJOHNS AVENUE
LONDON NW3

ASSUMED SEQUENCE OF CONSTRUCTION

PRELIMINARY

A3

Memorandum

To:	Patrick Bonfield	**At:**	Webb Architects
From:	Daniel Watson	**At:**	SLR London
Date:	29th April 2016	**Ref:**	401.05595.00001
Subject:	66 FITZJOHN'S AVENUE BIA - RESPONSE TO AUDIT QUERY 2A		

Audit Query 2a: Further assessment of attenuation requirements for water infiltration to ground to ensure current regime is maintained Audit Para 4.21: *'Development increases the impermeable surface area. An assessment was undertaken in accordance with CIRIA Suds Manual C697 and concluded that there is no material impact from the increased surface area. However, it did state that attenuation could be provided if needed to ensure the existing condition is maintained and detailed drainage design could also include grassed filter strips. Further analyses and design are required to further develop this.'*

SLR Response: There are two drainage receptors for the proposed development. These are:

1) The sewer beneath Fitzjohn's Avenue - Query 2a does not relate to this system. Discharge rates to this feature would be mirrored by the original proposals which only positively drain the roof (unchanged in area) to the sewer. Revised proposals include a green sedum roof on the roof of the building which will significantly reduce total runoff volumes and will also help to slow flows and reduce peak rates of runoff during larger storms.
2) Ground to the south of the basement - Query 2a relates to this system and further possible requirements for attenuation and filter strips are discussed below.

Currently the area where the basement footprint would extend outside the above ground footprint is covered by cobbles and flowerbeds. Such surfaces are permeable and so rainfall falling on this area will currently infiltrate through into the clayey gravel made ground that was observed to be present in BH01 down to a depth of 1m below ground level. Significant deeper infiltration is however likely to be limited by the underlying sandy clay and as such excess flows are currently likely to migrate laterally downslope to the south within the upper layer passing into, and beneath, the adjacent garden which is slightly sunken compared to onsite ground levels. This is the baseline situation and the drainage proposals developed are aimed at maintaining this regime.

Post-development, runoff from the area of hardstanding (and skylights), to the west of the building, would be directed towards the lawn. These flows, and rainfall falling directly on the lawn, would (less any losses resulting from evaporation) infiltrate down towards the underlying basement. Prior to reaching the impermeable roof of the basement, flows would drain due south within a shallow sub base drainage layer to be installed above the basement. The presence of a grassed filter strip at the southern extent of the basement (see Figure 1) and very shallow gradient sloping down to the south along the roof of the basement would help ensure that this water drains southwards and does not pond above the basement. The precise approach will be confirmed at the detailed design stage.

Upon reaching the edge of the basement these flows would passively infiltrate to the ground (i.e. mimicking the existing pre-development regime in that portion of the site). It should be noted that this passive infiltration is distinct and separate from the French drain and sump system proposed to the west of the building to control any exceptional groundwater levels.

The passive infiltration would occur to the south of the basement where it would not impact upon the neighbours' sunken patios (located to the west). The suggestion that attenuation storage might be provided relates to the possible need to store excess water during severe storms prior to it either infiltrating into the deeper sandy clay (at a slow rate) or progressing laterally downslope within the shallower more permeable layer. The requirement for such attenuation storage and its sizing would be dependent upon infiltration rates. Further review and, if necessary, detail design of any necessary features would be carried out after the planning application is granted, when infiltration testing is recommended to confirm potential infiltration rates.

In concept, based on the additional footprint of $89.5m^2$ due to the proposed basement, the design storm considered in the drainage impact assessment (half hour, 1 in 100 annual probability event) would result in a maximum uplift in runoff of 1.84 l/s[1]. Over the duration of this event (half an hour) this would equate to a total volume of storm water of 3.3 m^3 (1.82 x 30 x 60 / 1000). Following the same methodology the 1 in 100 annual probability six hour storm, which is also often considered with respect to drainage design, would generate an estimated total storm volume of 5.6 m^3. In reality, for these events, the total amount of water that would need to be managed would be somewhat less as a proportion of these flows would infiltrate during the storm event.

Based on a permeable area (lawn and paths) of $41.1m^2$, a soil / gravel depth between the ground and the top of the basement of 0.3 m, and an indicative soil / gravel void ratio of 0.3, the total volume of storage available within the soil beneath the lawn is estimated to be 3.7 m^3. It is acknowledged that a proportion of this void may not be free draining; however provided that the sub base layers beneath the lawn are formed by sandy free draining soils the large majority of this volume could reasonably be expected to be available to store and regulate storm flows. The volume of available storage is therefore less than the volume of runoff generated by the 1 in 100 annual probability six hour storm duration event indicating that additional attenuation storage will be required unless infiltration testing demonstrates that flow will readily infiltrate at the southern edge of the basement.

If following infiltration testing the shallow geology is found to have a low permeability, further storage may need to be created to avoid the potential for uncontrolled runoff away from the site to the south. How this is provided would be determined through detailed design, but conceptually could involve;

- construction of the hardstanding area above the north of the basement with permeable material (i.e. open structure brocks or similar) set above gravel. Assuming a hardstanding formation depth of 0.2 m (probably thicker than necessary) the gravel bed would be at least 0.1 m thick. Rainwater falling on the hardstanding would percolate through and would be slowed and stored within the void spaces prior to discharging to the south. Based on a hardstanding area of $48.4m^2$, a 0.2m deep layer of gravel and a void ratio of 0.3, this would provide $1.5m^3$ of additional storage to hold and attenuate flows prior to discharge.

1 This includes a 20% uplift in rainfall depth to allow for potential increases in storm severity associated with climate change. This value is slightly different to that quoted in the BIA due to updates to the development design which have changed the area being considered.

- a grassed filter drain constructed parallel to the southern edge of the western part of the basement. This would provide additional storage required to hold and attenuate flows and would also assist in recharge of groundwater via infiltration. Conceptually, a 0.5m wide, 5m long trench that extends from the surface down to 0.5m below the top of the basement could be created (i.e. 0.9m overall depth). If this was filled with coarse gravel it would provide an additional $0.7m^3$ of storage.

The total possible additional available storage, in combination with the storage inherently provided within and beneath the lawn area, would be $5.9m^3$. This should be sufficient to manage projected volumes of runoff from a major rainfall event ($5.6m^3$ for 1% annual probability 6 hour storm).

A high level overflow from the filter drain to the storm water sewer system beneath Fitzjohn's Avenue could also be included to ensure that uncontrolled surface runoff in this area is prevented during exceedance events (i.e. very extreme in excess of design standard). The system could be designed such that this overflow would not be required under design condition (1 in 100 annual probability event). If under very severe conditions (or other system failure) it was required, this would however not constitute an increase in runoff to the storm water sewer network as the small additional flows from the new contributing areas would be more than offset by reductions in total storm volumes and peak rates of discharge from the main roof area resulting from the incorporation of the green sedum roof.

Figure 1: Sketch plan of site

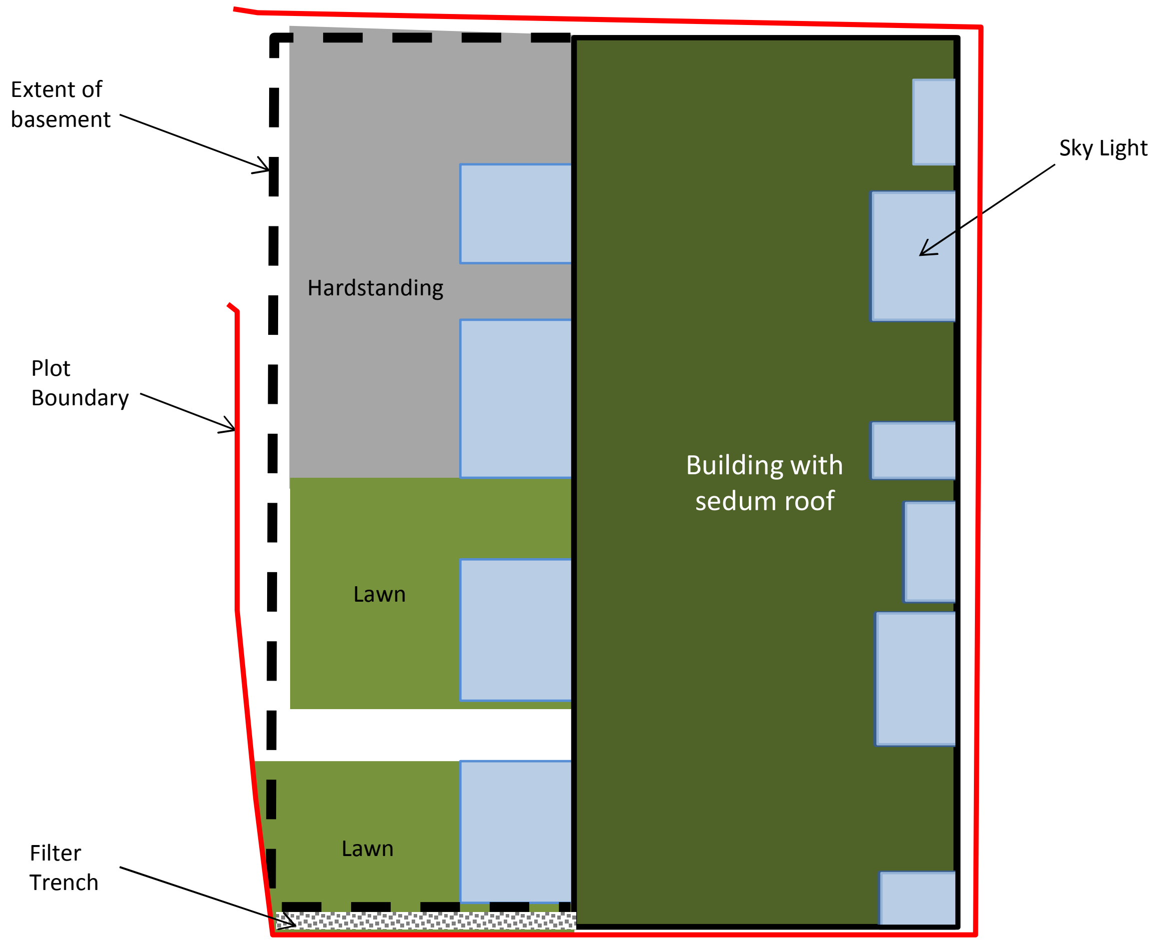
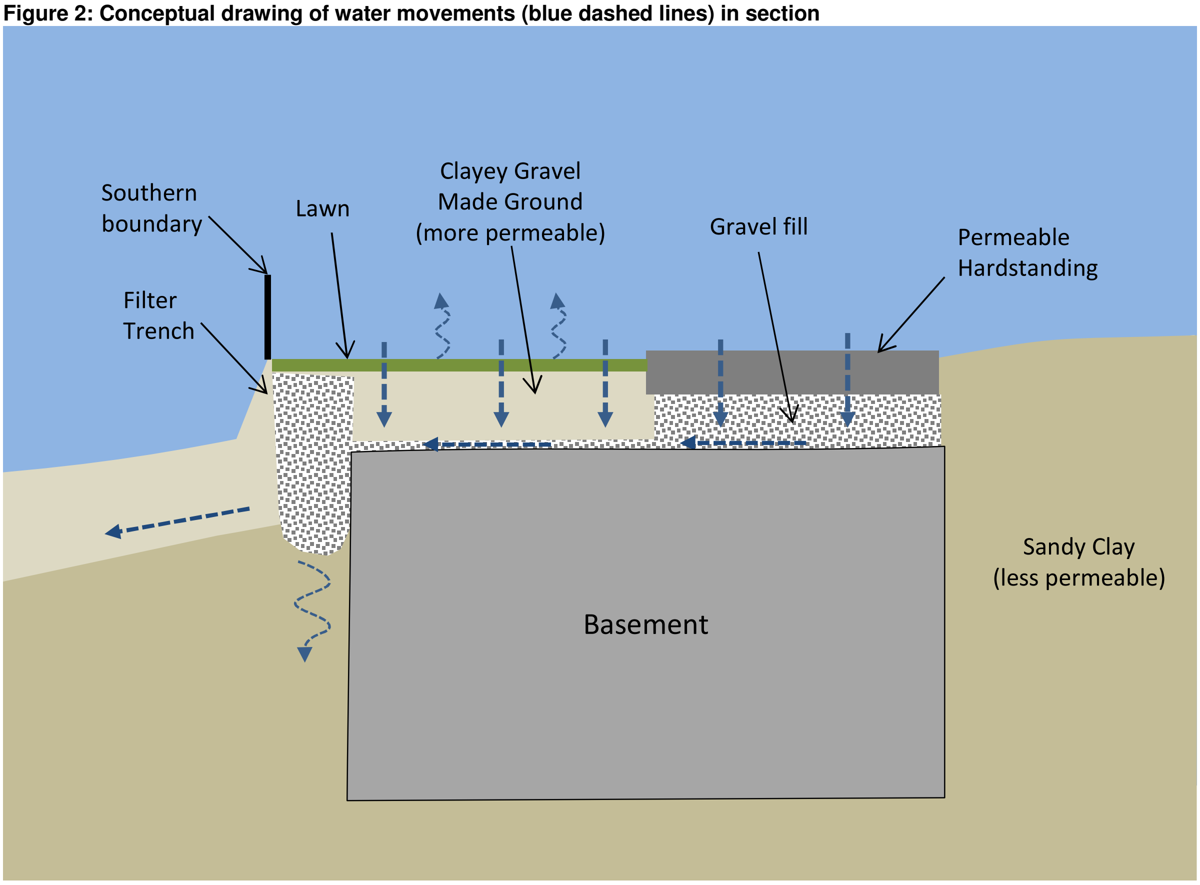


Figure 2: Conceptual drawing of water movements (blue dashed lines) in section



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