230705
August 2023

CRØFT
STRUCTURAL+
CIVIL

Subterranean Construction Method Statement

Site Address:

23A Hampton Road
Teddington
TW11 0JN

Client Address:

Simon Kinsman
23A Hampton Road
Teddington
TW11 0JN

Report by	Jack Hicks BEng MSc
Report Reviewed by	Eleni Pappa BEng MSc
Structural Design Reviewed by	Chris Tomlin MEng CEng MIStructE
Hydrogeology, Soils &Above Ground Drainage Reviewed by	Vijaya Dubagunta M.Tech B.Tech CEng MICE

Rev	Date	Rev By	Comment
-	10/08/2023	JH	First issue
1	19/09/2023	JH	GMA added
2	28/09/2023	JH	GMA final issue

Croft Structural Engineers Ltd
Rear of 60 Saxon Road, London, SE25 5EH
T: 020 8684 4744 E: enquiries@croftse.co.uk

CRØFT STRUCTURAL+ CIVIL

1. Contents

CRØFT STRUCTURAL+ CIVIL

1. Executive Summary

Croft Structural Engineers has reviewed the scope of the proposed basement development at 23A Hampton Road, Teddington.

This Basement Impact Assessment (BIA) has been produced following the London Borough of Richmond *Good Practice Guide on Basement Developments (2015)*.

The key elements of the report are:

- Desk Study
- Inspection of Site and Adjacent Site
- Listed Buildings
- Geology
- Hydrology
- Assessment of Ground Movements
- Anticipated movements are expected to be 0-1 on the Burland Scale.
- Engineering design work completed by a Chartered Structural Engineer
- Initial Flood Risk, Drainage and SuDS completed by a Chartered Civil Engineer
- Construction sequence
- Temporary works
- Structural GAs and sections

Should the proposal receive planning permission and, ultimately, progress to site, the client has been informed that the services of a chartered structural engineer must be retained for the duration of the project.

2. Screening Assessment

2.1. Subterranean Characteristics

Does the recorded water table extend above the base of the proposed subsurface structure?

No.

Is the proposed subsurface development structure within 100m of a watercourse or spring line?

No.

Are infiltration methods proposed as part of the site's drainage strategy?

No.

Does the proposed excavation extend below the local water table level or spring line during the construction phase?

No.

Is the most shallow geological strata at the site London Clay?

Yes.

Is the site underlain by an aquifer and/or permeable geology?

Yes, the site is underlain by the Kempton Park Gravel member.

2.2. Land Stability

Does the site, or neighbouring area, topography include slopes that are greater than 7°?

No.

Will changes to the site's topography result in slopes greater than 7°?

No.

Will the proposed subsurface structure extend significantly deeper underground compared to the foundations of the neighbouring properties?

Yes.

Will the construction of the proposed subsurface structure require the felling or uprooting of any trees?

No.

Has the ground at the site been previously worked?

No.

Is the site within the vicinity of any tunnels or railway lines?

No.

2.3. Flood Risk & Drainage

Will the proposed subsurface development result in a change in impermeable area coverage on the site?

Yes.

Will the proposed subsurface development impact the flow profile of throughflow, surface water or ground water to downstream area?

Yes.

Will the proposed subsurface development increase throughflow or ground water flood risk to neighbouring properties?

No.

3. Desk Study

3.1. Proposed Works

The proposed works are comprised of the demolition of the existing building on the site followed by the construction of a domestic property with three above ground storeys as well as a single-storey basement.

The basement will be constructed using pile walls and reinforced concrete retaining walls with concrete slabs at both basement and ground floor levels and multiple lightwells around the property.

3.2. Site History

Figure 1: Extract from OS map c. 1936

The property does not appear on the OS maps from the 1930s and was likely build in c. 1960s. The site appears to have previously been greenfield.

3.3. Listed Buildings

Figure 2: Extract from Historic England maps of listed buildings

The existing property is not listed.

The closest listed building is Teddington Hall, approximately 50m away.

3.4. Adjacent Properties

Visual inspections of the external façades of the adjacent buildings has been inspected to consider whether the proposed basement will significantly affect their structure.

3.4.1. 23 Hampton Road - Property to the Left

- Property age: c. 19th century
- Property use: Care home
- Number of storeys: 3
- Basement present: Unknown
- Structural defects noted: None.

Figure 3: 23 Hampton Road

3.4.2. Teddington Hall - Property to the Right

- Property age: c. 1863
- Property use: Domestic
- Number of storeys: 4

- Basement present: Yes, lower ground floor as can been seen in Figure 4 below.
- Structural defects noted: Property not visible from road.

Figure 4: Teddington Hall front elevation

3.4.3. 2A Coleshill Road - Property to the Rear

- Property age: c. 2010
- Property use: Domestic
- Number of storeys: 2
- Basement present: No
- Structural defects noted: None

Figure 5: Front of 2A Coleshill Road

3.5. Topography

Figure 6: Topography of Richmond upon Thames

Does the existing site include slopes, natural or manmade, greater than 7° (approximately 1:8)?

No. Site is approximately flat. There are no major falls within 20m which will increase the risk of land slip.

Will the proposed reprofiling of the site change slopes at the property boundary to more than 7° (approximately 1:8)?

No. The proposed landscaping does not affect the slope.

Does the development neighbour land including railway cuttings and the like with a slope greater than 7° (approximately 1:8)?

No. There are no railway cuttings adjacent to the property.

Is the site within a wider hillside setting in which the general slope is greater than 7° (approximately 1:8)?

No. The slope of the wider hillside setting is as per the property, approximately flat.

Is the London Clay the shallowest strata on site?

No. Kempton Park Gravel is the shallowest strata.

Will any tree(s) be felled as part of the proposed development and/or are any of the works proposed within any tree protection zones where trees are to be retained?

Yes. One small tree at the front of the property is to be removed.

Is there a history of seasonal shrink-swell subsidence in the local area and/ or evidence of such effects at the site?

No. Subsidence not considered as an issue on this site.

Is the site within an area of previously worked ground?

No.

3.6. Highways, Rail & London Underground

3.6.1. Highways

Is the site within 5m of a highway or pedestrian footway?

Yes. Site is within 5m of the highway.

Highways loading – allow:

- $10kN/m^2$ if within 45° of road
- 100kN point loads if under road or with in 1.5m

- 5kN/m^2 if within 45° of pavement
- Garden surcharge 2.5kN/m^2
- Surcharge for adjacent property 1.5kN/m^2 + 4kN/m^2 for concrete ground bearing slab

3.6.2. London Underground & Network Rail

Is the site over (or within the exclusion zone) of any tunnels, e.g. railway lines?

No. The site is approximately 400m from the nearest railway line.

3.6.3. UK Power Networks

Will the basement works affect any UK Power Network Assets (substations etc)?

No. No UK Power Networks assets were noted during the initial site visit. A utilities search has not been conducted.

3.7. Trees

While there are no trees within the bounds of the property, there are some in the immediately surrounding area.

- Laurel, approx. 8m tall, approx. 5m away from closest point of proposed basement
- Sycamore, approx. 10m tall, approx. 11m away from closest point of proposed basement
- Beech, approx. 10m tall, approx. 7m away from closest point of proposed basement
- Scots Pine, approx. 10m tall, approx. 3.5m away from closest point of proposed basement

Are any trees to be removed to make way for the proposed basement?

No. All existing trees are to remain.

3.7.1. Special Precautions due to Trees

The increased depth of the foundations necessary for the basement places the new foundations outside the effects of trees. The building will be more stable with the proposed basement.

3.8. Geology & Ground Investigation

A site specific ground investigation has not been completed for this planning application. However, the British Geological Survey maps show what ground conditions to expect and two previously undertaken boreholes in the vicinity of the site confirm what is shown on the maps.

In summary:

- Groundwater is anticipated to be at 4.25m below ground level
- Gravel is expected at formation level and heave potential is considered low
- An allowable ground bearing capacity of 100kN/m^2 is advised

3.8.1. British Geological Survey Data

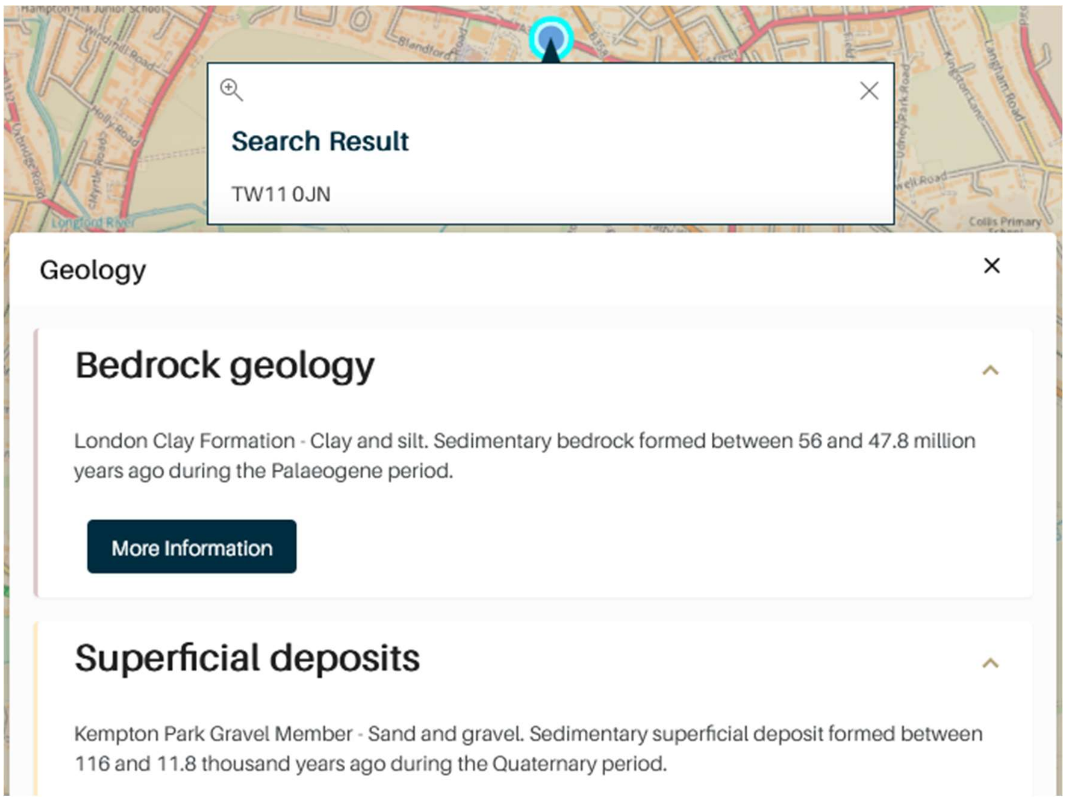


Figure 7: Extract from BGS maps

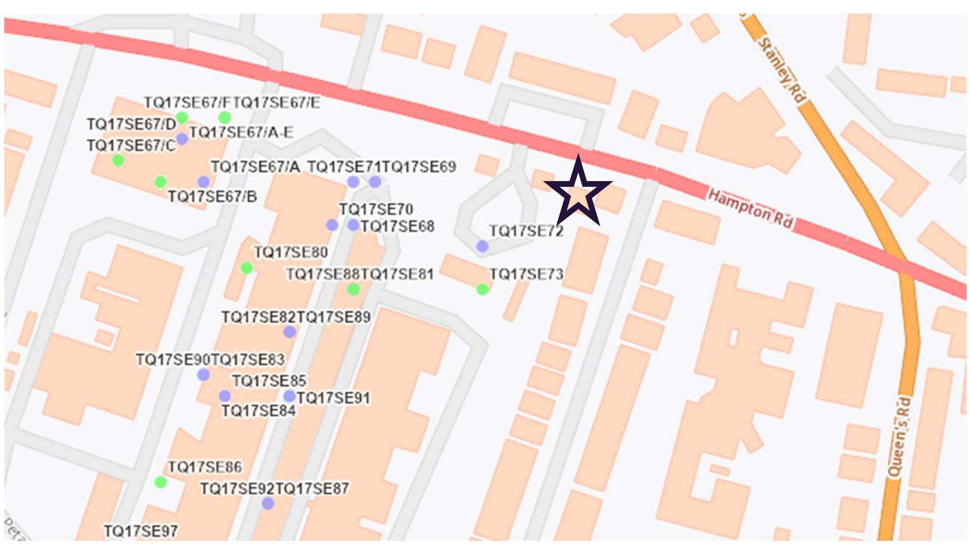
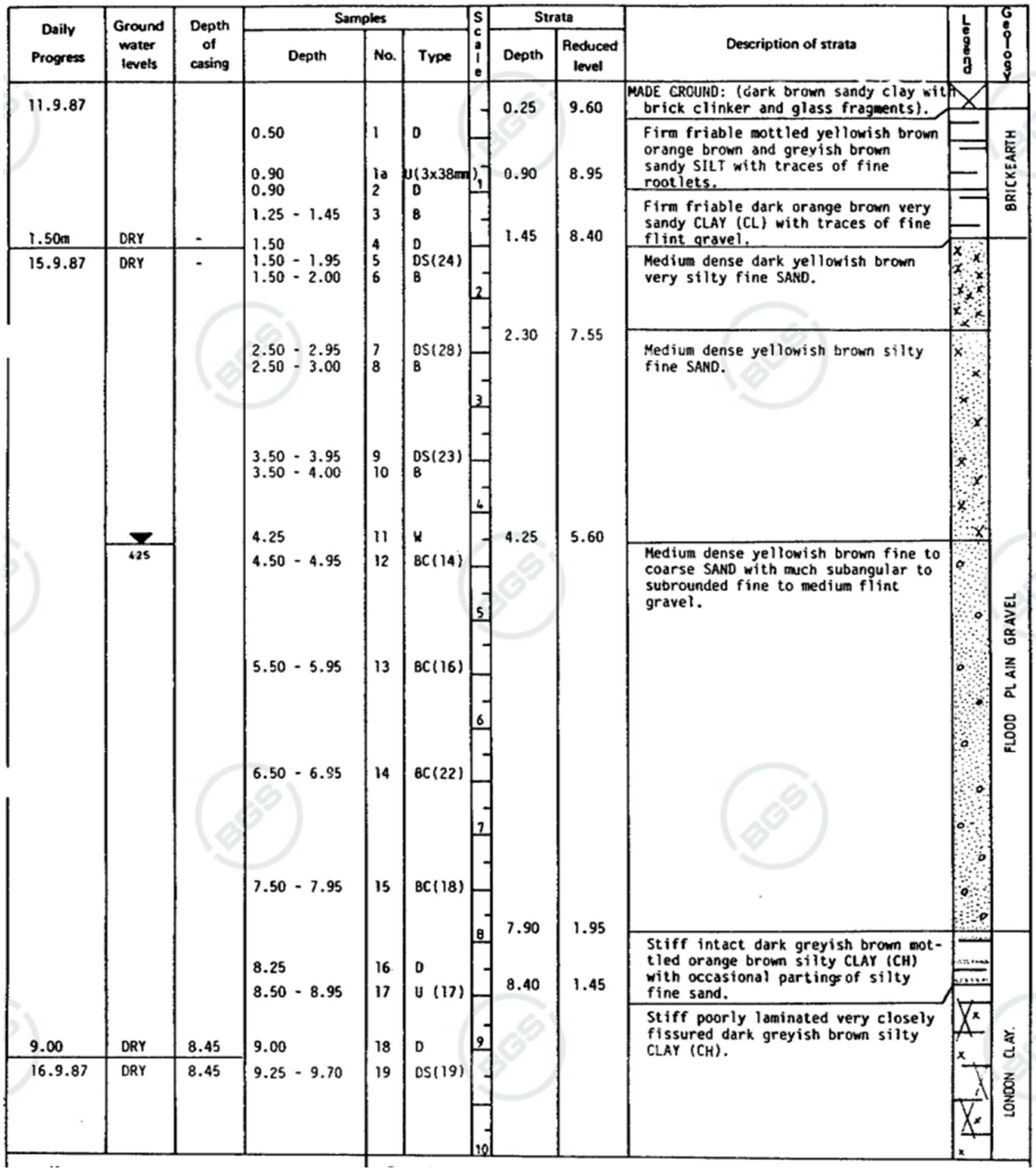


Figure 8: BGS map showing locations of two boreholes in vicinity of the site

Job No. 11311416 Ground level 9.89m O.D. Diameter / 150mm Casing / 150mm to 7.95m

Daily Progress	Ground water levels	Depth of casing	Samples: Depth	Samples: No.	Samples: Type	Scale	Strata: Depth	Strata: Reduced level	Description of strata	Legend	Geology
11.9.87							0.50	9.39	MADE GROUND:(Firm brown sandy clay with roots, fine to medium flint gravel and brick fragments).		
			0.60	1	D				Firm intact dark yellowish brown very sandy silty CLAY (CL).		BRICKEARTH
			0.70	1a	U(3x38mm)						
			1.00	2	D	1	0.90	8.99			
									Firm friable yellowish brown very sandy silty CLAY (CL) with traces of fine flint gravel.		
			1.30 - 1.45	3	B						
1.50	DRY	-	1.50	4	D		1.45	8.44			
15.9.87	DRY	-	1.50 - 1.95	5	U (16)				Stiff thickly laminated yellowish brown very sandy CLAY (CL) with many partings of fine sand.		
			2.00	6	D	2					
			2.25 - 2.70	7	DS(20)						
			2.25 - 2.75	8	B		2.50	7.39			
									Medium dense light yellowish brown silty fine to medium SAND.		FLOOD PLAIN GRAVEL
			3.00 - 3.45	9	DS(25)	3					
			3.50	10	B						
			4.00 - 4.45	11	DS(28)	4					
			4.00 - 4.50	12	B						
	4.70		4.70	13	W		4.70	5.19			
									Medium dense yellowish brown fine to coarse SAND with some subangular to subrounded fine to coarse flint gravel. KP		
			5.00 - 5.45	14	BC(18)	5					
			6.00 - 6.45	15	BC(27)	6					
			7.00 - 7.45	16	BC(28)	7					
			7.90	17	D		7.80	2.09			
			8.00 - 8.45	18	U (15)	8	8.00	1.89	Stiff intact dark greyish brown mottled dark brown silty CLAY (CH) with occasional pockets of fine sand.		LONDON CLAY
			8.50	19	D				Stiff very closely fissured dark greyish brown silty CLAY (CH) with occasional pockets of dark grey sandy clay.		
			8.75	20	D						
			9.00 - 9.45	21	DS(18)	9					
9.45	DRY	7.95					9.45	0.44	BOREHOLE COMPLETED.		
						10					

Figure 9: Borehole in immediate vicinity of site (TQ17SE72), c. 1987



Daily Progress	Ground water levels	Depth of casing	Samples: Depth	Samples: No.	Samples: Type	Scale	Strata: Depth	Strata: Reduced level	Description of strata	Legend	Geology
11.9.87							0.25	9.60	MADE GROUND: (dark brown sandy clay with brick clinker and glass fragments).		
			0.50	1	D				Firm friable mottled yellowish brown orange brown and greyish brown sandy SILT with traces of fine rootlets.		BRICKEARTH
			0.90	1a	U(3x38mm)	1	0.90	8.95			
			0.90	2	D				Firm friable dark orange brown very sandy CLAY (CL) with traces of fine flint gravel.		
			1.25 - 1.45	3	B						
1.50m	DRY	-	1.50	4	D		1.45	8.40			
15.9.87	DRY	-	1.50 - 1.95	5	DS(24)				Medium dense dark yellowish brown very silty fine SAND.		
			1.50 - 2.00	6	B	2					
							2.30	7.55	Medium dense yellowish brown silty fine SAND.		
			2.50 - 2.95	7	DS(28)						
			2.50 - 3.00	8	B	3					
			3.50 - 3.95	9	DS(23)						
			3.50 - 4.00	10	B	4					
	425		4.25	11	W		4.25	5.60	Medium dense yellowish brown fine to coarse SAND with much subangular to subrounded fine to medium flint gravel.		FLOOD PLAIN GRAVEL
			4.50 - 4.95	12	BC(14)	5					
			5.50 - 5.95	13	BC(16)	6					
			6.50 - 6.95	14	BC(22)	7					
			7.50 - 7.95	15	BC(18)	8					
							7.90	1.95	Stiff intact dark greyish brown mottled orange brown silty CLAY (CH) with occasional partings of silty fine sand.		
			8.25	16	D						
			8.50 - 8.95	17	U (17)		8.40	1.45	Stiff poorly laminated very closely fissured dark greyish brown silty CLAY (CH).		LONDON CLAY
9.00	DRY	8.45	9.00	18	D	9					
16.9.87	DRY	8.45	9.25 - 9.70	19	DS(19)	10					

Figure 10: Borehole in immediate vicinity of site (TQ17SE73), c. 1987

3.8.2. Ground Considerations

The basement will be founded in sand. Croft has completed several basements in this type of ground. The basement can be completed with a pile wall.

3.8.3. Bearing Stress

In line with CP111, assumed bearing design stress = 100 kN/m^2.

3.8.4. Ground Stability

Design overall stability to K_a & K_p values. Lateral movement necessary to achieve K_a mobilisation is height/500 (from Tomlinson). This is tighter than the deflection limits of the concrete wall.

The slope stability of gravels is in the region of 30˚. The design of the pile walls will take this into account.

3.9. Flood Risk

3.9.1. Fluvial Flooding

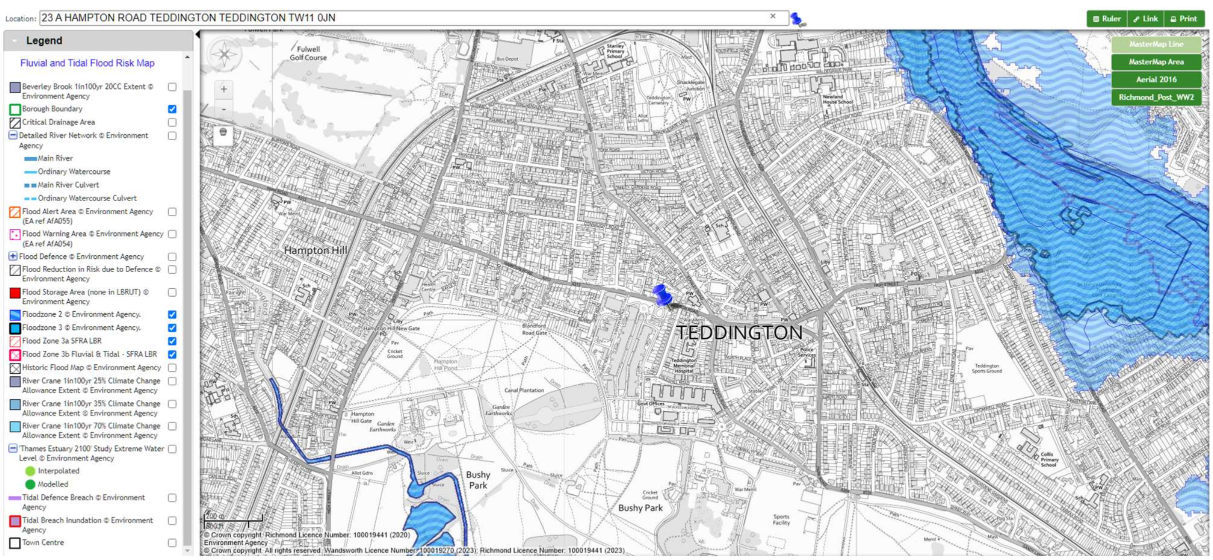


Figure 11: Extract from Richmond Strategic Flood Risk Assessment - Fluvial and Tidal Flood Risk Map

Is the site in a fluvial or tidal flood risk zone?

No.

3.9.2. Surface Water Flooding

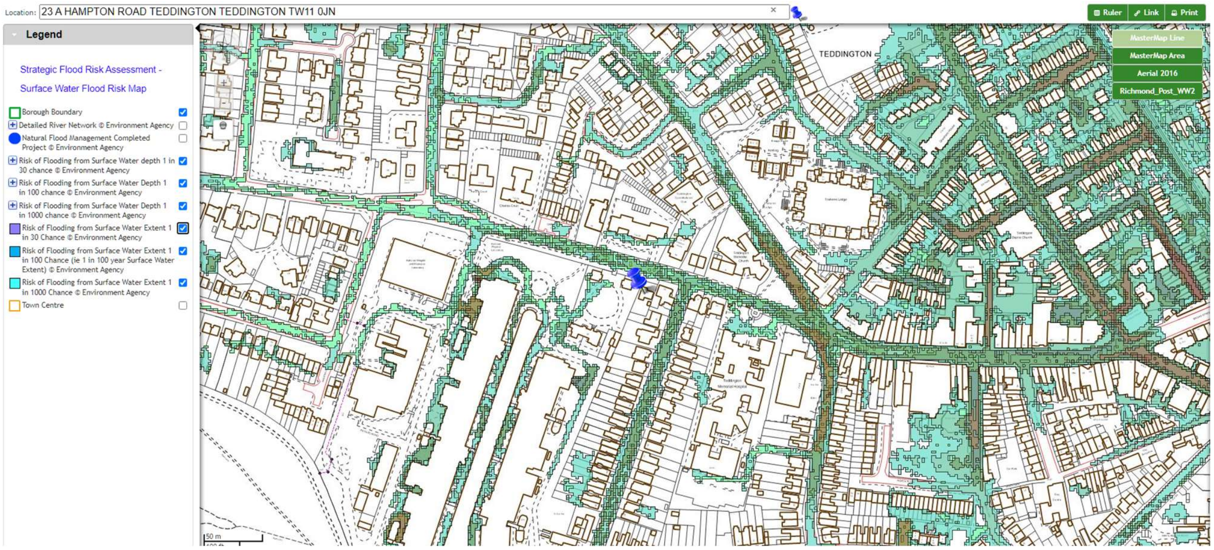


Figure 12: Extract from Richmond Strategic Flood Risk Assessment - Surface Water Floor Risk Map

Is the site in a surface water flood risk zone?

No.

3.9.3. Ground Water & Sewer Flooding

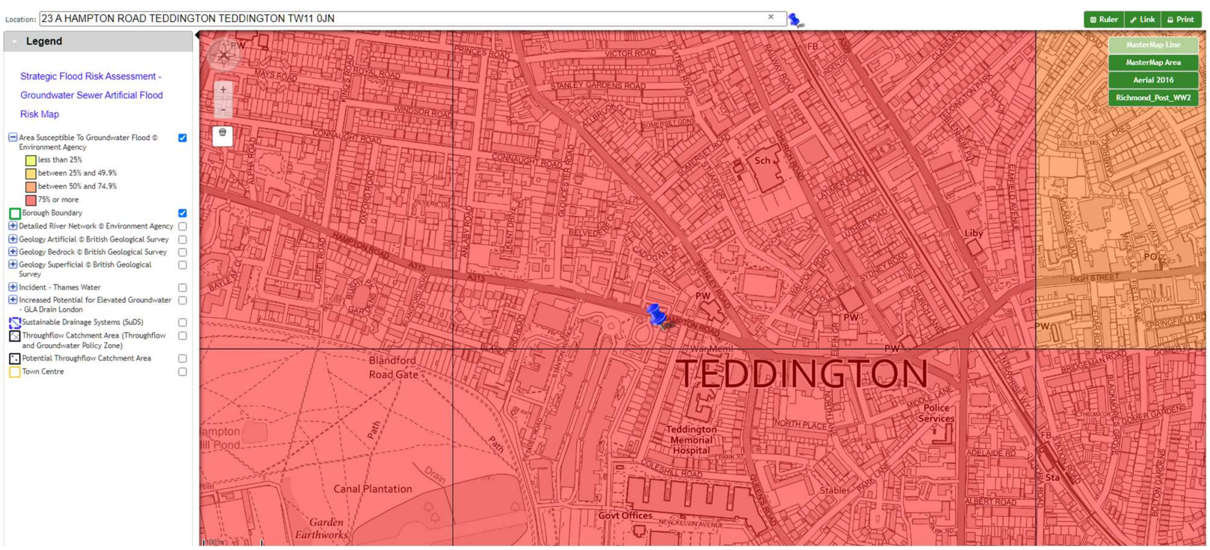


Figure 13: Extract from Richmond Strategic Flood Risk Assessment - Ground Water Sewer Artificial Flood Risk Map

Is the site at risk of flooding due to ground water or sewers?

Yes. However, there have been fewer than 10 incidents reported by Thames Water in this area.

3.9.4. Flood Risk Desk Study Summary

The site is located in flood zone 1. There is no evidence of a risk of flooding from fluvial, tidal nor surface water. The is in an area at risk of ground water or sewer flooding. However, there have been fewer than 10 incidents reported by Thames Water in this area.

A site-specific flood risk assessment is not required.

3.10. Ground Water, Surface Water & Drainage

The basement will be founded on sand and gravels and will not act as a dam. There will be capacity for the water to be displaced around and under the property.

If clay is encountered at depth, a 150mm thick layer of compacted type I should be provided to prevent damming.

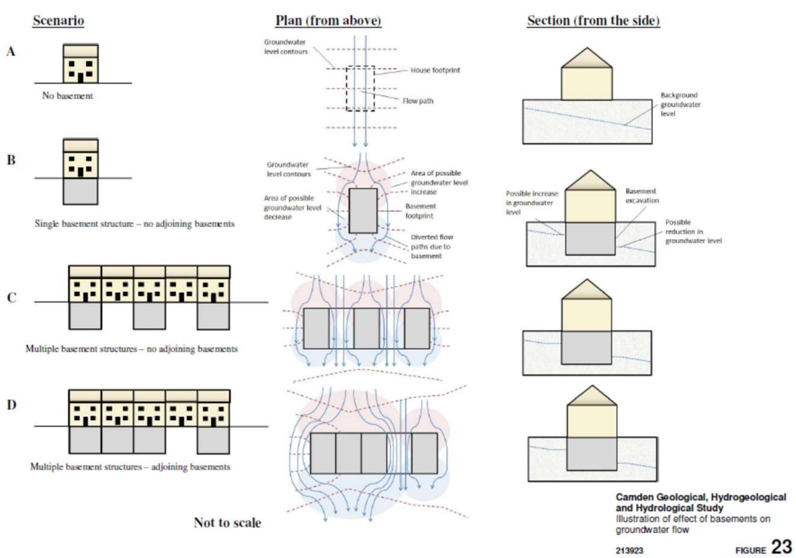


Figure 14: Extract from Arup report on ground water flow

The reinforced concrete retaining walls have been designed to withstand ground water flooding.

As part of the proposed site drainage, will surface water flows be materially changed from the existing route?

No.

Will the proposed basement development result in a change to the impermeable area of the site?

Yes. The impermeable area will increase from ~113m^2 to ~180m^2.

Will the proposed basement result in changes to the instantaneous and long-term surface water being received by the adjacent properties or downstream water courses?

No.

Will the proposed basement result in changes to the quality of the surface water being received by adjacent properties or downstream water courses?

No.

As part of the site drainage, will more surface water be discharged to the ground than currently?

No.

3.11. Localised Drainage & Damp-proofing

Concrete is not designed BS 8007. However, where possible, BS 8007 detailing should be observed to help limit crack widths of concrete.

All waterproofing must be made by the waterproofing specialist. They should review the structural engineer's details.

A waterproofing specialist should be appointed to ensure all the waterproofing requirements are met. The structural waterproofer must inspect the structural details and confirm that they are happy with the robustness.

4. Ground Movement Assessment & Predicted Damage Category

See full Ground Movement Assessment report in Appendix A.

4.1. Mitigation Measures

The existing building on the site is to be demolished. This means that the basement can be constructed from piles forming a box. With this method, the primary structure is in place before the excavation commences. This significantly reduces the risk of movement of neighbouring properties.

A method statement for the construction of the basement is appended. The procedures described in this have been formulated with Croft's experience of over 500 basements completed without error. The measures described in this statement will mitigate the impacts that the construction of the basement may have on nearby properties. Croft has been involved in a number of basement designs of a similar scale to the proposed development at 23A Hampton Road. These previous projects have been followed through to the construction phase and have involved the use of regular movement monitoring before, during and after the basement works are complete.

To reduce the risk of damage associated with the development, the following measures are advised:

- Employ a reputable contractor that has extensive knowledge of basement works.
- Employ suitably qualified consultants.
- Provide method statements for the contractors to follow.
- Investigate the ground.
- Record and monitor the properties close-by. This is usually completed by a condition survey, under the Party Wall Act, before and after the works are completed. Refer to the end of the appended Basement Construction Method Statement.

With the measures listed above, the maximum level of cracking anticipated is 0-1 cracking. This can be repaired with normal decorative works. At detailed design stage, the Party Wall Application and the appointment of Party Wall Surveyors will ensure that the above measures are applied. Under the Party Wall Act, minor damage, although unwanted, can be tolerated; it is permitted to occur to a neighbouring property as long as repairs are suitability undertaken to rectify this. To mitigate this risk, the Party Wall Act is to be followed and a Party Wall Surveyor will be appointed.

Temporary works are described further in the following section and a proposed construction sequence for the works is appended.

5. Engineering Considerations

The existing building is to be demolished, allowing for a piled solution rather than sectional underpins. Pile walls will form the perimeter of the basement. Reinforced concrete retaining walls will then be constructed within the piled box. Together, these will resist lateral forces and also transfer the loads from the superstructure to the ground, forming a new foundation to the property.

The design proposals in this report are intended to demonstrate feasibility to support the planning application. The information, drawings, calculations, method statement and other information in this report are for planning purposes. Croft provide no design warranty or insurances for the final design. Further information and design considerations must be undertaken before Building Regulations submission. <u>The information provided in this document is not for construction.</u>

See Appendix B for initial calculations of retaining wall designs.

5.1. Surcharge Loading

The following loads should be accounted for:

Garden surcharge 2.5kN/m^2

Surcharge for adjacent property 1.5kN/m^2 + 4kN/m^2 for concrete ground bearing slab

Loading from pavements and highways (see below)

Is the site within 5m of a highway or pedestrian footway?

Yes. Site is within 5m of a highway.

Highways loading allow:

- 10kN/m^2 if within 45° of road
- 100kN point loads if under road or with in 1.5m
- 5kN/m^2 if within 45° of pavement

6. Temporary Works

A proposed construction method statement is appended.

7. Noise, Vibration & Dust

Full investigations and reports (such as ground investigations and construction traffic and management plans) should be carried out ahead of building works to formalise the best practical means to be used.

Best practice construction methods should be chosen to reduce unnecessary noise, vibration and dust. The following table is a guidance to minimise the effect of the same.

CONSTRUCTION METHOD	**MITIGATION MEASURES**	**NOISE**	**DUST**	**VIBRATION**
In accordance with the best practical means, to be used **To minimize, noise, vibration and dust during the construction of the basement, including the excavation, that is likely to affect adjacent residential premises and school(if any)**				

CONSTRUCTION METHOD	MITIGATION MEASURES	NOISE	DUST	VIBRATION
1. Preparation of site to fully contain the area	Boarding to front of house enclosing entrance, and windows kept in place for complete duration of construction	Boarding keeps noise inside the house and keeps house more rigid stopping attenuation, absorbs sound and Stops airborne sound escaping	Dust from debris stored internally is contained within boarded up house preventing it from escaping to neighbours before collection.	Any internal vibration is further reduced by additional boarding to absorb before emitting to neighbour: as timber absorbs vibration better than metal or glass. The house is also more rigid, stopping vibration
	Windows retained and sealed shut during construction, including front door and terrace doors kept closed	Airborne noise is contained within development	Airborne dust is contained within the development	Windows being sealed shut (taped) stops any rattling of windows or accentuation of any vibrations on site
	Hording and sheeting to cover roof terrace.	Covering with hording and sheeting restricts airborne noise from escaping as best can be.	Sheeting to roof terrace stops window blowing up dust from excavation and any dust generated from works escaping to vicinity.	Hording and sheeting stops vibration as best is practicable.

CONSTRUCTION METHOD	MITIGATION MEASURES	NOISE	DUST	VIBRATION
	Retention of internal floors and structure during excavation works	Keeping the internal floors in situ during works allows the house to work as a buffer to contain noise and reduces the site area to the smallest volume reducing the effect noise can have.	Dust is contained to a smaller area and has several filters (ie floors and walls) to pass through and thus get stopped before it can affect neighbours, thus reduced.	Retaining the existing structure reduces vibration by keeping the house rigid and secondly by having a mix of materials all with different attenuation frequencies; vibration is absorbed and not accentuated, lastly floors and walls act as a break in otherwise continuous structure which acts as a buffer to stop vibration continuing out to neighbours.
	Temporary works and structure	Temporary works allow the house to be kept rigid and allow for small scale, less noise emitting methods of construction to be used.	Temporary works keep the house rigid and safe so stop other areas of the house degenerating through works and thus dust being created.	Temporary works keep the house rigid which stops vibrations.
2. Management and hours of working	Project manager to manage all works on site, member of Considerate Contractors Scheme	Hours of working are restricted and staff supervised to use tools appropriately. No radio on site. Small team working reducing noise.	Hours of working are restricted and staff supervised to use tools appropriately with appropriate guarding to prevent dust migration.	Hours of working are restricted and staff supervised to use tools appropriately and reduced use of power tools to minimize vibration.

CONSTRUCTION METHOD	MITIGATION MEASURES	NOISE	DUST	VIBRATION
		Coordination between workers ensured.		
3. Excavation of basement	Non-percussive tools used for excavation (ie hand dug)	Hand tools are quieter. Method chosen reduces need for any heavy noisy machinery	Less dust generated by hand tools than fast repetitive motor driven tools.	Vibration is minimized by not using percussive tools
	Excavation limited to 1m runs and shuttered for reinforced concrete foundations.	Each underpin is restricted to 1m lengths containing noise and amount of work that can be done at once to small area thus reducing overall hubbub. Method is quieter than piling or machine methods.	Dust is contained within shuttering, area is dampened with water to allow digging and eliminate dust.	Shuttering contains any subsequent vibration from excavation and keeping surrounding area soil intact.
	Removal of spoil	All spoil is hand bagged and stored internally by hand so no noise from skip or large refuse area, removed as per CTMP by small van and hand loaded	Spoil hand bagged, not using electric conveyor belt, and reducing emission of dust.	Spoil bagged by hand (ie shovel) so no machinery to transmit vibration

CONSTRUCTION METHOD	MITIGATION MEASURES	NOISE	DUST	VIBRATION
	Removal of debris	Bagged debris is stored internally in a covered area and removed by waiting small van as per CTMP timed to cause least disruption	Debris removed by hand; dust contained within refuse sack, sealed shut.	Debris removed by hand, vibration minimized, in bags.
	Mixing and pouring of concrete for underpins	Concrete is mixed on site for small quantities for underpin, contained within the site for noise and for short period of time once underpin and shuttering formed (ie Separate activity)	Area set aside and shuttered off for mixing concrete to contain dust. Only small quantities mixed at time. Only small amounts of dry concrete Stored on site in internal area to avoid unnecessary dust.	Concrete mixer put on level base in clear working area to avoid vibration.
	Delivery of concrete for floor reinforced floor slabs	Large quantities are not mixed on site but delivered and pumped by specialist lorry to site in speedy low noise method from front of house through hording	No dust emitted from delivery of liquid concrete, area of road washed down before and after delivery. Area cordoned off as per CTMP (approx. ½ hour).	Large quantities of concrete mixed off site to reduce continuous vibration and delivered to site.

Appendix A – Ground Movement Assessment

23A Hampton Road, London

Preliminary Ground Movement Assessment

September 2023

Croft Structural Engineers Ltd

Clock Shop Mews

Rear of 60 Saxon Road

London

SE25 5EH

Final Report

Report No. 100896

Document Verification

Prepared for	Prepared by
Croft Structural Engineers Ltd Clock Shop Mews Rear of 60 Saxon Road London SE25 5EH	Ground and Project Consultants Ltd, Ground Floor 5 Ambassador Place Stockport Road Altrincham WA158DB

Signatures and Approvals				
Author	S Chan-Fitzpatrick		Date	28/09/2023
Checker and Approver	J Smithson		Date	28/09/2023

Report Number	Revision	Date	Comments
100896	0	18/09/2023	Draft
100896	0	28/09/2023	Final

Disclaimer

This report has been prepared by Ground and Project Consultants Ltd, with reasonable skill, care and diligence within the agreed scope and terms of contract and taking account of the manpower and resources devoted to it by agreement with its client, and is provided by Ground and Project Consultants Ltd solely for the use of its client, Croft Structural Engineers.

The advice and opinions in this report should be read and relied on only in the context of the report as a whole, taking account of the terms of reference agreed with the client. The findings are based on the information made available to Ground and Project Consultants Ltd at the date of the report (and will have been assumed to be correct) and on current UK standards, codes, technology and practices as at that time. They do not purport to include any manner of legal advice or opinion. New information or changes in conditions and regulatory requirements may occur in future, which will change the conclusions presented here.

This report is confidential to the client. Unless otherwise agreed in writing by Ground and Project Consultants Ltd, no other party may use, make use of or rely on the contents of the report. No liability is accepted by Ground and Project Consultants Ltd for any use of this report, other than for the purposes for which it was originally prepared and provided.

Executive Summary

Site Location	The site is located at 23a Hampton Road, Teddington, London, TW11 0JN.
Coordinates	TQ 15366 71033
Proposals	The proposals include the demolition of the existing bungalow and the construction of a new-build three-storey dwelling, with the lower ground floor founded at around 3.5m bgl.
Scope of Services	Ground Movement Assessment only.
Site Description	The site is currently a bungalow. The site is bound by No.23 Hampton Road to the East and No. 25 Hampton Road to the West. The site is on the south side of Hampton Road.
Anticipated Ground Conditions	The anticipated ground conditions at site are thin Made Ground overlying Kempton Park Gravels overlying London Clay Formation.
Ground Movement Assessment	The results of the ground movement and building damage assessment have found the maximum potential risk to surrounding properties from the basement construction is in the Category 2 Slight damage.

Table of Contents

Tables and Figures

Appendices

1 Introduction

Ground and Project Consultants Ltd (GPCL) has been instructed by Croft Structural Engineers to undertake a Ground Movement Assessment for No.23a Hampton Road, Teddington, London.

The proposals for the site comprise the demolition of an existing bungalow and construction of a new-build three-storey house with a basement founded at a depth of approximately 3.0m bgl.

The scope of this report is as follows:

- A review of the existing data supplied by the Client:
  - Subterranean Construction Method Statement, Croft (Ref 230705 date August 2023)
  - Proposed Plan Drawing, Croft (Ref: 230705 SL-10)
  - Merged Drawing file
- Summarise the geology and hydrogeology
- Undertake a Ground Movement and Building Damage Assessment.

2 Site Information

The information on the site and surrounding area has been obtained from freely available sources included in the references in Section 5. Where appropriate, figures and tables have been provided throughout the report for ease of assessment.

2.1 Site Location

The site is located to the rear of No. 23a Hampton Road, Teddington, London, TW11 0JN. The site is in the London Borough of Richmond. The national grid reference for the site is TQ 15366 71033. The site is approximately 300m northeast of Bushy Park and immediately northeast of the National Physical Laboratory. Hampton Road forms the A313. The location of the site is located on Figure 1 below.

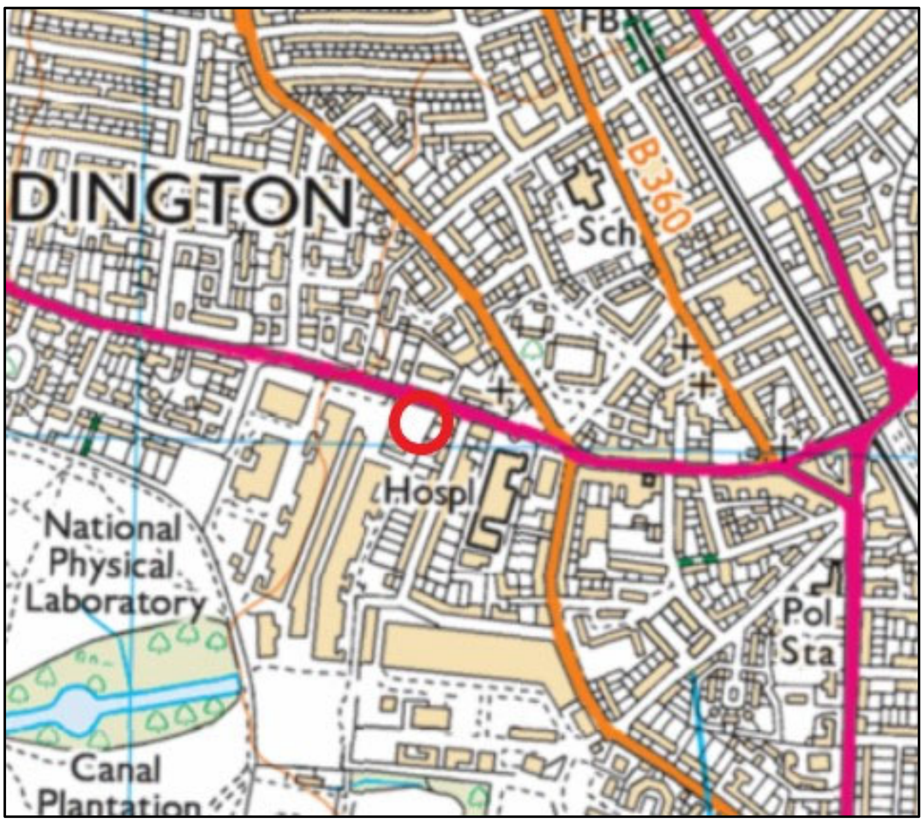


Figure 1: Site Location and Topography (Ordnance Survey, copyright 2023)

2.2 Site Description and Topography

The site is at an elevation of approximately 9 m AOD. The general location is on flat ground falling slightly towards the River Thames to the east. The site is currently occupied by a bungalow which is to be demolished. Access to the site is via Hampton Road to the north.

The site is bound by No.25 Hampton Road to the west, No.23 Hampton Road to the east with Hampton Road itself to the north.

No underground railways are anticipated beneath the site.

Several trees are present on and nearby the site.

2.3 Proposals

The proposals for the site comprise the demolition of the existing bungalow and the development of a new-build three above ground storey dwelling with a basement. The basement level will be founded at around 3.0m bgl. The basement will be constructed using pile walls and reinforced concrete retaining walls with concrete slabs at both basement and ground floor levels and multiple lightwells around the property.

2.4 Geology

The geology of the site is indicated on BGS Sheet 270 (South London) and the BGS Viewer. An extract of the geological map is included below.

The geology on site is indicated to comprise Kempton Park Gravels (terrace deposits) overlying the London Clay Formation. The Taplow Gravels are mapped a few hundred metres to the west. London Clay is shown to outcrop about 200m to the Southwest (i.e., no drift). Isolated patches of Made Ground are mapped within 500m of the site to the north, east and west.

The London Clay Formation is described by the BGS Lexicon as “bioturbated or poorly laminated, blue-grey or grey-brown, slightly calcareous, silty to very silty clay, clayey silt and sometimes silt, with some layers of sandy clay”.

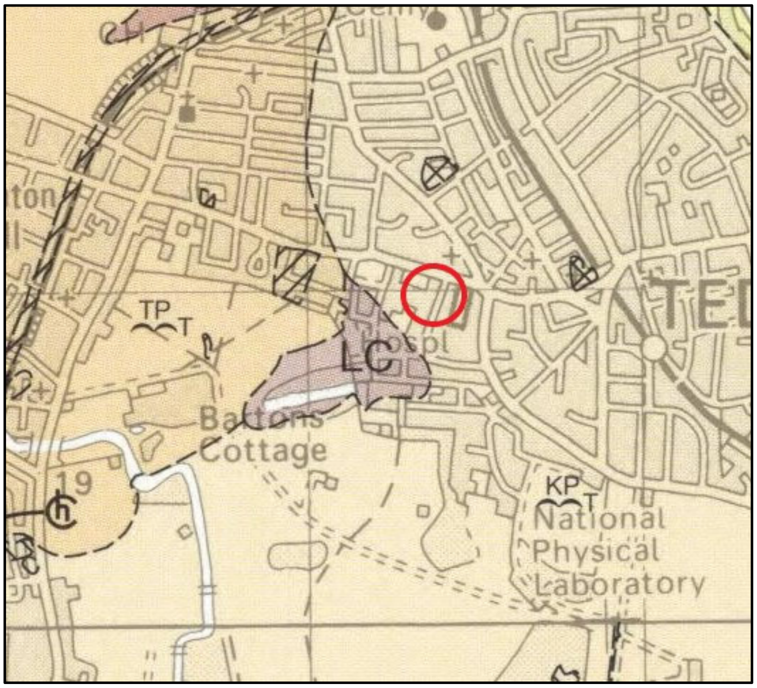


Figure 2: Geology (BGS South London Sheet 270, BGS Copyright 2023)

There are a large number of BGS recorded boreholes close to the site associated with the National Physical Laboratory. These are indicated on Figure 3 below.

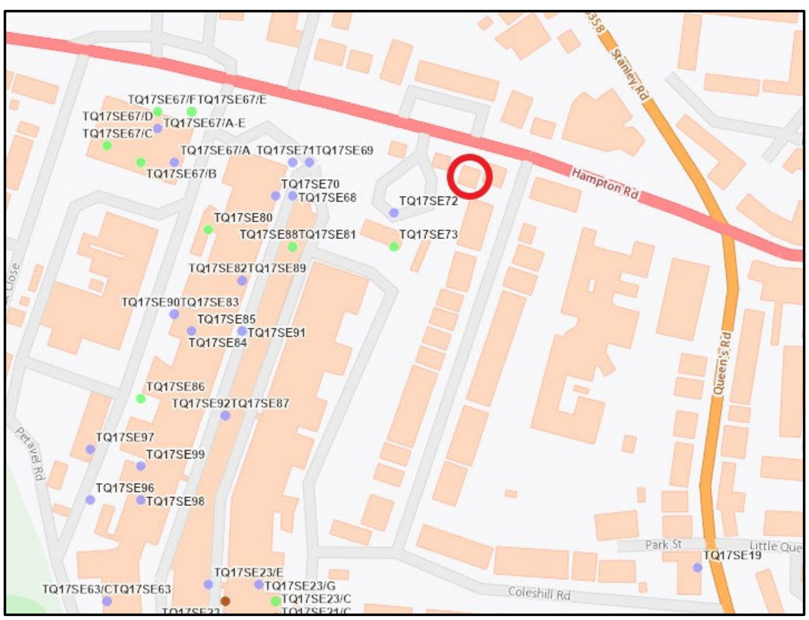


Figure 2: Geology (BGS South London Sheet 270, BGS Copyright 2023)

Two boreholes are relatively close to the site, about 50m and 55m to the South-West. The boreholes were drilled in 1987.

These boreholes indicate the following .

BGS BH Ref		TQ17SE72		TQ17SE73	
Strata	Made Ground	Firm sandy clay with roots and gravel of brick and flint.	GL to 0.50m	Dark brown sandy clay with brick clinker and ash fragments	GL to 0.25m
	Terrace Deposit (Cohesive)	Firm to stiff yellowish brown very sandy silty CLAY, sand partings	0.5 to 4.70m	Firm yellowish and orange-brown sandy SILT or CLAY, occasional gravel	0.25 to 1.45m
	Terrace Deposit (Granular)	Medium Dense silty fine to medium SAND, some gravel	4.70 to 7.80m	Medium dense yellowish-brown SAND, becoming less silty and coarser with gravel at depth.	1.45 to 7.90m
	London Clay	Stiff fissured dark grey-brown CLAY with sand pockets	7.80m to end of BH (9.45m)	Stiff poorly laminated fissured dark grey-brown CLAY with silt/sand partings	7.90 to end of BH (12.0m)
Groundwater		Struck at 4.7m no rise		Struck at 4.25m no rise	

2.5 Hydrology and Hydrogeology

The London Clay Formation is designated as an unproductive aquifer. The superficial deposits (Kempton Park Gravel) are designated as being a Principal aquifer.

The government flood risk data indicates the site is in a low risk area for surface water flooding and a very low risk area from river and sea sources. The site is within a low risk from groundwater sources.

3 Groundwater Screening and Scoping

The purpose of this screening stage is to identify any matters of concern via key aspects relating to groundwater (as per Camden's CPG4 report) and the scoping stage identifies the potential impacts of these. A screening and scoping exercise has been carried out as follows:

Table 1: Groundwater Screening and Scoping Summary

Impact Question	Answer and Justification (Screening)	Impact and Action (Scoping)
Question 1a: Is the site located directly above an aquifer?	Yes. Kempton Park Gravel is a Principal Aquifer.	Refer to Section 4
Question 1b: Will the proposed basement extend beneath the water table surface?	Possibly.. The groundwater was struck at 4.25m (i.e. below basement level) with no rise. However, this data is not site-specific and groundwater levels vary seasonally.	Refer to Section 4.
Question 2: Is the site within 100m of a watercourse, well (used/disused) or potential spring line?	No.	None.
Question 3: Will the proposed basement development result in a change in the proportion of hard surface/paved areas?	Yes. Some increase in hard cover.	Refer to Section 4.
Question 4: As part of the drainage, will more surface water than at present be discharged to the ground (e.g. via soakaways)?	Possibly. Soakaways may be viable.	Refer to Section 4.
Question 5: Is the lowest point of the proposed excavation close to or lower than the mean water level in any local pond or spring line?	No.	None.
Question 6: As part of the proposed site drainage will surface water flows (e.g. volume of rainfall and peak run-off) be materially changed from the existing route?	Drainage is to be introduced for the new property however, this will redirect to mains sewers and is not anticipated to impact hydrogeology.	None.

4 Basement Impact Assessment: Hydrogeology

The identified areas of potential impact from the screening and scoping assessment with respect to hydrogeology/groundwater are discussed below.

4.1 Principal Aquifer

The proposed development is to be constructed to a depth of 3.0m bgl and will be founded within the Kempton Park Gravel (KPG), which is classed as a major aquifer. It is not known whether the adjacent houses have basements or cellars. There may be some impact on groundwater flow, although gaps will remain between the buildings which will allow water flow. The nearby boreholes suggest that the KPG is quite thick here so it may be that the basement will not penetrate the gravels, although the secant pile wall will. However, the secant wall will have gaps allowing some flow.

4.2 Hardstanding

The proposed development will marginally increase the hardstanding area. This will lead to some increase in runoff and drainage requirements. Conversely the recharge of the aquifer will be marginally decreased.

4.3 Drainage

The roof area of the building will be slightly larger and therefore drainage requirements may increase. Based on the data available soakaways may be viable.

4.4 Further Ground Investigation

The assessment above is based on available data (BGS boreholes some distance from the site). Given ground and groundwater conditions vary laterally the data provides only indicative data and a site-specific ground investigation is strongly recommended.

5 Ground Movement Assessment

An assessment of ground movement has been assessed for the property at 23 Hampton Road. The drawings used in the assessment for determining basement dimensions and distances to nearby properties are included in Appendix A. It is understood the basement will be constructed beneath the entire proposed footprint to a depth of approximately 3.0m bgl. The existing building on the site is understood to be demolished and then the piled walls will be installed prior to excavation.

It is recognised that settlements are generally small where care and appropriate measures are taken in this type of basement construction.

It is recommended that where the understanding of movements is significant, appropriate instrumentation should be installed to monitor ground movement before and during construction.

The following key assumptions have been made:

- The detailed design of the basement (and associated temporary works) has been carried out by an appropriately qualified and experiences structural engineer, to current professional standards and best practice.
- A uniform excavation depth of 3.0m below existing ground level has been taken for the basement and lightwells.
- No site-specific ground investigation data is available. It has been assumed that the base of the basement's excavation will be within the cohesive Kempton Park Gravel. The piled wall installation has been assumed to be embedded within the London Clay Formation. This has been taken from BGS boreholes over 50m to the south.
- The basement has been assumed to be constructed using a secant piled wall technique and will be carried out with due skill by an appropriately experiences contractor.
- The depth of the piled walls has not been provided. It has been assumed in our analysis that the pile walls are installed to a depth of 10m bgl.
- A high stiffness wall has been assumed.
- The wall will be propped promptly using closely spaced props in the temporary case.
- In the permanent case, the wall will be permanently propped at basement floor level and ceiling level.

- The assessment assumes that neighbouring buildings are in good condition, with no pre-existing damage.
- It is assumed that the neighbouring properties do not have basements.

For the purposes of the calculations, the parameters of the subject properties have been estimated as included in the table below. The analysis considers the walls of the most pertinent building which is No.23 Hampton Road. The plan location of each individual wall is also included in the figure below.

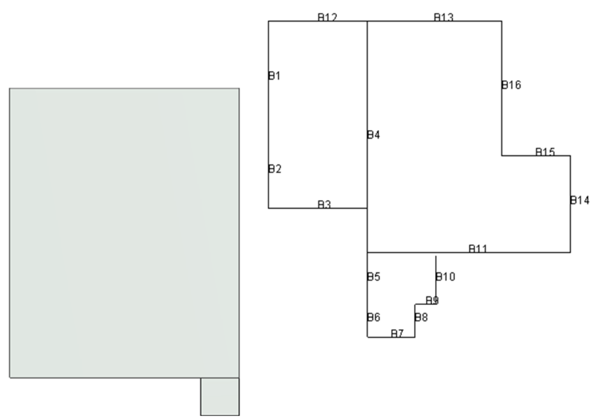


Figure 3: Wall Location Plan

Table 2: Approximate Dimensions of Walls at 23 Hampton Road

Wall No.	Wall Height (m)	Wall Length (m)
B1	7.5	6
B2	2.8	3.7
B3	2.8	5.2
B4	11.5	12.2
B5	3	2.5
B6	2.8	1.7
B7	2.8	2.5
B8	2.8	1.7
B9	3	1.1
B10	3	2.5
B11	11.5	10.6
B12	7.5	5.2
B13	11.5	7
B14	11.5	5
B15	11.5	3.6
B16	11.5	7

5.1 Movement due to wall installation and excavation following C760

The following ground movements have been calculated for the wall installation and excavation using XDISP and methodology outlined in CIRIA C760.

Empirical ground movement curves from CIRIA C760 have been used to assess the impact of the basement construction. The basement excavation has been modelled using the C760 curve "Excavation in front of high stiffness wall in stiff clay". The piled wall has been modelled using the C760 curve "Installation of a secant bored pile wall in stiff clay".

The Burland methodology has been adopted to assess the category of damage for the neighbouring structures. Burland Scale categories 0, 1, and 2 refer to aesthetic damage, category 3 and 4 relate to serviceability and function, and 5 represents damage which relates to stability. The main objective of design and construction is to maintain a level of risk to buildings no higher than category 2 where only aesthetic damage is considered acceptable.

The results of the assessment are present in the table below.

Table 3: Damage Assessment results using the Burland Scale

Wall No.	Maximum Vertical Deflection Δ (mm)	Maximum Horizontal Movement dh (mm)	Building Damage Assessment
B1	7	15	Category 0 (Negligible)
B2	7	15	Category 0 (Negligible)
B3	7	11	Category 2 (Slight)
B4	5	7	Category 0 (Negligible)
B5	5	7	Category 0 (Negligible)
B6	6	7	Category 0 (Negligible)
B7	6	7	Category 2 (Slight)
B8	4	4	Category 0 (Negligible)
B9	4	4	Category 2 (Slight)
B10	3	3	Category 0 (Negligible)
B11	5	6	Category 2 (Slight)
B12	4	4	Category 0 (Negligible)
B13	2	4	Category 0 (Negligible)
B14	1	<1	Category 0 (Negligible)
B15	2	1	Category 0 (Negligible)
B16	2	1	Category 0 (Negligible)

The results of the building damage assessment indicate a maximum of Category 2 "Slight Damage" to Walls B3, B7, B9 and B11.

Given the results of the building damage assessment at Wall B9 located at approximately 9.0m from the proposed basement is Category 2 (Slight Damage), it is predicted that the walls of the gatehouse building at 25 Hampton Road perpendicular to the basement will also likely be Category 2 (Slight Damage).

Note that the figures above do not necessarily represent the total ground movement but the maximum differential movements which are predicted to be experienced by the building. The ground movement and building damage calculations are appended.

There are a number of key points to note in using this assessment:

- Most ground movement will occur during excavation of the basement and construction so the adequacy of temporary support will be critical in limiting ground movements.
- The existing building will be demolished and the basement walls will be constructed from piles forming a box, prior to the excavation. This will significantly reduce the risk of movement to neighbouring properties.
- The speed of propping and support is key to limiting ground movements and limiting unpropped wall heights.
- Good workmanship will contribute to minimising ground movements.

Ground movement can be minimised by adopting a number of measures, including:

- Ensuring that adequate propping and support is in place at all times during construction.
- Installation of the first stiff support quickly and early in the construction sequence.
- Avoid leaving ground unsupported.
- Minimise deterioration of the unexcavated soil mass by the use of blinding/covering with a waterproof membrane.
- Avoid overbreak.
- If dewatering is required, the control and appropriate design of the process must ensure that fines removal and drawdown are minimised.

It must be noted that the movements are calculated values based on the findings and methods of CIRIA C760. Larger movements may be generated if anyone or any combination of the above recommendations and/or assumptions are not heeded or if ground conditions are different from those anticipated by the investigation.

The actual magnitude of these movements will depend upon a number of factors described above and the nature of the ground expected may give rise to larger movements.

6 Conclusions and Recommendations

The results of the ground movement and building damage assessment indicate a maximum damage category of Category 2 “Slight” to walls of No.23 Hampton Road. As the walls with Category 2 “Slight” damage were up to 9m away at No.23 Hampton Road, it is predicted that the walls of the gatehouse at No.25 Hampton Road may also suffer Category 2 Damage. The gatehouse structure at No.25 was not modelled during the analysis.

The results of the ground movement assessment are based on assumptions of the ground conditions from geological mapping and historical off-site boreholes. It is strongly recommended that a ground investigation is undertaken to confirm the ground conditions at the site. Should the ground conditions be found to differ significantly, then the ground movement assessment should be revised to reflect the site-specific ground conditions.

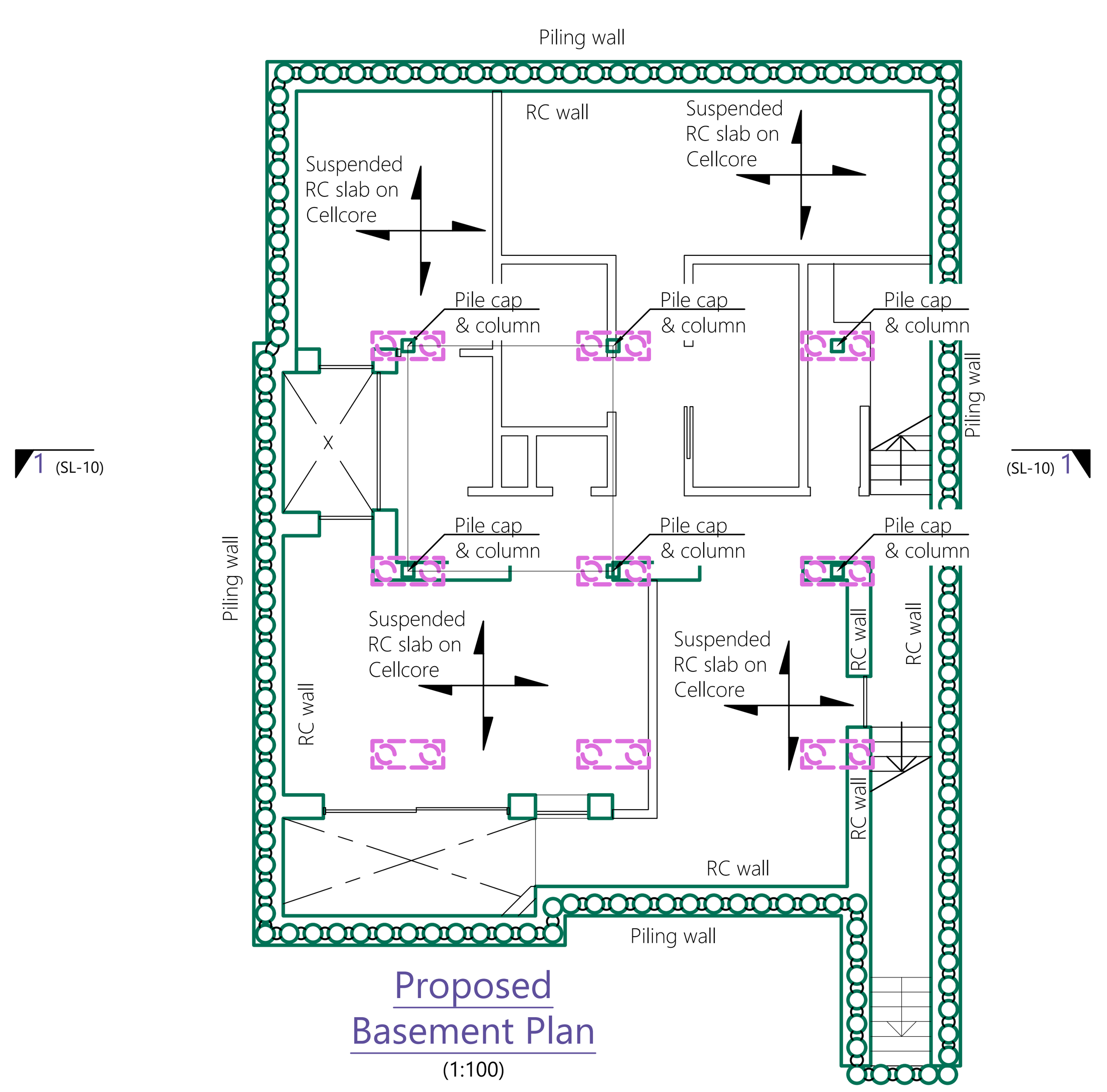
The depth of embedment of the piled retaining walls has been assumed to be 10m bgl. Should the depth of the walls change during detailed design, then it will be necessary to undertake a revised ground movement assessment to reflect the change in pile length.

7 References

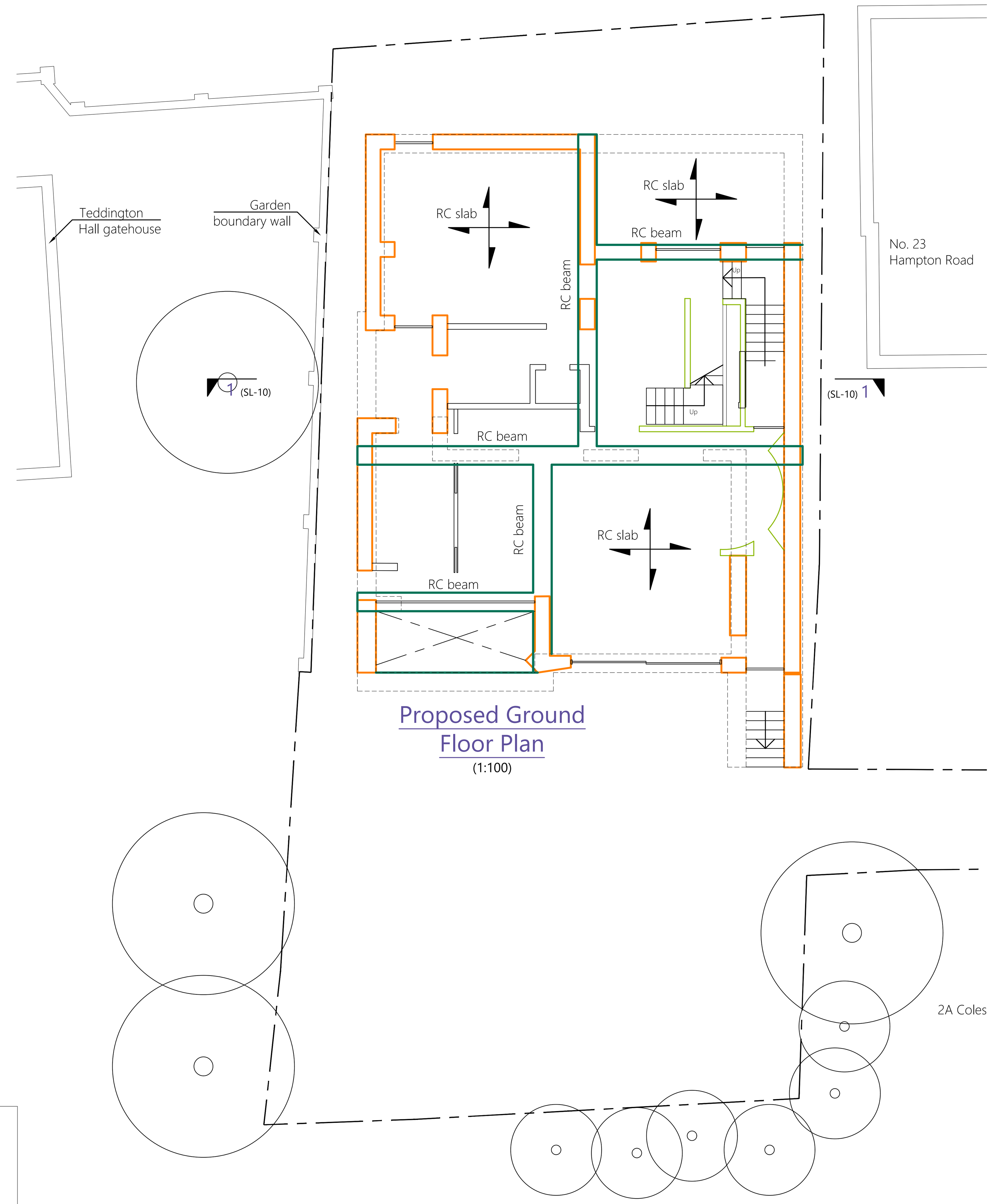
1. BGS Sheet 1:50000 scale Sheet 270 South London.
2. BRE Special Digest 1 (SD1). Concrete in Aggressive Ground. Building Research Establishment 2005.
3. BSI 8004, 2015. Code of practice for foundations. The British Standards Institution.
4. BSI 8002, 1994. Code of practice for earth retaining structures. The British Standards Institution.
5. Guidance on embedded retaining wall design, CIRIA C760.
6. Ordnance Survey mapping.
7. Padfield and Sharrock, 1983. Settlement of structures on clay soils. CIRIA Special Publication 27.

Appendix A

Drawings

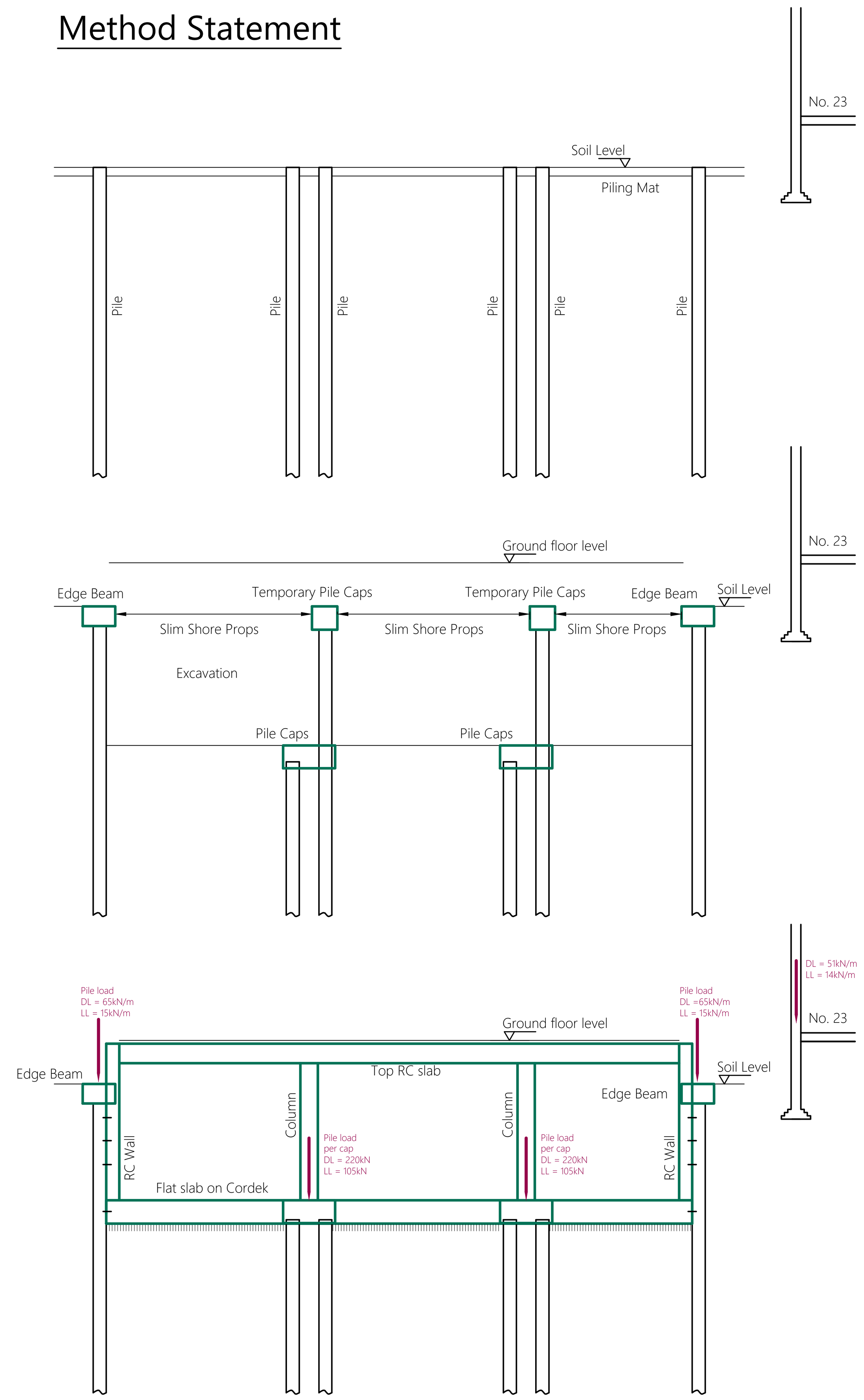


Proposed Basement Plan
(1:100)



Proposed Ground Floor Plan
(1:100)

Method Statement



Phase 1

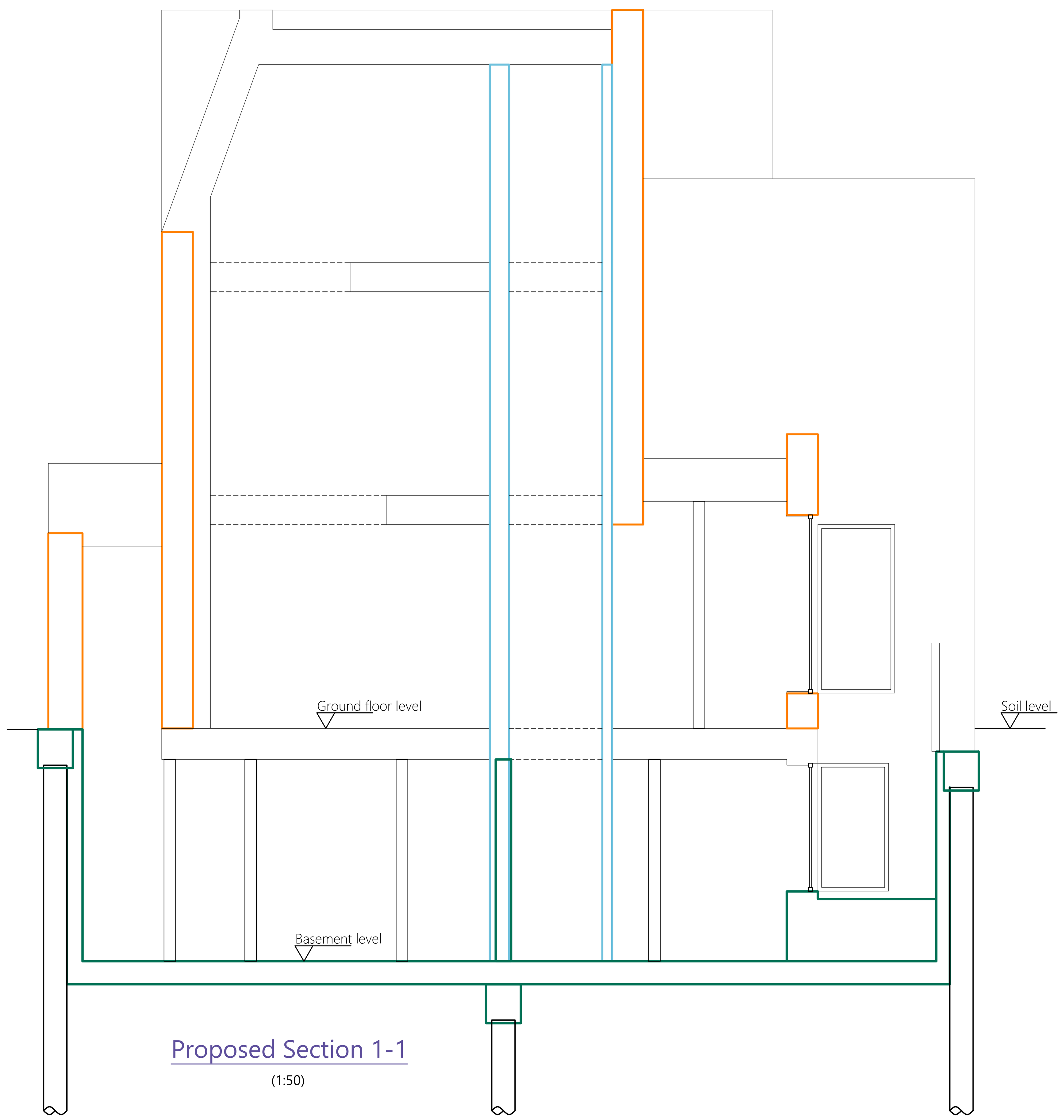
1. Install Piles

Phase 2

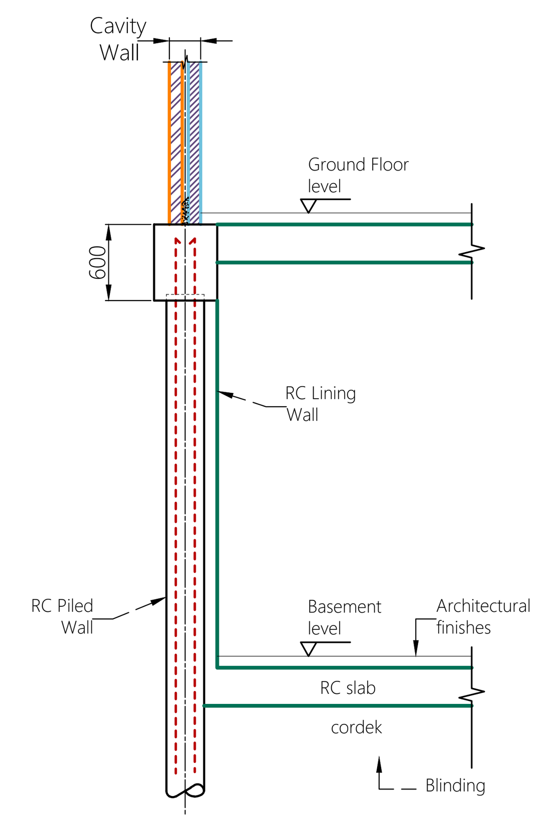
1. Bulk excavate 600mm of soil
2. Cut Edge Piles as required, cast Edge Beams, temporary Pile Caps and Slim Shore Props between them
3. Continue the excavation to the desired level
4. Cut middle piles as required, install Pile caps

Phase 3

1. Cast min. 50mm thick concrete blinding and Place Cordek anti-heave protection.
2. Install Flat Slab, this to be anchored onto the piles.
3. Install RC Walls
4. Install Columns
5. Install Flat Slab at ground floor level (Slab to be cast sequentially with local removal of top propping).



Proposed Section 1-1
(1:50)



Typical RC Pile Wall Section
(1:50)

Rev	Date	By	Amendments
-	??/08/23	JH	First issue

CRØFT STRUCTURAL+ CIVIL

r/o 60 Saxon Rd, London, SE25 5EH.
020 8684 4744
www.croftse.co.uk

Simon Kinsman

23A Hampton Road
Teddington TW11 0JN

Proposed Plans

Issued for **INFORMATION ONLY**

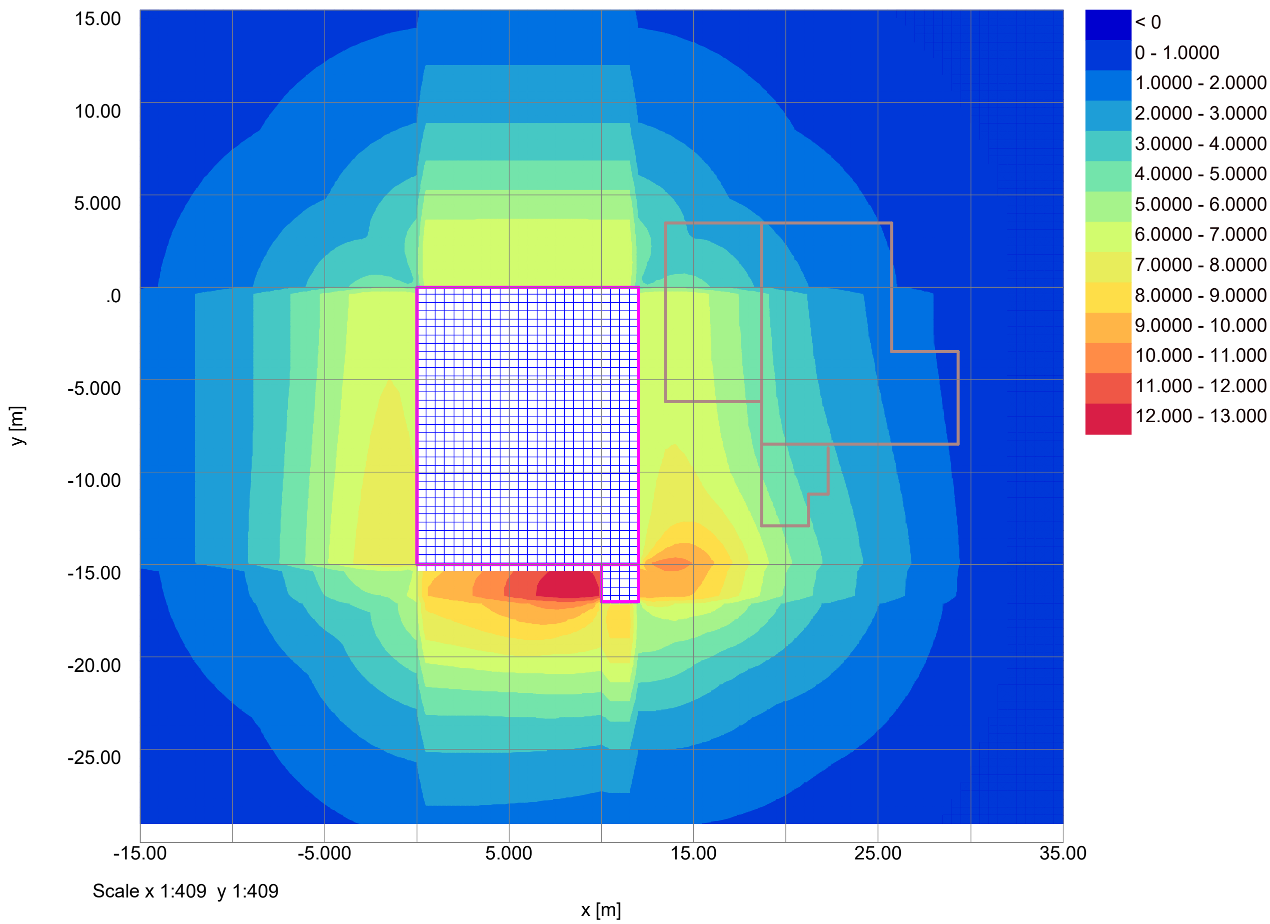
Job Number	Dwg Number	Rev
230705	SL-10	-
By	Approved by	Scale
JH	EP	1:50 @ A1

Appendix B

XDISP Outputs

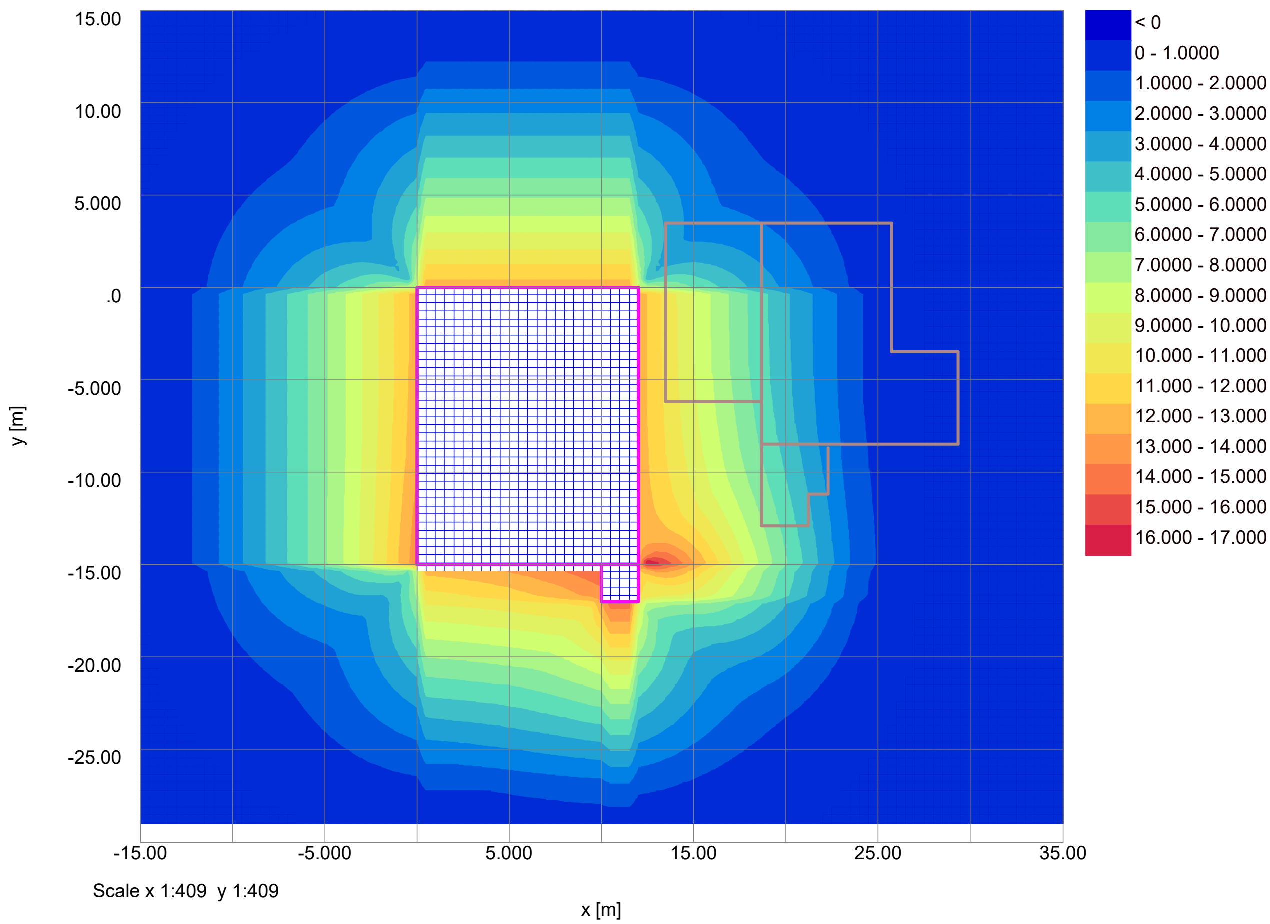
Oasys	Job No.	Sheet No.	Rev.
23A Hampton Road GMA	100896		
	Drg. Ref.		
	Made by	Date 18-Sep-2023	Checked / Date

Vertical Settlement Contours: Grid 1 (level 0.000m) (Interval 1mm)



Scale x 1:409 y 1:409

Horizontal Displacement Contours: Grid 1 (level 0.000m) Interval 1mm



Scale x 1:409 y 1:409

Stage: Ref.	Stage: Name	Specific Building: Ref.	Specific Building: Name	Sub-building Name	Vertical Offset from Line for Vertical Movement	Segment	Start	Length	Curvature	Deflection Ratio	Average Horizontal Strain	Max Tensile Strain	Max Gradient of Horizontal Displacement	Max Gradient of Vertical Displacement Curve	Min Radius of Curvature	Damage Category

Tensile horizontal strains are +ve, compressive horizontal strains are -ve.

Specific Building Damage Results - Critical Values for All Segments within Each Sub-Building

Stage: Ref.	Stage: Name	Specific Building: Ref.	Specific Building: Name	Sub-building Name	Vertical Offset from Line for Vertical Movement Calculations [m]	Deflection Ratio [%]	Average Horizontal Strain [%]	Max Slope	Max Settlement [mm]	Max Tensile Strain [%]	Max Gradient of Horizontal Displacement Curve	Max Gradient of Vertical Displacement Curve	Min Radius of Curvature (Hogging) [m]	Min Radius of Curvature (Sagging) [m]	Damage Category
0	Base Model	1	23	Sub 1	0.0	0.037446	-0.079062	-0.0016547	6.7848	0.027632	0.0011923	-0.0016547	-	-	0 (Negligible)
		2	23	Sub 2	0.0	297.27E-6	0.0	-28.412E-6	6.8627	417.46E-6	0.0	-28.412E-6	-	-	0 (Negligible)
		3	23	Sub 3	0.0	0.0088891	0.10389	627.29E-6	6.8584	0.11667	-0.0011194	627.29E-6	-	-	2 (Slight)
		4	23	Sub 4	0.0	0.016064	-0.017881	-872.74E-6	4.7257	0.015167	232.50E-6	-872.74E-6	-	-	0 (Negligible)
		5	23	Sub 4a	0.0	0.0017705	0.0	-232.08E-6	5.2353	0.0019269	0.0	-232.08E-6	-	-	0 (Negligible)
		6	23	Sub 5	0.0	214.57E-6	0.0	-295.43E-6	5.7337	196.02E-6	0.0	-295.43E-6	-	-	0 (Negligible)
		7	23	Sub 6	0.0	0.0035706	0.11034	766.04E-6	5.7337	0.11442	-0.0011141	766.04E-6	-	-	2 (Slight)
		8	23	Sub 7	0.0	130.63E-6	0.0	161.71E-6	4.1141	119.55E-6	0.0	161.71E-6	-	-	0 (Negligible)
		9	23	Sub 8	0.0	815.16E-6	0.10063	435.28E-6	3.8446	0.10108	-0.0010067	435.28E-6	-	-	2 (Slight)
		10	23	Sub 9	0.0	144.89E-6	0.0	123.94E-6	3.4016	157.68E-6	0.0	123.94E-6	-	-	0 (Negligible)
		11	23	Sub 10	0.0	0.0033111	0.078321	530.13E-6	4.6759	0.081044	-926.44E-6	530.13E-6	-	-	2 (Slight)
		12	23	Sub 11	0.0	0.0044656	-0.022733	430.98E-6	3.4589	0.018343	428.29E-6	430.98E-6	-	-	0 (Negligible)
		13	23	Sub 12	0.0	0.0010254	0.024273	217.33E-6	2.0469	0.025150	-330.31E-6	217.33E-6	-	-	0 (Negligible)
		14	23	Sub 13	0.0	799.66E-6	0.0	41.912E-6	0.82018	769.72E-6	0.0	41.912E-6	-	-	0 (Negligible)
		15	23	Sub 14	0.0	0.0010637	0.024056	-304.00E-6	1.7003	0.024100	-351.25E-6	-304.00E-6	-	-	0 (Negligible)
		16	23	Sub 15	0.0	0.011691	-796.49E-6	756.15E-6	1.7003	0.011363	10.472E-6	756.15E-6	-	-	0 (Negligible)

Specific Building Damage Results - Critical Segments within Each Building

Stage: Ref.	Stage: Name	Specific Building: Ref.	Specific Building: Name	Parameter	Critical Sub-Building	Critical Segment	Start [m]	End [m]	Curvature	Max Slope	Max Settlement [mm]	Max Tensile Strain [%]	Min Radius of Curvature (Hogging) [m]	Min Radius of Curvature (Sagging) [m]	Damage Category
0	Base Model	0	23	Max Slope	Sub 1	1	0.0	3.0494	None	**0.0016547**	5.0283	0.027632	-	-	0 (Negligible)
				Max Settlement	Sub 2	1	0.0	3.7000	None	28.412E-6	**6.8627**	417.46E-6	-	-	0 (Negligible)
				Max Tensile Strain	Sub 3	1	0.0	3.9575	None	627.29E-6	6.8584	**0.11667**	-	-	2 (Slight)
				Min Radius of Curvature (Hogging)		-	-	-	-	-	-	-	-	-	-
				Min Radius of Curvature (Sagging)		-	-	-	-	-	-	-	-	-	-

Appendix B – Structural Design

As part of the building control application, full calculations must be undertaken and provided at detailed design stage once planning permission is granted. The calculations must be completed to a recognised standard (British Standards or Eurocode). The calculations must take into account the findings of this report.

The design must resist:

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

The final proposed scheme must:

1. Provide stability in the temporary condition to all forces.
2. Provide stability to all forces in the permanent condition.

As part of the planning process, Croft Structural Engineers has considered some of the pertinent parts of the basement structure to ensure that it can be constructed. The following calculations are not a full set of calculations for the final design. The structural calculations that Croft considers pertinent are included in this appendix. Calculations relevant to the temporary works are in the proposed method statement in the next appendix.

Retaining Wall

Location	Area			Type	L	Action	Actions, kN or kN/m			
	L	W	m^2			kN/m^2	Perm., g_k	%	Var., q_k	Total
Retaining Wall										
Pitched roof	1.7	1	1.7	g_k		1.15	2.0			
				q_k		0.60			1.0	
Second floor	1.7	1	1.7	g_k		0.88	1.5			
				q_k		2.30			3.9	
First floor	1.7	1	1.7	g_k		0.88	1.5			
				q_k		2.30			3.9	
Ground floor	1.7	1	1.7	g_k		9.07	15.4			
				q_k		2.30			3.9	
External wall	10	1	10	g_k		3.98	39.8			
							60.2	kN/m	12.8	kN/m

RETAINING WALL ANALYSIS

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

Tedds calculation version 2.9.21

Analysis summary

Design summary

Overall design utilisation 1.28

Overall design status Fail

Description	Unit	Capacity	Applied	F o S	Result
Overturning stability	kNm/m	191	145.5	1.313	PASS
Bearing pressure	kN/m^2	100	76.5	1.308	PASS

Design summary

Description	Unit	Provided	Required	Utilisation	Result
Shear resistance	kN/m	60.4	165.1	0.366	PASS
Stem p1 - Shear resistance	kN/m	126.9	65.7	0.518	PASS
Base bottom face - Flexural reinforcement	mm^2/m	2010.6	1171.1	0.582	PASS
Base - Shear resistance	kN/m	165.1	60.4	0.366	PASS
Min. transverse stem reinf.	mm^2/m	565.5	502.7	0.889	PASS
Min. transverse base reinf.	mm^2/m	565.5	402.1	0.711	PASS

Retaining wall details

Stem type Cantilever

Stem height h_{stem} = **3000** mm

Stem thickness t_{stem} = **350** mm

Angle to rear face of stem α = **90** deg

Stem density	γ_{stem} = **25** kN/m^3
Toe length	l_{toe} = **1800** mm
Base thickness	t_{base} = **350** mm
Base density	γ_{base} = **25** kN/m^3
Height of retained soil	h_{ret} = **3000** mm
Angle of soil surface	β = **0** deg
Depth of cover	d_{cover} = **0** mm
Height of water	h_{water} = **3000** mm
Water density	γ_w = **9.8** kN/m^3

Retained soil properties

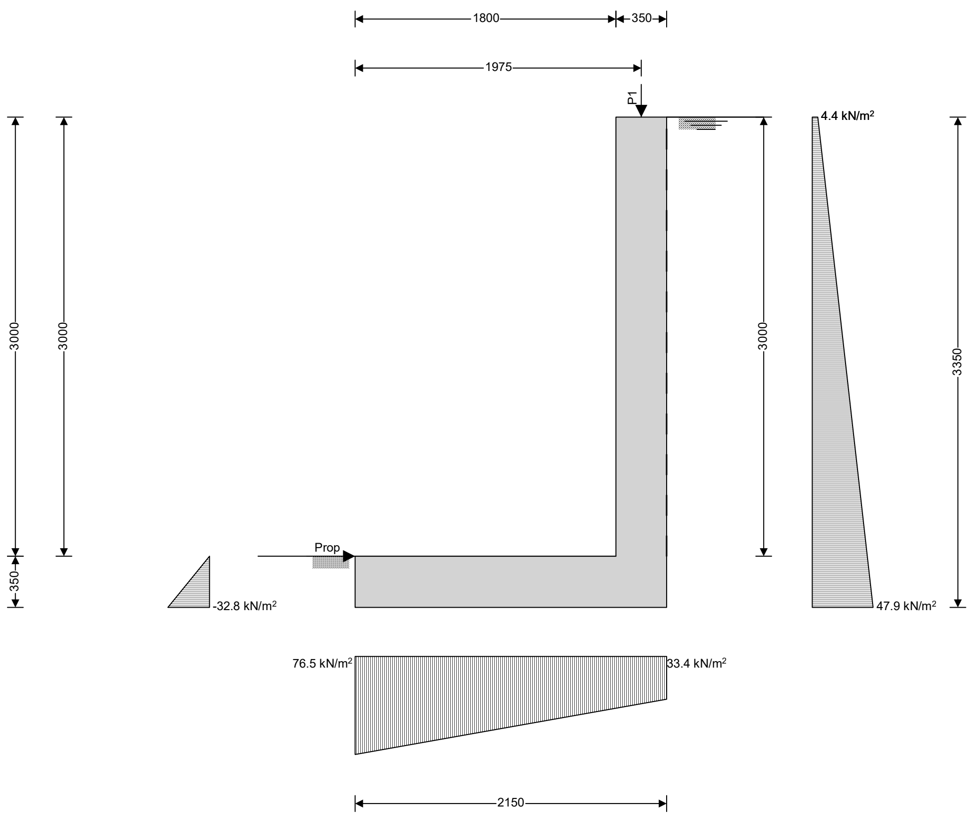
Soil type	Medium dense coarse and medium sand
Moist density	γ_{mr} = **17.5** kN/m^3
Saturated density	γ_{sr} = **20.8** kN/m^3
Characteristic effective shear resistance angle	$\phi'_{r.k}$ = **30** deg
Characteristic wall friction angle	$\delta_{r.k}$ = **15** deg

Base soil properties

Soil type	Medium dense well graded sand
Soil density	γ_b = **19.5** kN/m^3
Characteristic effective shear resistance angle	$\phi'_{b.k}$ = **30** deg
Characteristic wall friction angle	$\delta_{b.k}$ = **15** deg
Characteristic base friction angle	$\delta_{bb.k}$ = **20** deg
Presumed bearing capacity	$P_{bearing}$ = **100** kN/m^2

Loading details

Permanent surcharge load	$Surcharge_G$ = **5** kN/m^2
Variable surcharge load	$Surcharge_Q$ = **10** kN/m^2
Vertical line load at 1975 mm	P_{G1} = **60.2** kN/m
	P_{Q1} = **12.8** kN/m



General arrangement - sketch pressures relate to bearing check

Calculate retaining wall geometry

Base length	$l_{base} = l_{toe} + t_{stem}$ = **2150** mm
Saturated soil height	$h_{sat} = h_{water} + d_{cover}$ = **3000** mm
Moist soil height	$h_{moist} = h_{ret} - h_{water}$ = **0** mm
Length of surcharge load	$l_{sur} = l_{heel}$ = **0** mm
- Distance to vertical component	$x_{sur\_v} = l_{base} - l_{heel} / 2$ = **2150** mm
Effective height of wall	$h_{eff} = h_{base} + d_{cover} + h_{ret}$ = **3350** mm
- Distance to horizontal component	$x_{sur\_h} = h_{eff} / 2$ = **1675** mm
Area of wall stem	$A_{stem} = h_{stem} \times t_{stem}$ = **1.05** m^2
- Distance to vertical component	$x_{stem} = l_{toe} + t_{stem} / 2$ = **1975** mm
Area of wall base	$A_{base} = l_{base} \times t_{base}$ = **0.753** m^2
- Distance to vertical component	$x_{base} = l_{base} / 2$ = **1075** mm

Using Coulomb theory

Active pressure coefficient

$K_A = \sin(\alpha + \phi'_{r.k})^2 / (\sin(\alpha)^2 \times \sin(\alpha - \delta_{r.k}) \times [1 + \sqrt{[\sin(\phi'_{r.k} + \delta_{r.k}) \times \sin(\phi'_{r.k} - \beta) / (\sin(\alpha - \delta_{r.k}) \times \sin(\alpha + \beta))]}]^2)$ = **0.301**

Passive pressure coefficient — $K_P = \sin(90 - \phi'_{b.k})^2 / (\sin(90 + \delta_{b.k}) \times [1 - \sqrt{[\sin(\phi'_{b.k} + \delta_{b.k}) \times \sin(\phi'_{b.k}) / (\sin(90 + \delta_{b.k}))]}]^2)$ = **4.977**

Bearing pressure check

Vertical forces on wall

Wall stem	$F_{stem} = A_{stem} \times \gamma_{stem}$ = **26.3** kN/m
Wall base	$F_{base} = A_{base} \times \gamma_{base}$ = **18.8** kN/m
Line loads	$F_{P\_v} = P_{G1} + P_{Q1}$ = **73** kN/m
Total	$F_{total\_v} = F_{stem} + F_{base} + F_{P\_v} + F_{water\_v}$ = **118.1** kN/m

Horizontal forces on wall

Surcharge load	$F_{sur\_h} = K_A \times \cos(\delta_{r.k}) \times (Surcharge_G + Surcharge_Q) \times h_{eff}$ = **14.6** kN/m
Saturated retained soil	$F_{sat\_h} = K_A \times \cos(\delta_{r.k}) \times (\gamma_{sr} - \gamma_w) \times (h_{sat} + h_{base})^2 / 2$ = **17.9** kN/m
Water	$F_{water\_h} = \gamma_w \times (h_{water} + d_{cover} + h_{base})^2 / 2$ = **55** kN/m
Base soil	$F_{pass\_h} = -K_P \times \cos(\delta_{b.k}) \times \gamma_b \times (d_{cover} + h_{base})^2 / 2$ = **-5.7** kN/m
Total	$F_{total\_h} = F_{sur\_h} + F_{sat\_h} + F_{water\_h} + F_{moist\_h} + F_{pass\_h}$ = **81.8** kN/m

Moments on wall

Wall stem	$M_{stem} = F_{stem} \times x_{stem}$ = **51.8** kNm/m
Wall base	$M_{base} = F_{base} \times x_{base}$ = **20.2** kNm/m
Surcharge load	$M_{sur} = -F_{sur\_h} \times x_{sur\_h}$ = **-24.5** kNm/m
Line loads	$M_P = (P_{G1} + P_{Q1}) \times p_1$ = **144.2** kNm/m
Saturated retained soil	$M_{sat} = -F_{sat\_h} \times x_{sat\_h}$ = **-20** kNm/m
Water	$M_{water} = -F_{water\_h} \times x_{water\_h}$ = **-61.5** kNm/m
Moist retained soil	$M_{moist} = -F_{moist\_h} \times x_{moist\_h}$ = **0** kNm/m
Total	$M_{total} = M_{stem} + M_{base} + M_{sur} + M_P + M_{sat} + M_{water} + M_{moist}$ = **110.3** kNm/m

Check bearing pressure

Propping force	$F_{prop\_base} = F_{total\_h}$ = **81.8** kN/m
Distance to reaction	$\bar{x} = M_{total} / F_{total\_v}$ = **934** mm
Eccentricity of reaction	$e = \bar{x} - l_{base} / 2$ = **-141** mm
Loaded length of base	$l_{load} = l_{base}$ = **2150** mm
Bearing pressure at toe	$q_{toe} = F_{total\_v} / l_{base} \times (1 - 6 \times e / l_{base})$ = **76.5** kN/m^2
Bearing pressure at heel	$q_{heel} = F_{total\_v} / l_{base} \times (1 + 6 \times e / l_{base})$ = **33.4** kN/m^2
Factor of safety	$FoS_{bp} = P_{bearing} / \max(q_{toe}, q_{heel})$ = **1.308**

PASS - Allowable bearing pressure exceeds maximum applied bearing pressure

RETAINING WALL DESIGN

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

Tedds calculation version 2.9.21

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

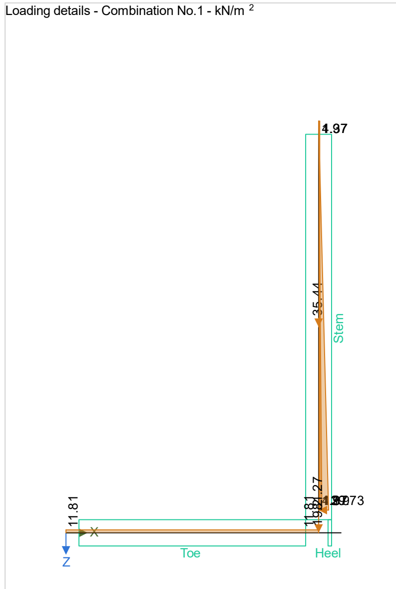
Concrete strength class	C28/35
Characteristic compressive cylinder strength	f_{ck} = **28** N/mm^2
Characteristic compressive cube strength	$f_{ck,cube}$ = **35** N/mm^2
Mean value of compressive cylinder strength	$f_{cm} = f_{ck} + 8$ N/mm^2 = **36** N/mm^2
Mean value of axial tensile strength	$f_{ctm} = 0.3$ N/mm$^2 \times (f_{ck}$ / 1 N/mm$^2)^{2/3}$ = **2.8** N/mm^2
5% fractile of axial tensile strength	$f_{ctk,0.05} = 0.7 \times f_{ctm}$ = **1.9** N/mm^2
Secant modulus of elasticity of concrete	$E_{cm} = 22$ kN/mm$^2 \times (f_{cm}$ / 10 N/mm$^2)^{0.3}$ = **32308** N/mm^2
Partial factor for concrete - Table 2.1N	γ_C = **1.50**
Compressive strength coefficient - cl.3.1.6(1)	α_{cc} = **0.85**
Design compressive concrete strength - exp.3.15	$f_{cd} = \alpha_{cc} \times f_{ck} / \gamma_C$ = **15.9** N/mm^2
Maximum aggregate size	h_{agg} = **20** mm
Ultimate strain - Table 3.1	ε_{cu2} = **0.0035**
Shortening strain - Table 3.1	ε_{cu3} = **0.0035**
Effective compression zone height factor	λ = **0.80**
Effective strength factor	η = **1.00**
Bending coefficient k_1	K_1 = **0.40**
Bending coefficient k_2	$K_2 = 1.00$ ´ $(0.6 + 0.0014/\varepsilon_{cu2})$ = **1.00**
Bending coefficient k_3	K_3 =**0.40**
Bending coefficient k_4	$K_4 = 1.00$ ´ $(0.6 + 0.0014/\varepsilon_{cu2})$ =**1.00**

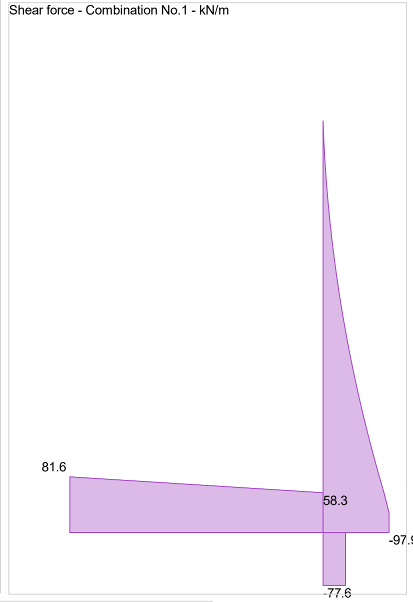
Reinforcement details

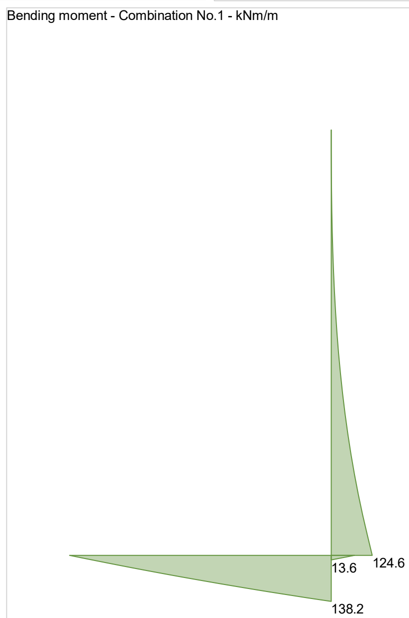
Characteristic yield strength of reinforcement	f_{yk} = **500** N/mm^2
Modulus of elasticity of reinforcement	E_s = **200000** N/mm^2
Partial factor for reinforcing steel - Table 2.1N	γ_S = **1.15**
Design yield strength of reinforcement	$f_{yd} = f_{yk} / \gamma_S$ = **435** N/mm^2

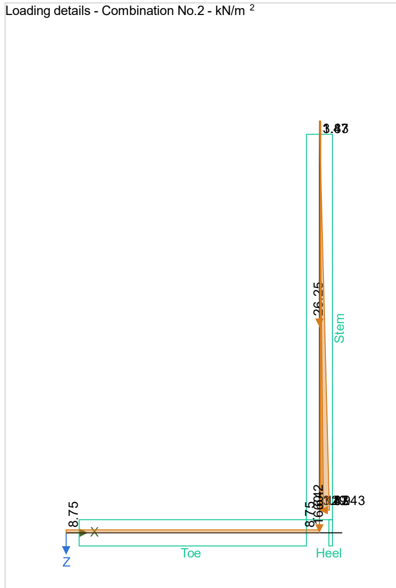
Cover to reinforcement

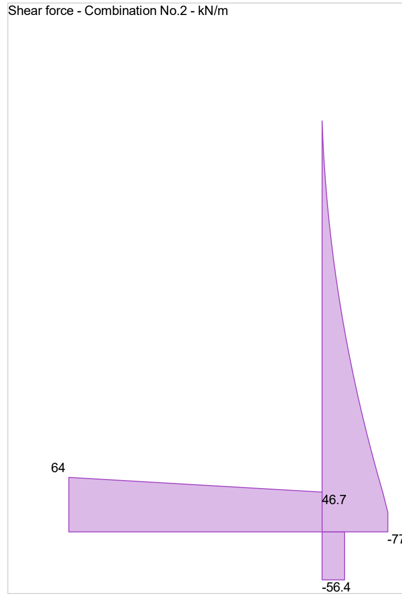
Front face of stem	c_{sf} = **40** mm
Rear face of stem	c_{sr} = **75** mm
Top face of base	c_{bt} = **40** mm
Bottom face of base	c_{bb} = **75** mm

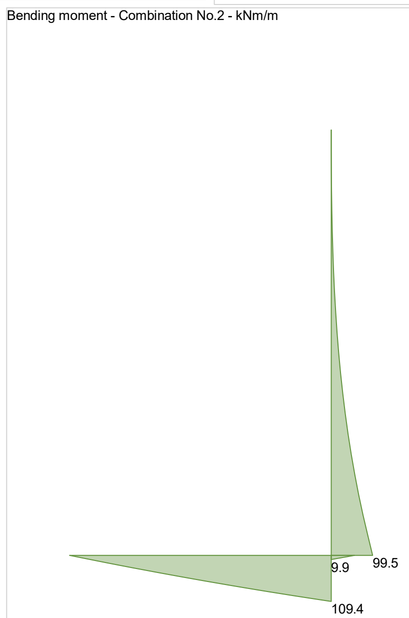

Loading details - Combination No.1 - kN/m 2
4.97
35.44
Stem
11.81
11.81
Toe
Heel
Z
X


Shear force - Combination No.1 - kN/m
81.6
58.3
-97.9
-77.6


Bending moment - Combination No.1 - kNm/m
124.6
13.6
138.2







Check stem design at base of stem

Depth of section h = **350** mm

Rectangular section in flexure - Section 6.1

Design bending moment combination 1 M = **107.4** kNm/m

Depth to tension reinforcement $d = h - c_{sr} - \phi_{sr} / 2 =$ **267** mm

$K = M / (d^2 \times f_{ck}) =$ **0.054**

K' = (2 ´ η ´ α_{cc}/γ_C) ´ (1 - λ ´ (δ - K_1)/(2 ´ K_2)) ´ (λ ´ (δ - K_1)/(2 ´ K_2))

K' = **0.207**

K' > K - No compression reinforcement is required

Lever arm	$z = \min(0.5 + 0.5$ ´ $(1 - 2$ ´ $K / (\eta$ ´ $\alpha_{cc} / \gamma_C))^{0.5}, 0.95)$ ´
d = **254** mm	
Depth of neutral axis	$x = 2.5 \times (d – z)$ = **33** mm
Area of tension reinforcement required	$A_{sr.req} = M / (f_{yd} \times z)$ = **974** mm^2/m
Tension reinforcement provided	16 dia.bars @ 100 c/c
Area of tension reinforcement provided	$A_{sr.prov} = \pi$ ´ $\phi_{sr}^2 / (4$ ´ $s_{sr})$ = **2011** mm^2/m
Minimum area of reinforcement - exp.9.1N	$A_{sr.min} = \max(0.26$ ´ $f_{ctm} / f_{yk}, 0.0013)$ ´ d = **384** mm^2/m
Maximum area of reinforcement - cl.9.2.1.1(3)	$A_{sr.max} = 0.04$ ´ h = **14000** mm^2/m
	$\max(A_{sr.req}, A_{sr.min}) / A_{sr.prov}$ = **0.485**

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

Library item: Rectangular single output

Deflection control - Section 7.4

Reference reinforcement ratio	$\rho_0 = \sqrt{}(f_{ck} / 1\ N/mm^2) / 1000$ = **0.005**
Required tension reinforcement ratio	$\rho = A_{sr.req} / d$ = **0.004**
Required compression reinforcement ratio	$\rho' = A_{sr.2.req} / d_2$ = **0.000**
Structural system factor - Table 7.4N	K_b = **0.4**
Reinforcement factor - exp.7.17	$K_s = \min(500\ N/mm^2 / (f_{yk}$ ´ $A_{sr.req} / A_{sr.prov}), 1.5)$ = **1.5**
Limiting span to depth ratio - exp.7.16.a	$\min(K_s$ ´ K_b ´ $[11 + 1.5$ ´ $\sqrt{}(f_{ck} / 1\ N/mm^2)$ ´ $\rho_0 / \rho +$
	3.2 ´ $\sqrt{}(f_{ck} / 1\ N/mm^2)$ ´ $(\rho_0 / \rho - 1)^{3/2}], 40$ ´ $K_b)$ = **16**
Actual span to depth ratio	h_{stem} / d = **11.2**

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

Crack control - Section 7.3

Limiting crack width	w_{max} = **0.3** mm
Variable load factor - EN1990 – Table A1.1	ψ_2 = **0.6**
Serviceability bending moment	M_{sls} = **72.9** kNm/m
Tensile stress in reinforcement	$\sigma_s = M_{sls} / (A_{sr.prov}$ ´ $z)$ = **142.9** N/mm^2
Load duration	Long term
Load duration factor	k_t = **0.4**
Effective area of concrete in tension	$A_{c.eff} = \min(2.5$ ´ $(h - d), (h - x) / 3, h / 2)$
	$A_{c.eff}$ = **105542** mm^2/m
Mean value of concrete tensile strength	$f_{ct.eff} = f_{ctm}$ = **2.8** N/mm^2
Reinforcement ratio	$\rho_{p.eff} = A_{sr.prov} / A_{c.eff}$ = **0.019**
Modular ratio	$\alpha_e = E_s / E_{cm}$ = **6.19**
Bond property coefficient	k_1 = **0.8**
Strain distribution coefficient	k_2 = **0.5**
	k_3 = **3.4**
	k_4 = **0.425**
Maximum crack spacing - exp.7.11	$s_{r.max} = k_3$ ´ $c_{sr} + k_1$ ´ k_2 ´ k_4 ´ $\phi_{sr} / \rho_{p.eff}$ = **398** mm

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

w_k = **0.171** mm

w_k / w_{max} = **0.569**

PASS - Maximum crack width is less than limiting crack width

Rectangular section in shear - Section 6.2

Design shear force — V = **97.9** kN/m

$C_{Rd,c} = 0.18 / \gamma_C$ = **0.120**

$k = \min(1 + \sqrt{(200 \text{ mm} / d)}, 2)$ = **1.865**

Longitudinal reinforcement ratio — $\rho_l = \min(A_{sr.prov} / d, 0.02)$ = **0.008**

$v_{min} = 0.035$ N$^{1/2}$/mm ´ $k^{3/2}$ ´ $f_{ck}^{0.5}$ = **0.472** N/mm^2

Design shear resistance - exp.6.2a & 6.2b — $V_{Rd.c} = \max(C_{Rd.c}$ ´ k ´ (100 N^2/mm^4 ´ ρ_l ´ $f_{ck})^{1/3}$, v_{min})
´ d

$V_{Rd.c}$ = **165.1** kN/m

$V / V_{Rd.c}$ = **0.593**

PASS - Design shear resistance exceeds design shear force

Check stem design at 600 mm

Depth of section — h = **350** mm

Rectangular section in flexure - Section 6.1

Design bending moment combination 1 — M = **58.7** kNm/m

Depth to tension reinforcement — $d = h - c_{sr} - \phi_{sr1} / 2$ = **270** mm

$K = M / (d^2 \times f_{ck})$ = **0.029**

K' = (2 ´ η ´ α_{cc}/γ_C) ´ (1 - λ ´ (δ - K_1)/(2 ´ K_2)) ´ (λ ´
(δ - K_1)/(2 ´ K_2))

K' = **0.207**

K' > K - No compression reinforcement is required

Lever arm — z = min(0.5 + 0.5 ´ (1 - 2 ´ K / (η ´ α_{cc} / γ_C))$^{0.5}$, 0.95) ´
d = **257** mm

Depth of neutral axis — $x = 2.5 \times (d - z)$ = **34** mm

Area of tension reinforcement required — $A_{sr1.req} = M / (f_{yd} \times z)$ = **526** mm^2/m

Tension reinforcement provided — 10 dia.bars @ 100 c/c

Area of tension reinforcement provided — $A_{sr1.prov}$ = π ´ ϕ_{sr1}^2 / (4 ´ s_{sr1}) = **785** mm^2/m

Minimum area of reinforcement - exp.9.1N — $A_{sr1.min}$ = max(0.26 ´ f_{ctm} / f_{yk}, 0.0013) ´ d = **388** mm^2/m

Maximum area of reinforcement - cl.9.2.1.1(3) — $A_{sr1.max}$ = 0.04 ´ h = **14000** mm^2/m

$\max(A_{sr1.req}, A_{sr1.min}) / A_{sr1.prov}$ = **0.67**

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

Library item: Rectangular single output

Deflection control - Section 7.4

Reference reinforcement ratio	$\rho_0 = \sqrt{(f_{ck} / 1\ N/mm^2)} / 1000 = \mathbf{0.005}$
Required tension reinforcement ratio	$\rho = A_{sr1.req} / d = \mathbf{0.002}$
Required compression reinforcement ratio	$\rho' = A_{sr1.2.req} / d_2 = \mathbf{0.000}$
Structural system factor - Table 7.4N	$K_b = \mathbf{0.4}$
Reinforcement factor - exp.7.17	$K_s = \min(500\ N/mm^2 / (f_{yk} \times A_{sr1.req} / A_{sr1.prov}), 1.5) = \mathbf{1.493}$
Limiting span to depth ratio - exp.7.16.a	$\min(K_s \times K_b \times [11 + 1.5 \times \sqrt{(f_{ck} / 1\ N/mm^2)} \times \rho_0 / \rho + 3.2 \times \sqrt{(f_{ck} / 1\ N/mm^2)} \times (\rho_0 / \rho - 1)^{3/2}], 40 \times K_b) = \mathbf{16}$
Actual span to depth ratio	$(h_{stem} - 600\ mm) / d = \mathbf{8.9}$

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

Crack control - Section 7.3

Limiting crack width	$w_{max} = \mathbf{0.3}$ mm
Variable load factor - EN1990 – Table A1.1	$\psi_2 = \mathbf{0.6}$
Serviceability bending moment	$M_{sls} = \mathbf{39.2}$ kNm/m
Tensile stress in reinforcement	$\sigma_s = M_{sls} / (A_{sr1.prov} \times z) = \mathbf{194.4}\ N/mm^2$
Load duration	Long term
Load duration factor	$k_t = \mathbf{0.4}$
Effective area of concrete in tension	$A_{c.eff} = \min(2.5 \times (h - d), (h - x) / 3, h / 2)$
	$A_{c.eff} = \mathbf{105417}\ mm^2/m$
Mean value of concrete tensile strength	$f_{ct.eff} = f_{ctm} = \mathbf{2.8}\ N/mm^2$
Reinforcement ratio	$\rho_{p.eff} = A_{sr1.prov} / A_{c.eff} = \mathbf{0.007}$
Modular ratio	$\alpha_e = E_s / E_{cm} = \mathbf{6.19}$
Bond property coefficient	$k_1 = \mathbf{0.8}$
Strain distribution coefficient	$k_2 = \mathbf{0.5}$
	$k_3 = \mathbf{3.4}$
	$k_4 = \mathbf{0.425}$
Maximum crack spacing - exp.7.11	$s_{r.max} = k_3 \times c_{sr} + k_1 \times k_2 \times k_4 \times \phi_{sr1} / \rho_{p.eff} = \mathbf{483}$ mm
Maximum crack width - exp.7.8	$w_k = s_{r.max} \times \max(\sigma_s - k_t \times (f_{ct.eff} / \rho_{p.eff}) \times (1 + \alpha_e \times \rho_{p.eff}), 0.6 \times \sigma_s) / E_s$
	$w_k = \mathbf{0.282}$ mm
	$w_k / w_{max} = \mathbf{0.939}$

PASS - Maximum crack width is less than limiting crack width

Rectangular section in shear - Section 6.2

Design shear force	$V = \mathbf{65.7}$ kN/m
	$C_{Rd,c} = 0.18 / \gamma_C = \mathbf{0.120}$
	$k = \min(1 + \sqrt{(200\ mm / d)}, 2) = \mathbf{1.861}$
Longitudinal reinforcement ratio	$\rho_l = \min(A_{sr1.prov} / d, 0.02) = \mathbf{0.003}$
	$v_{min} = 0.035\ N^{1/2}/mm \times k^{3/2} \times f_{ck}^{0.5} = \mathbf{0.470}\ N/mm^2$

Design shear resistance - exp.6.2a & 6.2b $V_{Rd.c} = \max(C_{Rd.c} ´ k ´ (100\ N^2/mm^4 ´ \rho_l ´ f_{ck})^{1/3}, v_{min})$ ´ d

$V_{Rd.c}$ = **126.9** kN/m

$V / V_{Rd.c}$ = **0.518**

PASS - Design shear resistance exceeds design shear force

Horizontal reinforcement parallel to face of stem - Section 9.6

Minimum area of reinforcement – cl.9.6.3(1) $A_{sx.req} = \max(0.25 ´ A_{sr.prov}, 0.001 ´ t_{stem})$ = **503** mm^2/m

Maximum spacing of reinforcement – cl.9.6.3(2) s_{sx_max} = **400** mm

Transverse reinforcement provided 12 dia.bars @ 200 c/c

Area of transverse reinforcement provided $A_{sx.prov} = \pi ´ \phi_{sx}^2 / (4 ´ s_{sx})$ = **565** mm^2/m

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

Check base design at toe

Depth of section h = **350** mm

Rectangular section in flexure - Section 6.1

Design bending moment combination 1 M = **127.8** kNm/m

Depth to tension reinforcement $d = h - c_{bb} - \phi_{bb} / 2$ = **267** mm

$K = M / (d^2 \times f_{ck})$ = **0.064**

$K' = (2 ´ \eta ´ \alpha_{cc}/\gamma_C) ´ (1 - \lambda ´ (\delta - K_1)/(2 ´ K_2)) ´ (\lambda ´ (\delta - K_1)/(2 ´ K_2))$

K' = **0.207**

K' > K - No compression reinforcement is required

Lever arm $z = \min(0.5 + 0.5 ´ (1 - 2 ´ K / (\eta ´ \alpha_{cc} / \gamma_C))^{0.5}, 0.95) ´ d$ = **251** mm

Depth of neutral axis $x = 2.5 \times (d – z)$ = **40** mm

Area of tension reinforcement required $A_{bb.req} = M / (f_{yd} \times z)$ = **1171** mm^2/m

Tension reinforcement provided 16 dia.bars @ 100 c/c

Area of tension reinforcement provided $A_{bb.prov} = \pi ´ \phi_{bb}^2 / (4 ´ s_{bb})$ = **2011** mm^2/m

Minimum area of reinforcement - exp.9.1N $A_{bb.min} = \max(0.26 ´ f_{ctm} / f_{yk}, 0.0013) ´ d$ = **384** mm^2/m

Maximum area of reinforcement - cl.9.2.1.1(3) $A_{bb.max} = 0.04 ´ h$ = **14000** mm^2/m

$\max(A_{bb.req}, A_{bb.min}) / A_{bb.prov}$ = **0.582**

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

Library item: Rectangular single output

Crack control - Section 7.3

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

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

Serviceability bending moment M_{sls} = **87.4** kNm/m

Tensile stress in reinforcement $\sigma_s = M_{sls} / (A_{bb.prov} ´ z)$ = **173.3** N/mm^2

Load duration Long term

Load duration factor	k_t = **0.4**
Effective area of concrete in tension	$A_{c.eff}$ = min(2.5 ´ (h - d), (h - x) / 3, h / 2)
	$A_{c.eff}$ = **103295** mm^2/m
Mean value of concrete tensile strength	$f_{ct.eff} = f_{ctm}$ = **2.8** N/mm^2
Reinforcement ratio	$\rho_{p.eff} = A_{bb.prov} / A_{c.eff}$ = **0.019**
Modular ratio	$\alpha_e = E_s / E_{cm}$ = **6.19**
Bond property coefficient	k_1 = **0.8**
Strain distribution coefficient	k_2 = **0.5**
	k_3 = **3.4**
	k_4 = **0.425**
Maximum crack spacing - exp.7.11	$s_{r.max} = k_3$ ´ $c_{bb} + k_1$ ´ k_2 ´ k_4 ´ $\phi_{bb} / \rho_{p.eff}$ = **395** mm
Maximum crack width - exp.7.8	$w_k = s_{r.max} \times \max(\sigma_s - k_t \times (f_{ct.eff} / \rho_{p.eff}) \times (1 + \alpha_e \times \rho_{p.eff}), 0.6 \times \sigma_s) / E_s$
	w_k = **0.216** mm
	w_k / w_{max} = **0.721**

PASS - Maximum crack width is less than limiting crack width

Rectangular section in shear - Section 6.2

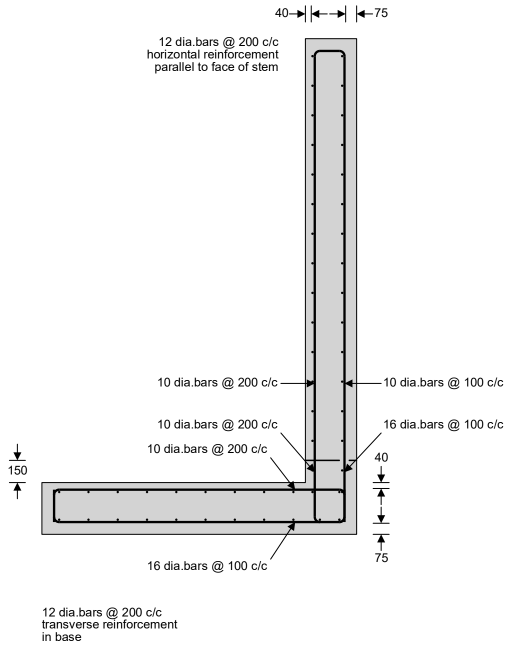
Design shear force	V = **60.4** kN/m
	$C_{Rd,c} = 0.18 / \gamma_C$ = **0.120**
	k = min(1 + $\sqrt{(200\ mm / d)}$, 2) = **1.865**
Longitudinal reinforcement ratio	$\rho_l = \min(A_{bb.prov} / d, 0.02)$ = **0.008**
	v_{min} = 0.035 $N^{1/2}/mm$ ´ $k^{3/2}$ ´ $f_{ck}^{0.5}$ = **0.472** N/mm^2
Design shear resistance - exp.6.2a & 6.2b	$V_{Rd.c}$ = max($C_{Rd.c}$ ´ k ´ (100 N^2/mm^4 ´ ρ_l ´ $f_{ck})^{1/3}$, v_{min}) ´ d
	$V_{Rd.c}$ = **165.1** kN/m
	$V / V_{Rd.c}$ = **0.366**

PASS - Design shear resistance exceeds design shear force

Secondary transverse reinforcement to base - Section 9.3

Minimum area of reinforcement – cl.9.3.1.1(2)	$A_{bx.req}$ = 0.2 ´ $A_{bb.prov}$ = **402** mm^2/m
Maximum spacing of reinforcement – cl.9.3.1.1(3)	s_{bx_max} = **450** mm
Transverse reinforcement provided	12 dia.bars @ 200 c/c
Area of transverse reinforcement provided	$A_{bx.prov} = \pi$ ´ ϕ_{bx}^2 / (4 ´ s_{bx}) = **565** mm^2/m

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



Reinforcement details

Slab

FLAT SLAB DESIGN TO BS8110:PART 1:1997

TEDDS calculation version 1.0.06

Slab geometry

Span of slab in x-direction Span_x = **4500** mm

Span of slab in y-direction Span_y = **4000** mm

Column dimension in x-direction l_x = **300** mm

Column dimension in y-direction l_y = **300** mm

External column dimension in x-direction l_{x1} = **300** mm

External column dimension in y-direction l_{y1} = **300** mm

Edge dimension in x-direction $e_x = l_{x1} / 2$ = **150** mm

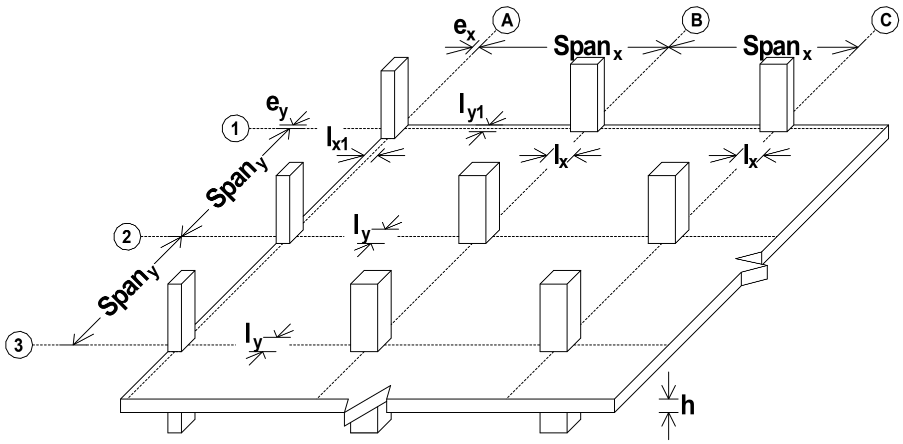
Edge dimension in y-direction $e_y = l_{y1} / 2$ = **150** mm

Effective span of internal bay in x direction $L_x = \text{Span}_x - l_x$ = **4200** mm

Effective span of internal bay in y direction $L_y = \text{Span}_y - l_y$ = **3700** mm

Effective span of end bay in x direction $L_{x1} = \text{Span}_x - l_x / 2$ = **4350** mm

Effective span of end bay in y direction $L_{y1} = Span_y – l_y / 2 =$ **3850** mm



Slab details

Depth of slab $h =$ **250** mm

Characteristic strength of concrete $f_{cu} =$ **40** N/mm^2

Characteristic strength of reinforcement $f_y =$ **500** N/mm^2

Characteristic strength of shear reinforcement $f_{yv} =$ **500** N/mm^2

Material safety factor $\gamma_m =$ **1.15**

Cover to bottom reinforcement $c =$ **75** mm

Cover to top reinforcement $c' =$ **40** mm

Loading details

Characteristic dead load $G_k =$ **7.500** kN/m^2

Characteristic imposed load $Q_k =$ **1.500** kN/m^2

Dead load factor $\gamma_G =$ **1.4**

Imposed load factor $\gamma_Q =$ **1.6**

Total ultimate load $N_{ult} = (G_k \times \gamma_G) + (Q_k \times \gamma_Q) =$ **12.900** kN/m^2

Moment redistribution ratio $\beta_b =$ **1.1**

Ratio of support moments to span moments $i =$ **1.0**

DESIGN SLAB IN THE X-DIRECTION

SAGGING MOMENTS

End bay A-B

Effective span $L =$ **4350** mm

Depth of reinforcement $d =$ **170** mm

Midspan moment $m = (N_{ult} \times L^2) / (2 \times (1 + \sqrt{(1 + i)})^2) =$ **20.940** kNm/m

Support moment $m' = i \times m =$ **20.940** kNm/m

Design reinforcement

Lever arm $K' = 0.402 \times (\beta_b – 0.4) – 0.18 \times (\beta_b – 0.4)^2 =$ **0.193**

$K = m / (d^2 \times f_{cu}) =$ **0.018**

Compression reinforcement is not required

$z = \min((0.5 + \sqrt{(0.25 – (K / 0.9))}), 0.95) \times d =$ **161.5** mm

Area of reinforcement designed $A_{s\_des} = m / (z \times f_y / \gamma_m) =$ **298** mm^2/m

Minimum area of reinforcement required $A_{s\_min} = 0.0013 \times h =$ **325** mm^2/m

Area of reinforcement required $As_{_req} = \max(As_{_des}, As_{_min}) = $ **325** mm^2/m

Provide 10 dia bars @ 200 centres

Area of reinforcement provided $As_{_prov} = \pi \times D^2 / (4 \times s) = $ **393** mm^2/m

PASS - Span reinforcement is OK

Check deflection

Design service stress $f_s = 2 \times f_y \times As_{_req} / (3 \times As_{_prov} \times \beta_b) = $ **251** N/mm^2

Modification factor $k_1 = \min(0.55+(477N/mm^2-f_s)/(120\times(0.9N/mm^2+(m/d^2))),2) = $ **1.710**

Allowable span to depth ratio $0.9 \times 26 \times k_1 = $ **40.022**

Actual span to depth ratio $L / d = $ **25.588**

PASS - Span to depth ratio is OK

Internal bay B-C

Effective span L = **4200** mm

Depth of reinforcement d = **170** mm

Midspan moment $m = (N_{ult} \times L^2) / (2 \times (\sqrt{(1 + i)} + \sqrt{(1 + i)})^2) = $ **14.222** kNm/m

Support moment $m' = i \times m = $ **14.222** kNm/m

Design reinforcement

Lever arm $K' = 0.402 \times (\beta_b - 0.4) - 0.18 \times (\beta_b - 0.4)^2 = $ **0.193**

$K = m / (d^2 \times f_{cu}) = $ **0.012**

Compression reinforcement is not required

$z = \min((0.5 + \sqrt{(0.25 - (K / 0.9))}), 0.95) \times d = $ **161.5** mm

Area of reinforcement designed $As_{_des} = m / (z \times f_y / \gamma_m) = $ **203** mm^2/m

Minimum area of reinforcement required $As_{_min} = 0.0013 \times h = $ **325** mm^2/m

Area of reinforcement required $As_{_req} = \max(As_{_des}, As_{_min}) = $ **325** mm^2/m

Provide 10 dia bars @ 200 centres

Area of reinforcement provided $As_{_prov} = \pi \times D^2 / (4 \times s) = $ **393** mm^2/m

PASS - Span reinforcement is OK

Check deflection

Design service stress $f_s = 2 \times f_y \times As_{_req} / (3 \times As_{_prov} \times \beta_b) = $ **251** N/mm^2

Modification factor $k_1 = \min(0.55+(477N/mm^2-f_s)/(120\times(0.9N/mm^2+(m/d^2))),2) = $ **1.904**

Allowable span to depth ratio $0.9 \times 26 \times k_1 = $ **44.556**

Actual span to depth ratio $L / d = $ **24.706**

PASS - Span to depth ratio is OK

HOGGING MOMENTS – INTERNAL STRIP

Penultimate column B3

Consider the reinforcement concentrated in half width strip over the support

Depth of reinforcement d' = **206** mm

Support moment $m' = 2 \times i \times m = $ **41.881** kNm/m

Lever arm $K' = 0.402 \times (\beta_b - 0.4) - 0.18 \times (\beta_b - 0.4)^2 = $ **0.193**

$K = m' / (d'^2 \times f_{cu}) = $ **0.025**

Compression reinforcement is not required

$z = \min((0.5 + \sqrt{(0.25 - (K / 0.9))}), 0.95) \times d' = $ **195.7** mm

Area of reinforcement required $As_{_des} = m' / (z \times f_y / \gamma_m) = $ **492** mm^2/m

Minimum area of reinforcement required $As_{_min} = 0.0013 \times h = $ **325** mm^2/m

Area of reinforcement required $As_{_req} = \max(As_{_des}, As_{_min}) = $ **492** mm^2/m

Provide 8 dia bars @ 100 centres

Area of reinforcement provided $A_{s\_prov} = \pi \times D^2 / (4 \times s) =$ **503** mm^2/m

PASS - Support reinforcement is OK

Internal column C3

Consider the reinforcement concentrated in half width strip over the support

Depth of reinforcement $d' =$ **205** mm

Support moment $m' = 2 \times i \times m =$ **28.444** kNm/m

Lever arm $K' = 0.402 \times (\beta_b - 0.4) - 0.18 \times (\beta_b - 0.4)^2 =$ **0.193**

$K = m' / (d'^2 \times f_{cu}) =$ **0.017**

Compression reinforcement is not required

$z = \min((0.5 + \sqrt{(0.25 - (K / 0.9))}), 0.95) \times d' =$ **194.7** mm

Area of reinforcement required $A_{s\_des} = m' / (z \times f_y / \gamma_m) =$ **336** mm^2/m

Minimum area of reinforcement required $A_{s\_min} = 0.0013 \times h =$ **325** mm^2/m

Area of reinforcement required $A_{s\_req} = \max(A_{s\_des}, A_{s\_min}) =$ **336** mm^2/m

Provide 10 dia bars @ 200 centres

Area of reinforcement provided $A_{s\_prov} = \pi \times D^2 / (4 \times s) =$ **393** mm^2/m

PASS - Support reinforcement is OK

HOGGING MOMENTS – EXTERNAL STRIP

Penultimate column B1, B2

Consider one and a half bays of negative moment being resisted over the edge and penultimate column

Width of span $B =$ **4000** mm

Edge distance $e =$ **150** mm

Depth of reinforcement $d' =$ **205** mm

Support moment $m' = m \times i \times (e + B + B / 2) / ((0.5 \times B) + (0.2 \times B) + e) =$ **43.656**

kNm/m

Lever arm $K' = 0.402 \times (\beta_b - 0.4) - 0.18 \times (\beta_b - 0.4)^2 =$ **0.193**

$K = m' / (d'^2 \times f_{cu}) =$ **0.026**

Compression reinforcement is not required

$z = \min((0.5 + \sqrt{(0.25 - (K / 0.9))}), 0.95) \times d' =$ **194.7** mm

Area of reinforcement required $A_{s\_des} = m' / (z \times f_y / \gamma_m) =$ **516** mm^2/m

Minimum area of reinforcement required $A_{s\_min} = 0.0013 \times h =$ **325** mm^2/m

Area of reinforcement required $A_{s\_req} = \max(A_{s\_des}, A_{s\_min}) =$ **516** mm^2/m

Provide 10 dia bars @ 150 centres

Area of reinforcement provided $A_{s\_prov} = \pi \times D^2 / (4 \times s) =$ **524** mm^2/m

PASS - Support reinforcement is OK

Internal column C1, C2

Consider one and a half bays of negative moment being resisted over the edge and penultimate column

Width of span $B =$ **4000** mm

Edge distance $e =$ **150** mm

Depth of reinforcement $d' =$ **205** mm

Support moment $m' = m \times i \times (e + B + B / 2) / ((0.5 \times B) + (0.2 \times B) + e) =$ **29.650**

kNm/m

Lever arm $K' = 0.402 \times (\beta_b - 0.4) - 0.18 \times (\beta_b - 0.4)^2 =$ **0.193**

$K = m' / (d'^2 \times f_{cu}) =$ **0.018**

Compression reinforcement is not required

$z = \min((0.5 + \sqrt{(0.25 - (K / 0.9))}), 0.95) \times d' =$ **194.7** mm

Area of reinforcement required $A_{s\_des} = m' / (z \times f_y / \gamma_m) =$ **350** mm²/m

Minimum area of reinforcement required $A_{s\_min} = 0.0013 \times h =$ **325** mm²/m

Area of reinforcement required $A_{s\_req} = \max(A_{s\_des}, A_{s\_min}) =$ **350** mm²/m

Provide 10 dia bars @ 200 centres

Area of reinforcement provided $A_{s\_prov} = \pi \times D^2 / (4 \times s) =$ **393** mm²/m

PASS - Support reinforcement is OK

Corner column A1

Depth of reinforcement $d' =$ **205** mm

Total load on column $S = ((Span_x / 2) + e_x) \times ((Span_y / 2) + e_y) \times N_{ult} =$ **67** kN

Area of column head $A = l_x \times l_{y1} =$ **0.090** m²

Support moment $m' = S \times (1 - (N_{ult} \times A / S)^{1/3}) / 2 =$ **24.651** kNm/m

Lever arm $K' = 0.402 \times (\beta_b - 0.4) - 0.18 \times (\beta_b - 0.4)^2 =$ **0.193**

$K = m' / (d'^2 \times f_{cu}) =$ **0.015**

Compression reinforcement is not required

$z = \min((0.5 + \sqrt{(0.25 - (K / 0.9))}), 0.95) \times d' =$ **194.7** mm

Area of reinforcement required $A_{s\_des} = m' / (z \times f_y / \gamma_m) =$ **291** mm²/m

Minimum area of reinforcement required $A_{s\_min} = 0.0013 \times h =$ **325** mm²/m

Area of reinforcement required $A_{s\_req} = \max(A_{s\_des}, A_{s\_min}) =$ **325** mm²/m

Provide 10 dia bars @ 225 centres

Area of reinforcement provided $A_{s\_prov} = \pi \times D^2 / (4 \times s) =$ **349** mm²/m

PASS - Support reinforcement is OK

Edge column A2, A3

Depth of reinforcement $d' =$ **205** mm

Total load on column $S = Span_x \times (Span_y / 2 + e_y) \times N_{ult} =$ **125** kN

Area of column head $A = l_{x1} \times l_y =$ **0.090** m²

Support moment $m' = S \times (1 - (N_{ult} \times A / S)^{1/3}) / 5.14 =$ **19.175** kNm/m

Lever arm $K' = 0.402 \times (\beta_b - 0.4) - 0.18 \times (\beta_b - 0.4)^2 =$ **0.193**

$K = m' / (d'^2 \times f_{cu}) =$ **0.011**

Compression reinforcement is not required

$z = \min((0.5 + \sqrt{(0.25 - (K / 0.9))}), 0.95) \times d' =$ **194.7** mm

Area of reinforcement required $A_{s\_des} = m' / (z \times f_y / \gamma_m) =$ **226** mm²/m

Minimum area of reinforcement required $A_{s\_min} = 0.0013 \times h =$ **325** mm²/m

Area of reinforcement required $A_{s\_req} = \max(A_{s\_des}, A_{s\_min}) =$ **325** mm²/m

Provide 10 dia bars @ 200 centres

Area of reinforcement provided $A_{s\_prov} = \pi \times D^2 / (4 \times s) =$ **393** mm²/m

PASS - Support reinforcement is OK

Between columns 1-2, 2-3

Around the perimeter between the column heads provide a minimum of 50% of the required end span bottom reinforcement.

Area of reinforcement required $A_{s\_req} = A_{sx1} / 2 =$ **196** mm²/m

Provide 10 dia bars @ 200 centres - 'U' bars with 1000 mm long legs

Area of reinforcement provided $A_{s\_prov} = \pi \times D^2 / (4 \times s) =$ **393** mm²/m

PASS - Edge reinforcement is OK

Distribution reinforcement

Provide 10 dia bars @ 200 centres

Area of reinforcement provided $A_{s\_prov} = \pi \times D^2 / (4 \times s) =$ **393** mm²/m

DESIGN SLAB IN THE Y-DIRECTION

SAGGING MOMENTS

End bay 1-2

Effective span $L = \mathbf{3850}$ mm
Depth of reinforcement $d = \mathbf{160}$ mm
Midspan moment $m = (N_{ult} \times L^2) / (2 \times (1 + \sqrt{}(1 + i))^2) = \mathbf{16.403}$ kNm/m
Support moment $m' = i \times m = \mathbf{16.403}$ kNm/m

Design reinforcement

Lever arm $K' = 0.402 \times (\beta_b - 0.4) - 0.18 \times (\beta_b - 0.4)^2 = \mathbf{0.193}$
$K = m / (d^2 \times f_{cu}) = \mathbf{0.016}$

Compression reinforcement is not required

$z = \min((0.5 + \sqrt{}(0.25 - (K / 0.9))), 0.95) \times d = \mathbf{152.0}$ mm
Area of reinforcement designed $A_{s\_des} = m / (z \times f_y / \gamma_m) = \mathbf{248}$ mm²/m
Minimum area of reinforcement required $A_{s\_min} = 0.0013 \times h = \mathbf{325}$ mm²/m
Area of reinforcement required $A_{s\_req} = \max(A_{s\_des}, A_{s\_min}) = \mathbf{325}$ mm²/m
Provide 10 dia bars @ 200 centres
Area of reinforcement provided $A_{s\_prov} = \pi \times D^2 / (4 \times s) = \mathbf{393}$ mm²/m

PASS - Span reinforcement is OK

Check deflection

Design service stress $f_s = 2 \times f_y \times A_{s\_req} / (3 \times A_{s\_prov} \times \beta_b) = \mathbf{251}$ N/mm²
Modification factor $k_1 = \min(0.55+(477\text{N/mm}^2-f_s)/(120\times(0.9\text{N/mm}^2+(m/d^2))),2) = \mathbf{1.773}$
Allowable span to depth ratio $0.9 \times 26 \times k_1 = \mathbf{41.500}$
Actual span to depth ratio $L / d = \mathbf{24.063}$

PASS - Span to depth ratio is OK

Internal bay 2-3

Effective span $L = \mathbf{3700}$ mm
Depth of reinforcement $d = \mathbf{160}$ mm
Midspan moment $m = (N_{ult} \times L^2) / (2 \times (\sqrt{}(1 + i) + \sqrt{}(1 + i))^2) = \mathbf{11.038}$ kNm/m
Support moment $m' = i \times m = \mathbf{11.038}$ kNm/m

Design reinforcement

Lever arm $K' = 0.402 \times (\beta_b - 0.4) - 0.18 \times (\beta_b - 0.4)^2 = \mathbf{0.193}$
$K = m / (d^2 \times f_{cu}) = \mathbf{0.011}$

Compression reinforcement is not required

$z = \min((0.5 + \sqrt{}(0.25 - (K / 0.9))), 0.95) \times d = \mathbf{152.0}$ mm
Area of reinforcement designed $A_{s\_des} = m / (z \times f_y / \gamma_m) = \mathbf{167}$ mm²/m
Minimum area of reinforcement required $A_{s\_min} = 0.0013 \times h = \mathbf{325}$ mm²/m
Area of reinforcement required $A_{s\_req} = \max(A_{s\_des}, A_{s\_min}) = \mathbf{325}$ mm²/m
Provide 10 dia bars @ 200 centres
Area of reinforcement provided $A_{s\_prov} = \pi \times D^2 / (4 \times s) = \mathbf{393}$ mm²/m

PASS - Span reinforcement is OK

Check deflection

Design service stress $f_s = 2 \times f_y \times A_{s\_req} / (3 \times A_{s\_prov} \times \beta_b) = \mathbf{251}$ N/mm²

Modification factor $k_1 = \min(0.55+(477N/mm^2-f_s)/(120\times(0.9N/mm^2+(m/d^2))),2) = \mathbf{1.966}$

Allowable span to depth ratio $0.9 \times 26 \times k_1 = \mathbf{46.007}$

Actual span to depth ratio $L / d = \mathbf{23.125}$

PASS - Span to depth ratio is OK

HOGGING MOMENTS – INTERNAL STRIP

Penultimate column C2

Consider the reinforcement concentrated in half width strip over the support

Depth of reinforcement $d' = \mathbf{195}$ mm

Support moment $m' = 2 \times i \times m = \mathbf{32.806}$ kNm/m

Lever arm $K' = 0.402 \times (\beta_b - 0.4) - 0.18 \times (\beta_b - 0.4)^2 = \mathbf{0.193}$

$K = m' / (d'^2 \times f_{cu}) = \mathbf{0.022}$

Compression reinforcement is not required

$z = \min((0.5 + \sqrt{(0.25 - (K / 0.9))}), 0.95) \times d' = \mathbf{185.2}$ mm

Area of reinforcement required $A_{s\_des} = m' / (z \times f_y / \gamma_m) = \mathbf{407}$ mm^2/m

Minimum area of reinforcement required $A_{s\_min} = 0.0013 \times h = \mathbf{325}$ mm^2/m

Area of reinforcement required $A_{s\_req} = \max(A_{s\_des}, A_{s\_min}) = \mathbf{407}$ mm^2/m

Provide 10 dia bars @ 150 centres

Area of reinforcement provided $A_{s\_prov} = \pi \times D^2 / (4 \times s) = \mathbf{524}$ mm^2/m

PASS - Support reinforcement is OK

Internal column C3

Consider the reinforcement concentrated in half width strip over the support

Depth of reinforcement $d' = \mathbf{195}$ mm

Support moment $m' = 2 \times i \times m = \mathbf{22.075}$ kNm/m

Lever arm $K' = 0.402 \times (\beta_b - 0.4) - 0.18 \times (\beta_b - 0.4)^2 = \mathbf{0.193}$

$K = m' / (d'^2 \times f_{cu}) = \mathbf{0.015}$

Compression reinforcement is not required

$z = \min((0.5 + \sqrt{(0.25 - (K / 0.9))}), 0.95) \times d' = \mathbf{185.2}$ mm

Area of reinforcement required $A_{s\_des} = m' / (z \times f_y / \gamma_m) = \mathbf{274}$ mm^2/m

Minimum area of reinforcement required $A_{s\_min} = 0.0013 \times h = \mathbf{325}$ mm^2/m

Area of reinforcement required $A_{s\_req} = \max(A_{s\_des}, A_{s\_min}) = \mathbf{325}$ mm^2/m

Provide 10 dia bars @ 200 centres

Area of reinforcement provided $A_{s\_prov} = \pi \times D^2 / (4 \times s) = \mathbf{393}$ mm^2/m

PASS - Support reinforcement is OK

HOGGING MOMENTS – EXTERNAL STRIP

Penultimate column A2, B2

Consider one and a half bays of negative moment being resisted over the edge and penultimate column

Width of span $B = \mathbf{4500}$ mm

Edge distance $e = \mathbf{150}$ mm

Depth of reinforcement $d' = \mathbf{195}$ mm

Support moment $m' = m \times i \times (e + B + B / 2) / ((0.5 \times B) + (0.2 \times B) + e) = \mathbf{34.298}$

kNm/m

Lever arm $K' = 0.402 \times (\beta_b - 0.4) - 0.18 \times (\beta_b - 0.4)^2 = \mathbf{0.193}$

$K = m' / (d'^2 \times f_{cu}) = \mathbf{0.023}$

Compression reinforcement is not required

$z = \min((0.5 + \sqrt{(0.25 - (K / 0.9))}), 0.95) \times d' = \mathbf{185.2}$ mm

Area of reinforcement required $A_{s\_des} = m' / (z \times f_y / \gamma_m) =$ **426** mm^2/m

Minimum area of reinforcement required $A_{s\_min} = 0.0013 \times h =$ **325** mm^2/m

Area of reinforcement required $A_{s\_req} = \max(A_{s\_des}, A_{s\_min}) =$ **426** mm^2/m

Provide 10 dia bars @ 175 centres

Area of reinforcement provided $A_{s\_prov} = \pi \times D^2 / (4 \times s) =$ **449** mm^2/m

PASS - Support reinforcement is OK

Internal column A3, B3

Consider one and a half bays of negative moment being resisted over the edge and penultimate column

Width of span B = **4500** mm

Edge distance e = **150** mm

Depth of reinforcement d' = **195** mm

Support moment $m' = m \times i \times (e + B + B / 2) / ((0.5 \times B) + (0.2 \times B) + e) =$ **23.079** kNm/m

Lever arm $K' = 0.402 \times (\beta_b - 0.4) - 0.18 \times (\beta_b - 0.4)^2 =$ **0.193**

$K = m' / (d'^2 \times f_{cu}) =$ **0.015**

Compression reinforcement is not required

$z = \min((0.5 + \sqrt{(0.25 - (K / 0.9))}), 0.95) \times d' =$ **185.2** mm

Area of reinforcement required $A_{s\_des} = m' / (z \times f_y / \gamma_m) =$ **287** mm^2/m

Minimum area of reinforcement required $A_{s\_min} = 0.0013 \times h =$ **325** mm^2/m

Area of reinforcement required $A_{s\_req} = \max(A_{s\_des}, A_{s\_min}) =$ **325** mm^2/m

Provide 10 dia bars @ 225 centres

Area of reinforcement provided $A_{s\_prov} = \pi \times D^2 / (4 \times s) =$ **349** mm^2/m

PASS - Support reinforcement is OK

Edge column B1, C1

Depth of reinforcement d' = **195** mm

Total load on column $S = (Span_x / 2 + e_x) \times Span_y \times N_{ult} =$ **124** kN

Area of column head $A = l_{y1} \times l_x =$ **0.090** m^2

Support moment $m' = S \times (1 - (N_{ult} \times A / S)^{1/3}) / 5.14 =$ **19.013** kNm/m

Lever arm $K' = 0.402 \times (\beta_b - 0.4) - 0.18 \times (\beta_b - 0.4)^2 =$ **0.193**

$K = m' / (d'^2 \times f_{cu}) =$ **0.013**

Compression reinforcement is not required

$z = \min((0.5 + \sqrt{(0.25 - (K / 0.9))}), 0.95) \times d' =$ **185.2** mm

Area of reinforcement required $A_{s\_des} = m' / (z \times f_y / \gamma_m) =$ **236** mm^2/m

Minimum area of reinforcement required $A_{s\_min} = 0.0013 \times h =$ **325** mm^2/m

Area of reinforcement required $A_{s\_req} = \max(A_{s\_des}, A_{s\_min}) =$ **325** mm^2/m

Provide 10 dia bars @ 225 centres

Area of reinforcement provided $A_{s\_prov} = \pi \times D^2 / (4 \times s) =$ **349** mm^2/m

PASS - Support reinforcement is OK

Between columns A-B, B-C

Around the perimeter between the column heads provide a minimum of 50% of the required end span bottom reinforcement.

Area of reinforcement required $A_{s\_req} = A_{sy1} / 2 =$ **196** mm^2/m

Provide 10 dia bars @ 225 centres - 'U' bars with 1000 mm long legs

Area of reinforcement provided $A_{s\_prov} = \pi \times D^2 / (4 \times s) =$ **349** mm^2/m

PASS - Edge reinforcement is OK

PUNCHING SHEAR

Corner column A1

Design shear transferred to column $V_t = ((0.45 \times Span_x) + e_x) \times ((0.45 \times Span_y) + e_y) \times N_{ult} =$ **55** kN

Design effective shear transferred to column $V_{eff} = 1.25 \times V_t =$ **68** kN

Area of tension steel in x-direction $A_{sx\_ten} = A_{scorner} =$ **349** mm^2/m

Area of tension steel in y-direction $A_{sy\_ten} = A_{scorner} =$ **349** mm^2/m

Column perimeter $u_c = l_{x1} + l_y =$ **600** mm

Average effective depth of reinforcementd = h – c' - ϕ_p = **194** mm

Maximum allowable shear stress $v_{max} = \min(0.8 \times \sqrt{(f_{cu})}, 5) =$ **5.000** N/mm^2

Design shear stress at column perimeter$v_0 = V_{eff} / (u_c \times d) =$ **0.588** N/mm^2

PASS - Maximum concrete shear stress not exceeded at column perimeter

Shear reinforcement at a perimeter of 1.50d - (291 mm)

Length of shear perimeter $u = u_c + (2 \times (k_x \times k_y) \times k \times d) =$ **1182** mm

Area of tension steel at shear perimeter $A_{s\_ten} = (k_y \times (p_x + (k_x \times k \times d)) \times A_{sy\_ten}) + (k_x \times (p_y + (k_y \times k \times d)) \times A_{sx\_ten})$

$A_{s\_ten} =$ **413** mm^2

Design concrete shear stress

$v_c=(\min(f_{cu},40)/25)^{1/3}\times0.79\times\min(100\times A_{s\_ten}/(u\times d),3)^{1/3}\times\max(400/d,1)^{1/4}/1.25$

$v_c =$ **0.500** N/mm^2

Nominal design shear stress at perimeter $v = V_{eff} / (u \times d) =$ **0.298** N/mm^2

$v < v_c$ no shear reinforcement required

Penultimate edge column A2

Design shear transferred to column $V_t = ((0.45 \times Span_x) + e_x) \times (1.05 \times Span_y) \times N_{ult} =$ **118** kN

Design effective shear transferred to column $V_{eff} = 1.4 \times V_t =$ **165** kN

Area of tension steel in x-direction $A_{sx\_ten} = A_{sx\_edge} =$ **392** mm^2/m

Area of tension steel in y-direction $A_{sy\_ten} = A_{sy1e} =$ **448** mm^2/m

Column perimeter $u_c = (2 \times l_{x1})+ l_y =$ **900** mm

Average effective depth of reinforcementd = h – c' - ϕ_p = **194** mm

Maximum allowable shear stress $v_{max} = \min(0.8 \times \sqrt{(f_{cu})}, 5) =$ **5.000** N/mm^2

Design shear stress at column perimeter$v_0 = V_{eff} / (u_c \times d) =$ **0.945** N/mm^2

PASS - Maximum concrete shear stress not exceeded at column perimeter

Shear reinforcement at a perimeter of 1.50d - (291 mm)

Length of shear perimeter $u = u_c + (2 \times (k_x \times k_y) \times k \times d) =$ **2064** mm

Area of tension steel at shear perimeter $A_{s\_ten} = (k_y \times (p_x + (k_x \times k \times d)) \times A_{sy\_ten}) + (k_x \times (p_y + (k_y \times k \times d)) \times A_{sx\_ten})$

$A_{s\_ten} =$ **875** mm^2

Design concrete shear stress

$v_c=(\min(f_{cu},40)/25)^{1/3}\times0.79\times\min(100\times A_{s\_ten}/(u\times d),3)^{1/3}\times\max(400/d,1)^{1/4}/1.25$

$v_c =$ **0.534** N/mm^2

Nominal design shear stress at perimeter $v = V_{eff} / (u \times d) =$ **0.412** N/mm^2

$v < v_c$ no shear reinforcement required

Internal edge column A3

Design shear transferred to column $V_t = ((0.45 \times Span_x) + e_x) \times Span_y \times N_{ult} =$ **112** kN

Design effective shear transferred to column $V_{eff} = 1.4 \times V_t =$ **157** kN

Area of tension steel in x-direction $A_{sx\_ten} = A_{sx\_edge} =$ **392** mm^2/m

Area of tension steel in y-direction $A_{sy\_ten} = A_{sye} =$ **349** mm^2/m

Column perimeter $u_c = (2 \times l_{x1}) + l_y =$ **900** mm

Average effective depth of reinforcement $d = h - c' - \phi_p =$ **194** mm

Maximum allowable shear stress $v_{max} = \min(0.8 \times \sqrt{(f_{cu})}, 5) =$ **5.000** N/mm^2

Design shear stress at column perimeter $v_0 = V_{eff} / (u_c \times d) =$ **0.900** N/mm^2

PASS - Maximum concrete shear stress not exceeded at column perimeter

Shear reinforcement at a perimeter of 1.50d - (291 mm)

Length of shear perimeter $u = u_c + (2 \times (k_x \times k_y) \times k \times d) =$ **2064** mm

Area of tension steel at shear perimeter $A_{s\_ten} = (k_y \times (p_x + (k_x \times k \times d)) \times A_{sy\_ten}) + (k_x \times (p_y + (k_y \times k \times d)) \times A_{sx\_ten})$

$A_{s\_ten} =$ **758** mm^2

Design concrete shear stress

$v_c=(\min(f_{cu},40)/25)^{1/3}\times 0.79\times \min(100\times A_{s\_ten}/(u\times d),3)^{1/3}\times \max(400/d,1)^{1/4}/1.25$

$v_c =$ **0.509** N/mm^2

Nominal design shear stress at perimeter $v = V_{eff} / (u \times d) =$ **0.392** N/mm^2

$v < v_c$ no shear reinforcement required

Penultimate edge column B1

Design shear transferred to column $V_t = (1.05 \times Span_x) \times ((0.45 \times Span_y) + e_y) \times N_{ult} =$ **119** kN

Design effective shear transferred to column $V_{eff} = 1.4 \times V_t =$ **166** kN

Area of tension steel in x-direction $A_{sx\_ten} = A_{sx1e} =$ **523** mm^2/m

Area of tension steel in y-direction $A_{sy\_ten} = A_{sy\_edge} =$ **349** mm^2/m

Column perimeter $u_c = l_x + (2 \times l_{y1}) =$ **900** mm

Average effective depth of reinforcement $d = h - c' - \phi_p =$ **194** mm

Maximum allowable shear stress $v_{max} = \min(0.8 \times \sqrt{(f_{cu})}, 5) =$ **5.000** N/mm^2

Design shear stress at column perimeter $v_0 = V_{eff} / (u_c \times d) =$ **0.953** N/mm^2

PASS - Maximum concrete shear stress not exceeded at column perimeter

Shear reinforcement at a perimeter of 1.50d - (291 mm)

Length of shear perimeter $u = u_c + (2 \times (k_x \times k_y) \times k \times d) =$ **2064** mm

Area of tension steel at shear perimeter $A_{s\_ten} = (k_y \times (p_x + (k_x \times k \times d)) \times A_{sy\_ten}) + (k_x \times (p_y + (k_y \times k \times d)) \times A_{sx\_ten})$

$A_{s\_ten} =$ **926** mm^2

Design concrete shear stress

$v_c=(\min(f_{cu},40)/25)^{1/3}\times 0.79\times \min(100\times A_{s\_ten}/(u\times d),3)^{1/3}\times \max(400/d,1)^{1/4}/1.25$

$v_c =$ **0.544** N/mm^2

Nominal design shear stress at perimeter $v = V_{eff} / (u \times d) =$ **0.416** N/mm^2

$v < v_c$ no shear reinforcement required

Penultimate central column B2

Design shear transferred to column $V_t = (1.05 \times Span_x) \times (1.05 \times Span_y) \times N_{ult} =$ **256** kN

Design effective shear transferred to column $V_{eff} = 1.15 \times V_t =$ **294** kN

Area of tension steel in x-direction $A_{sx\_ten} = A_{sx1e} =$ **523** mm^2/m

Area of tension steel in y-direction $A_{sy\_ten} = A_{sy1e} =$ **448** mm^2/m

Column perimeter $u_c = 2 \times (l_x + l_y) =$ **1200** mm

Average effective depth of reinforcement $d = h - c' - \phi_p =$ **194** mm

Maximum allowable shear stress $v_{max} = \min(0.8 \times \sqrt{(f_{cu})}, 5) =$ **5.000** N/mm^2

Design shear stress at column perimeter $v_0 = V_{eff} / (u_c \times d) =$ **1.265** N/mm^2

PASS - Maximum concrete shear stress not exceeded at column perimeter

Shear reinforcement at a perimeter of 1.50d - (291 mm)

Length of shear perimeter $u = u_c + (2 \times (k_x \times k_y) \times k \times d) = \mathbf{3528}$ mm

Area of tension steel at shear perimeter $A_{s\_ten} = (k_y \times (p_x + (k_x \times k \times d)) \times A_{sy\_ten}) + (k_x \times (p_y + (k_y \times k \times d)) \times A_{sx\_ten})$

$A_{s\_ten} = \mathbf{1713}$ mm^2

Design concrete shear stress

$v_c=(\min(f_{cu},40)/25)^{1/3}\times0.79\times\min(100\times A_{s\_ten}/(u\times d),3)^{1/3}\times\max(400/d,1)^{1/4}/1.25$

$v_c = \mathbf{0.558}$ N/mm^2

Nominal design shear stress at perimeter $v = V_{eff} / (u \times d) = \mathbf{0.430}$ N/mm^2

$v < v_c$ no shear reinforcement required

Internal central column B3

Design shear transferred to column $V_t = (1.05 \times Span_x) \times Span_y \times N_{ult} = \mathbf{244}$ kN

Design effective shear transferred to column $V_{eff} = 1.15 \times V_t = \mathbf{280}$ kN

Area of tension steel in x-direction $A_{sx\_ten} = A_{sx1i} = \mathbf{502}$ mm^2/m

Area of tension steel in y-direction $A_{sy\_ten} = A_{sye} = \mathbf{349}$ mm^2/m

Column perimeter $u_c = 2 \times (l_x + l_y) = \mathbf{1200}$ mm

Average effective depth of reinforcement $d = h – c' - \phi_p = \mathbf{194}$ mm

Maximum allowable shear stress $v_{max} = \min(0.8 \times \sqrt{(f_{cu})}, 5) = \mathbf{5.000}$ N/mm^2

Design shear stress at column perimeter $v_0 = V_{eff} / (u_c \times d) = \mathbf{1.204}$ N/mm^2

PASS - Maximum concrete shear stress not exceeded at column perimeter

Shear reinforcement at a perimeter of 1.50d - (291 mm)

Length of shear perimeter $u = u_c + (2 \times (k_x \times k_y) \times k \times d) = \mathbf{3528}$ mm

Area of tension steel at shear perimeter $A_{s\_ten} = (k_y \times (p_x + (k_x \times k \times d)) \times A_{sy\_ten}) + (k_x \times (p_y + (k_y \times k \times d)) \times A_{sx\_ten})$

$A_{s\_ten} = \mathbf{1501}$ mm^2

Design concrete shear stress

$v_c=(\min(f_{cu},40)/25)^{1/3}\times0.79\times\min(100\times A_{s\_ten}/(u\times d),3)^{1/3}\times\max(400/d,1)^{1/4}/1.25$

$v_c = \mathbf{0.534}$ N/mm^2

Nominal design shear stress at perimeter $v = V_{eff} / (u \times d) = \mathbf{0.410}$ N/mm^2

$v < v_c$ no shear reinforcement required

Internal edge column C1

Design shear transferred to column $V_t = Span_x \times ((0.45 \times Span_y) + e_y) \times N_{ult} = \mathbf{113}$ kN

Design effective shear transferred to column $V_{eff} = 1.4 \times V_t = \mathbf{158}$ kN

Area of tension steel in x-direction $A_{sx\_ten} = A_{sxe} = \mathbf{392}$ mm^2/m

Area of tension steel in y-direction $A_{sy\_ten} = A_{sy\_edge} = \mathbf{349}$ mm^2/m

Column perimeter $u_c = l_x + (2 \times l_{y1}) = \mathbf{900}$ mm

(Library item: Flat slab shear map C1) Average effective depth of reinforcement $d = h – c' - \phi_p = \mathbf{194}$ mm

Maximum allowable shear stress $v_{max} = \min(0.8 \times \sqrt{(f_{cu})}, 5) = \mathbf{5.000}$ N/mm^2

Design shear stress at column perimeter $v_0 = V_{eff} / (u_c \times d) = \mathbf{0.908}$ N/mm^2

PASS - Maximum concrete shear stress not exceeded at column perimeter

Shear reinforcement at a perimeter of 1.50d - (291 mm)

Length of shear perimeter $u = u_c + (2 \times (k_x \times k_y) \times k \times d) = \mathbf{2064}$ mm

Area of tension steel at shear perimeter $A_{s\_ten} = (k_y \times (p_x + (k_x \times k \times d)) \times A_{sy\_ten}) + (k_x \times (p_y + (k_y \times k \times d)) \times A_{sx\_ten})$

$A_{s\_ten} = \mathbf{771}$ mm^2

Design concrete shear stress

$v_c=(\min(f_{cu},40)/25)^{1/3}\times0.79\times\min(100\times A_{s\_ten}/(u\times d),3)^{1/3}\times\max(400/d,1)^{1/4}/1.25$

$v_c = \mathbf{0.512}$ N/mm^2

Nominal design shear stress at perimeter $v = V_{eff} / (u \times d) =$ **0.396** N/mm^2

$v < v_c$ no shear reinforcement required

Internal central column C2

Design shear transferred to column $V_t = Span_x \times (1.05 \times Span_y) \times N_{ult} =$ **244** kN

Design effective shear transferred to column $V_{eff} = 1.15 \times V_t =$ **280** kN

Area of tension steel in x-direction $A_{sx\_ten} = A_{sxe} =$ **392** mm^2/m

Area of tension steel in y-direction $A_{sy\_ten} = A_{sy1i} =$ **523** mm^2/m

Column perimeter $u_c = 2 \times (l_x + l_y) =$ **1200** mm

Average effective depth of reinforcementd $= h – c' - \phi_p =$ **194** mm

Maximum allowable shear stress $v_{max} = \min(0.8 \times \sqrt{(f_{cu})}, 5) =$ **5.000** N/mm^2

Design shear stress at column perimeter$v_0 = V_{eff} / (u_c \times d) =$ **1.204** N/mm^2

PASS - Maximum concrete shear stress not exceeded at column perimeter

Shear reinforcement at a perimeter of 1.50d - (291 mm)

Length of shear perimeter $u = u_c + (2 \times (k_x \times k_y) \times k \times d) =$ **3528** mm

Area of tension steel at shear perimeter $A_{s\_ten} = (k_y \times (p_x + (k_x \times k \times d)) \times A_{sy\_ten}) + (k_x \times (p_y + (k_y \times k \times d)) \times A_{sx\_ten})$

$A_{s\_ten} =$ **1614** mm^2

Design concrete shear stress

$v_c=(\min(f_{cu},40)/25)^{1/3}\times0.79\times\min(100\times A_{s\_ten}/(u\times d),3)^{1/3}\times\max(400/d,1)^{1/4}/1.25$

$v_c =$ **0.547** N/mm^2

Nominal design shear stress at perimeter $v = V_{eff} / (u \times d) =$ **0.410** N/mm^2

$v < v_c$ no shear reinforcement required

Internal column C3

Design shear transferred to column $V_t = Span_x \times Span_y \times N_{ult} =$ **232** kN

Design effective shear transferred to column $V_{eff} = 1.15 \times V_t =$ **267** kN

Area of tension steel in x-direction $A_{sx\_ten} = A_{sxi} =$ **392** mm^2/m

Area of tension steel in y-direction $A_{sy\_ten} = A_{syi} =$ **392** mm^2/m

Column perimeter $u_c = 2 \times (l_x + l_y) =$ **1200** mm

Average effective depth of reinforcementd $= h – c' - \phi_p =$ **194** mm

Maximum allowable shear stress $v_{max} = \min(0.8 \times \sqrt{(f_{cu})}, 5) =$ **5.000** N/mm^2

Design shear stress at column perimeter$v_0 = V_{eff} / (u_c \times d) =$ **1.147** N/mm^2

PASS - Maximum concrete shear stress not exceeded at column perimeter

Shear reinforcement at a perimeter of 1.50d - (291 mm)

Length of shear perimeter $u = u_c + (2 \times (k_x \times k_y) \times k \times d) =$ **3528** mm

Area of tension steel at shear perimeter $A_{s\_ten} = (k_y \times (p_x + (k_x \times k \times d)) \times A_{sy\_ten}) + (k_x \times (p_y + (k_y \times k \times d)) \times A_{sx\_ten})$

$A_{s\_ten} =$ **1383** mm^2

Design concrete shear stress

$v_c=(\min(f_{cu},40)/25)^{1/3}\times0.79\times\min(100\times A_{s\_ten}/(u\times d),3)^{1/3}\times\max(400/d,1)^{1/4}/1.25$

$v_c =$ **0.520** N/mm^2

Nominal design shear stress at perimeter $v = V_{eff} / (u \times d) =$ **0.390** N/mm^2

$v < v_c$ no shear reinforcement required

CURTAILMENT OF REINFORCEMENT

Internal column

Radius of circular yield line $r = (l_x \times l_y / \pi)^{1/2} \times (1.05 \times Span_x \times 1.05 \times Span_y / (l_x \times l_y))^{1/3}$ = **1023** mm

Minimum curtailment length in x-direction l_{int_x} = Max(r + 12 × D, 0.25 × $Span_x$) = **1143** mm

Minimum curtailment length in y-direction l_{int_y} = Max(r + 12 × D, 0.25 × $Span_y$) = **1143** mm

Corner column

Radius of yield line $r = (l_{x1} \times l_y / \pi)^{1/2} \times ((0.45 \times Span_x + e_x) \times (0.45 \times Span_y + e_y)/ (l_{x1} \times l_y))^{1/3}$

r = **611** mm

Minimum curtailment length in x-direction l_{corner_x} = Max(r + 12 × D, 0.2 × $Span_x$) = **900** mm

Minimum curtailment length in y-direction l_{corner_y} = Max(r + 12 × D, 0.2 × $Span_y$) = **800** mm

Edge columns

Radius of yield line in x-direction $r = (l_{x1} \times l_y / \pi)^{1/2} \times ((0.45 \times Span_x + e_x) \times (1.05 \times Span_y) / (l_{x1} \times l_y))^{1/3}$

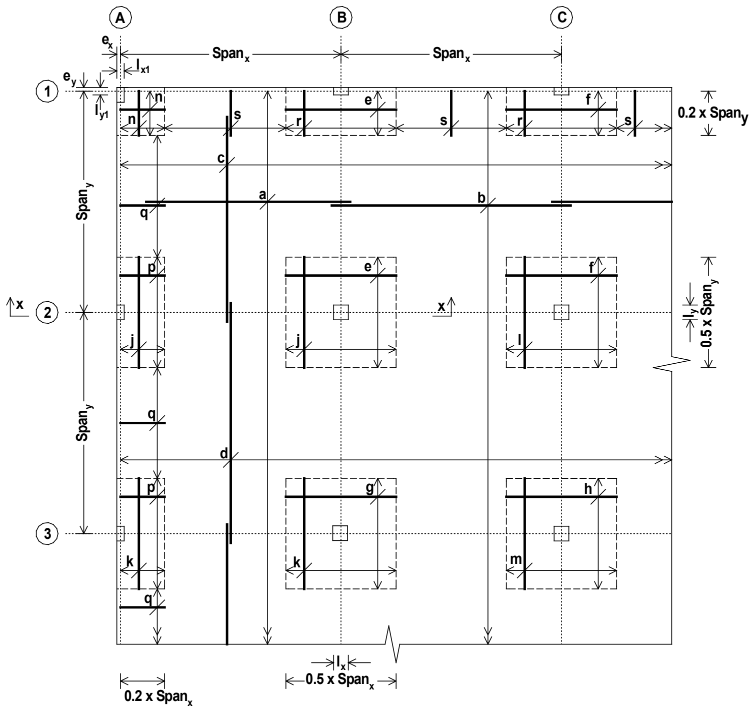
r = **790** mm

Minimum curtailment length in x-direction l_{edge_x} = Max(r + 12 × D, 0.2 × $Span_x$) = **910** mm

Radius of yield line in y-direction $r = (l_x \times l_{y1} / \pi)^{1/2} \times ((0.45 \times Span_y + e_y) \times (1.05 \times Span_x) / (l_x \times l_{y1}))^{1/3}$

r = **792** mm

Minimum curtailment length in y-direction l_{edge_y} = Max(r + 12 × D, 0.2 × $Span_y$) = **912** mm

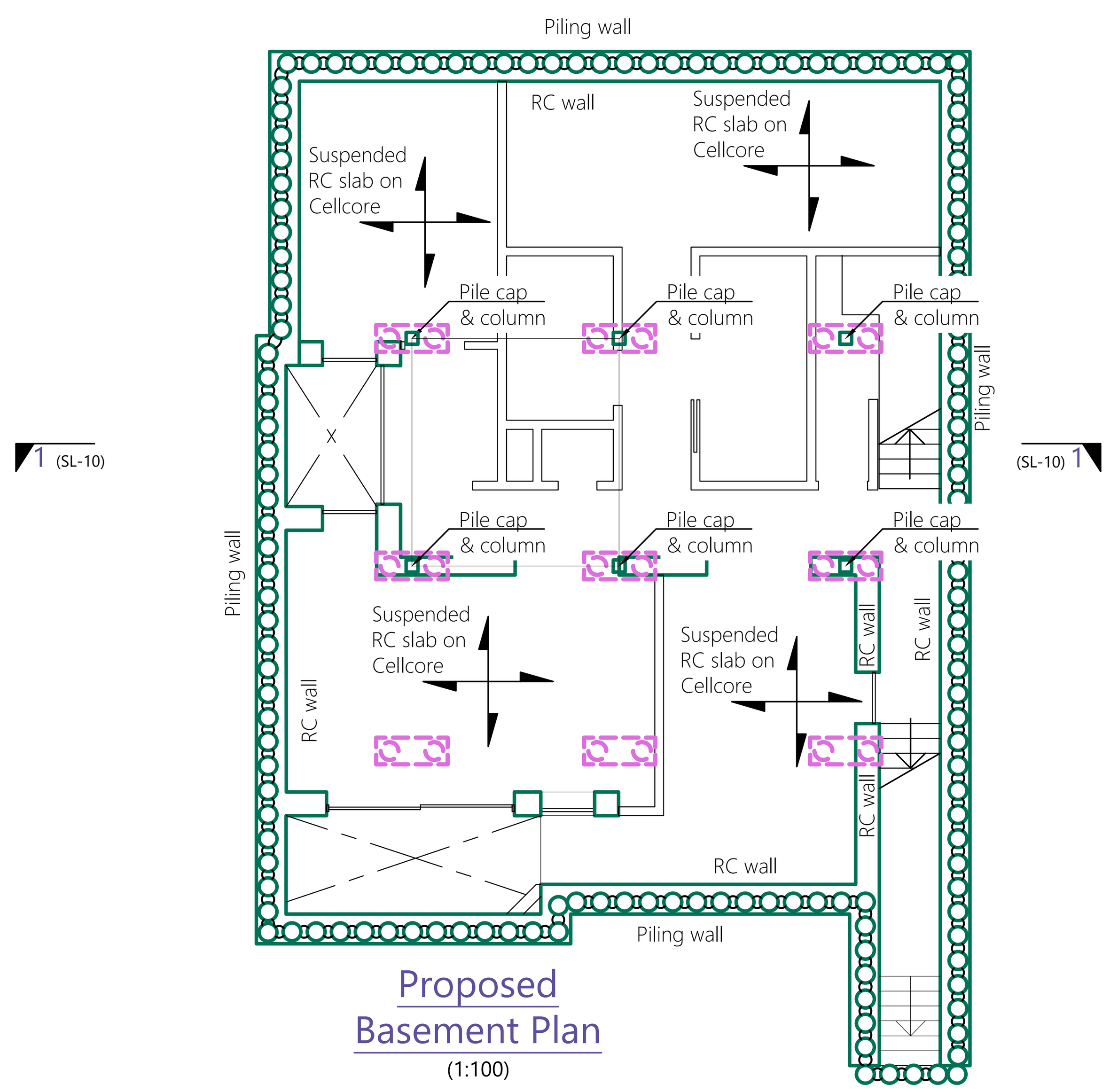


When the effective span in the x direction, L_x, is greater than the effective span in the y direction, L_y, the reinforcement in the outer layer is assumed to be that in the x direction otherwise it is assumed to be that in the y direction.

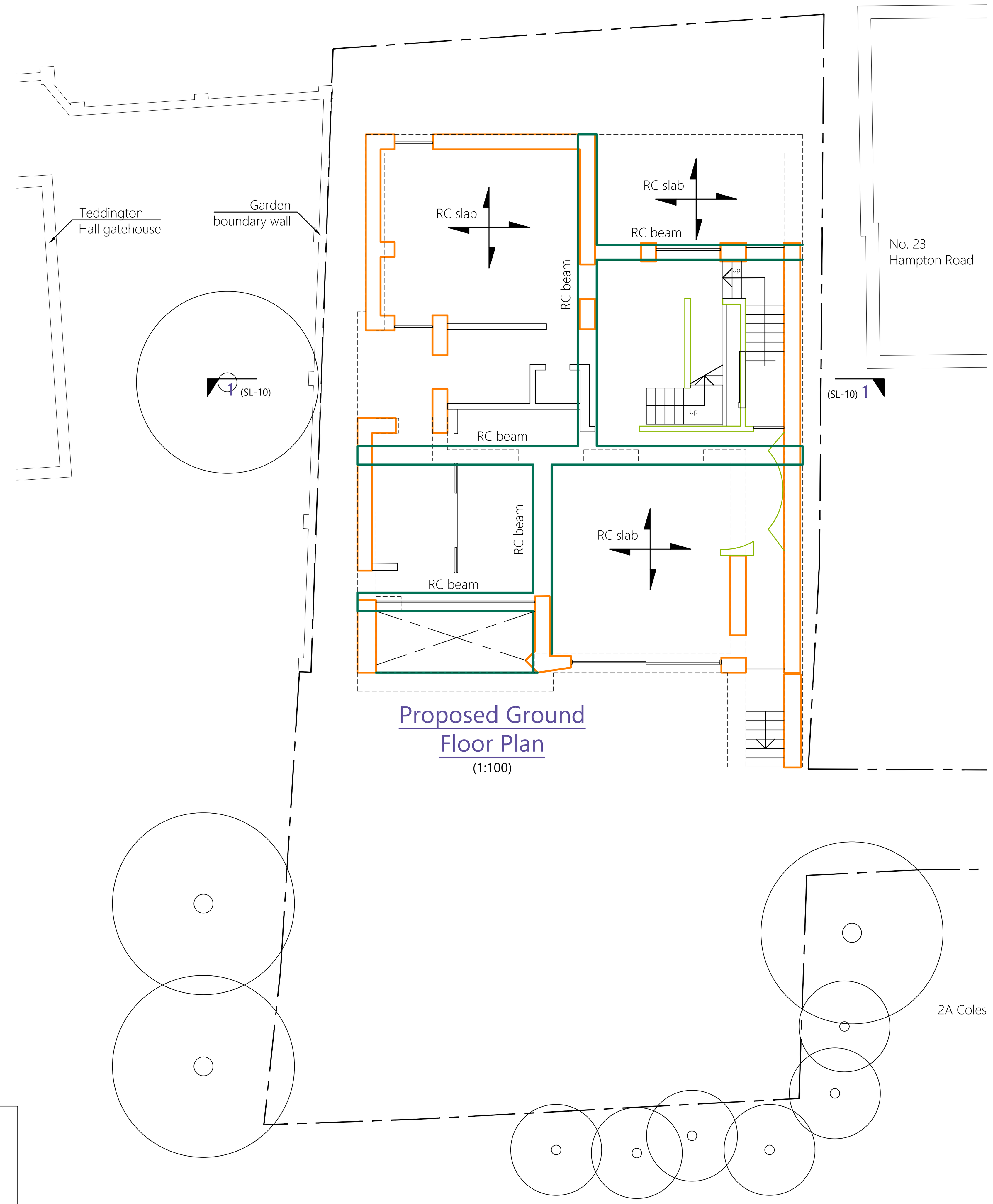
REINFORCEMENT KEY

a = 10 dia bars @ 200 centres - (392 mm^2/m) **b = 10 dia bars @ 200 centres - (392 mm^2/m)**
c = 10 dia bars @ 200 centres - (392 mm^2/m) **d = 10 dia bars @ 200 centres - (392 mm^2/m)**
e = 10 dia bars @ 150 centres - (523 mm^2/m) **f = 10 dia bars @ 200 centres - (392 mm^2/m)**
g = 8 dia bars @ 100 centres - (502 mm^2/m) **h = 10 dia bars @ 200 centres - (392 mm^2/m)**
j = 10 dia bars @ 175 centres - (448 mm^2/m) **k = 10 dia bars @ 225 centres - (349 mm^2/m)**
l = 10 dia bars @ 150 centres - (523 mm^2/m) **m = 10 dia bars @ 200 centres - (392 mm^2/m)**
n = 10 dia bars @ 225 centres - (349 mm^2/m)
p = 10 dia bars @ 200 centres - (392 mm^2/m) **q = 10 dia bars @ 200 centres - (392 mm^2/m)**
r = 10 dia bars @ 225 centres - (349 mm^2/m) **s = 10 dia bars @ 225 centres - (349 mm^2/m)**
Distribution bars = 10 dia bars @ 200 centres - (393 mm^2/m)

Appendix C – Structural Plans & Method Statement

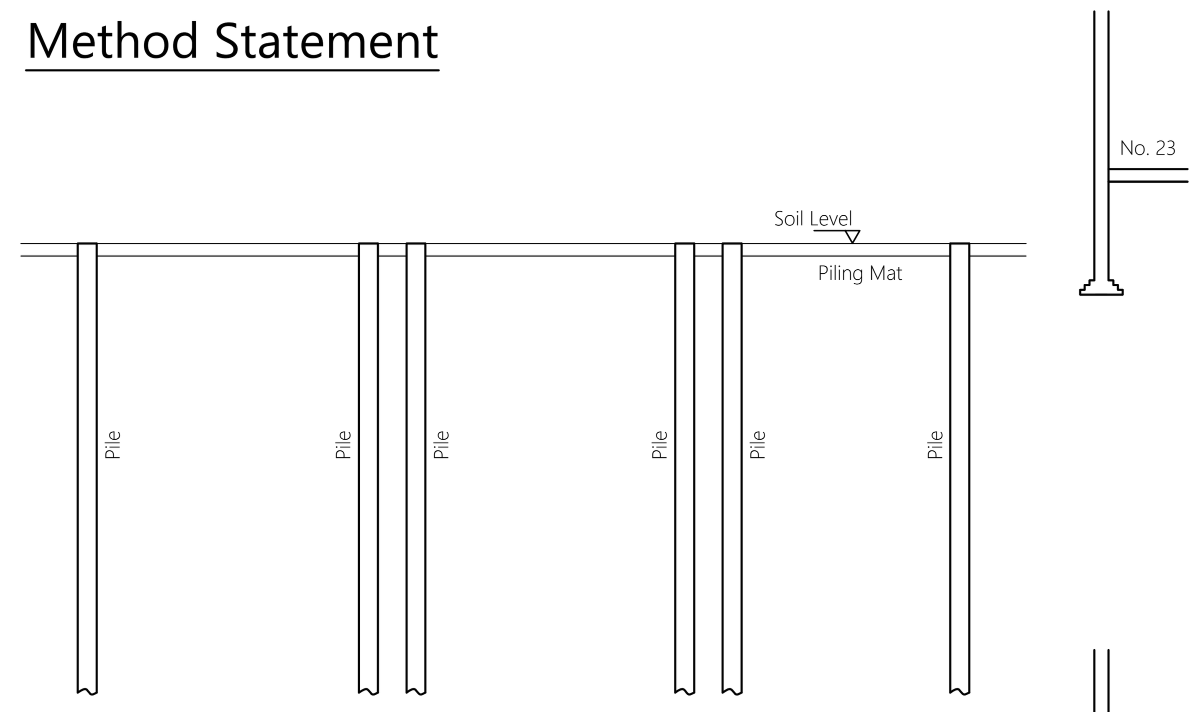


Proposed Basement Plan
(1:100)



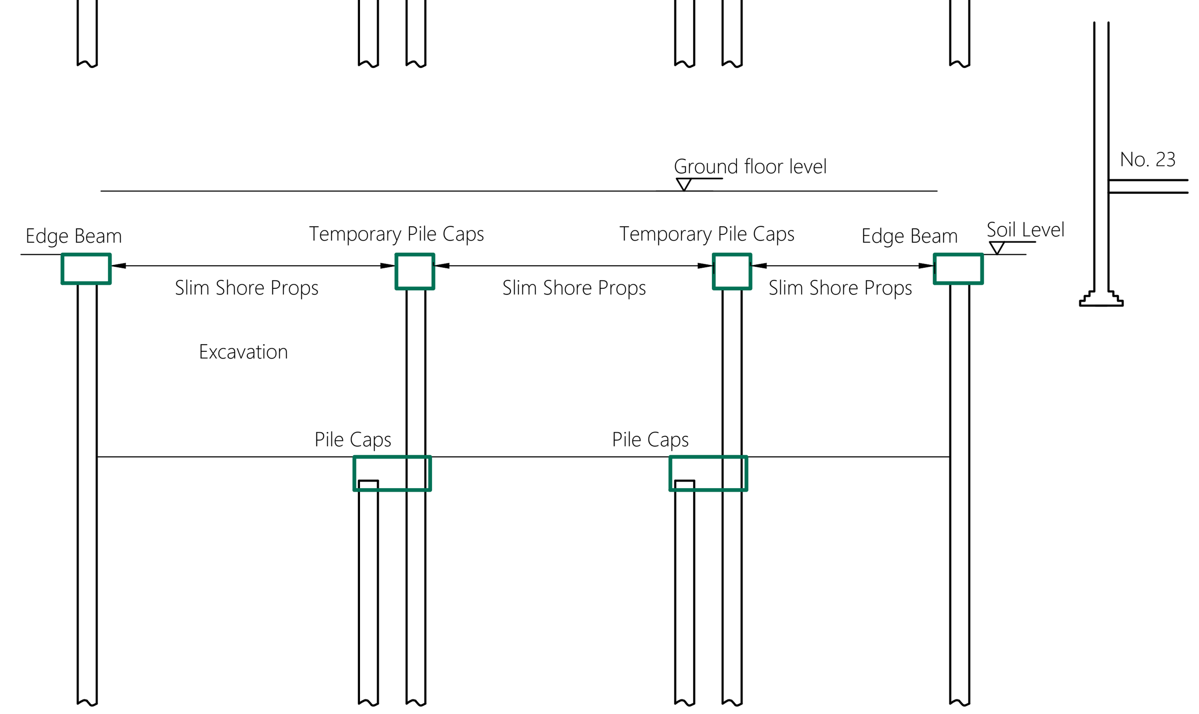
Proposed Ground Floor Plan
(1:100)

Method Statement



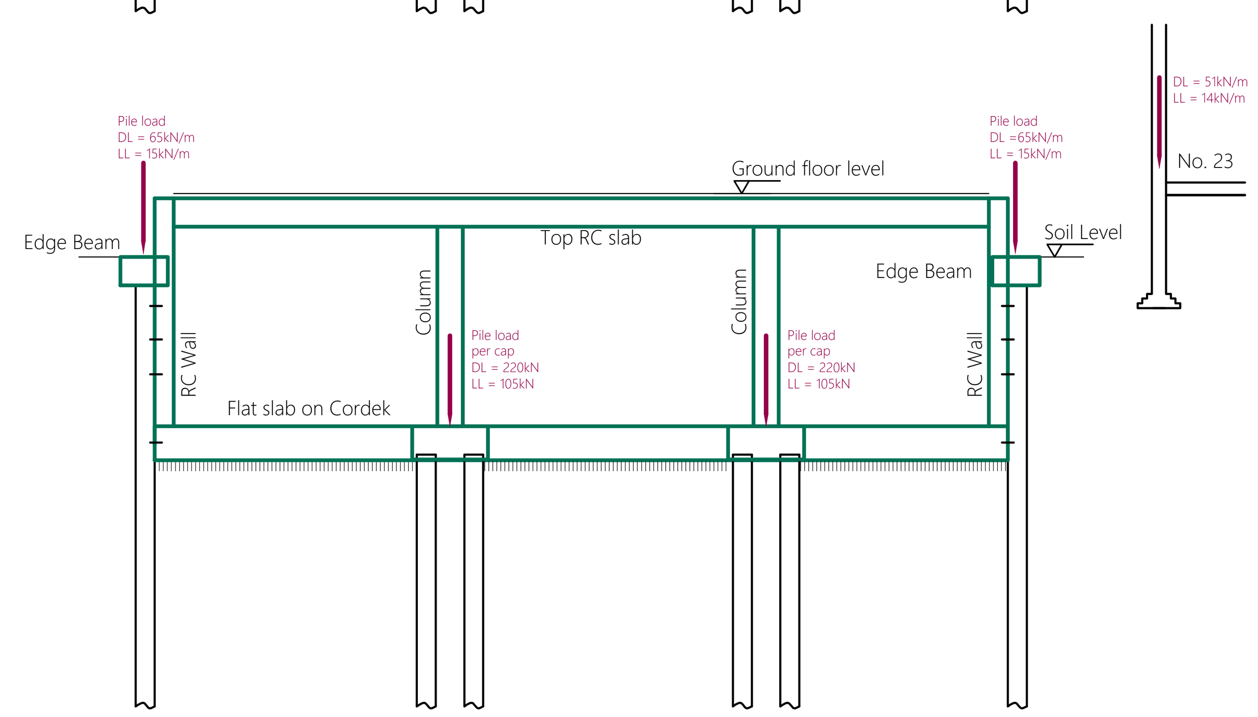
Phase 1

1. Install Piles



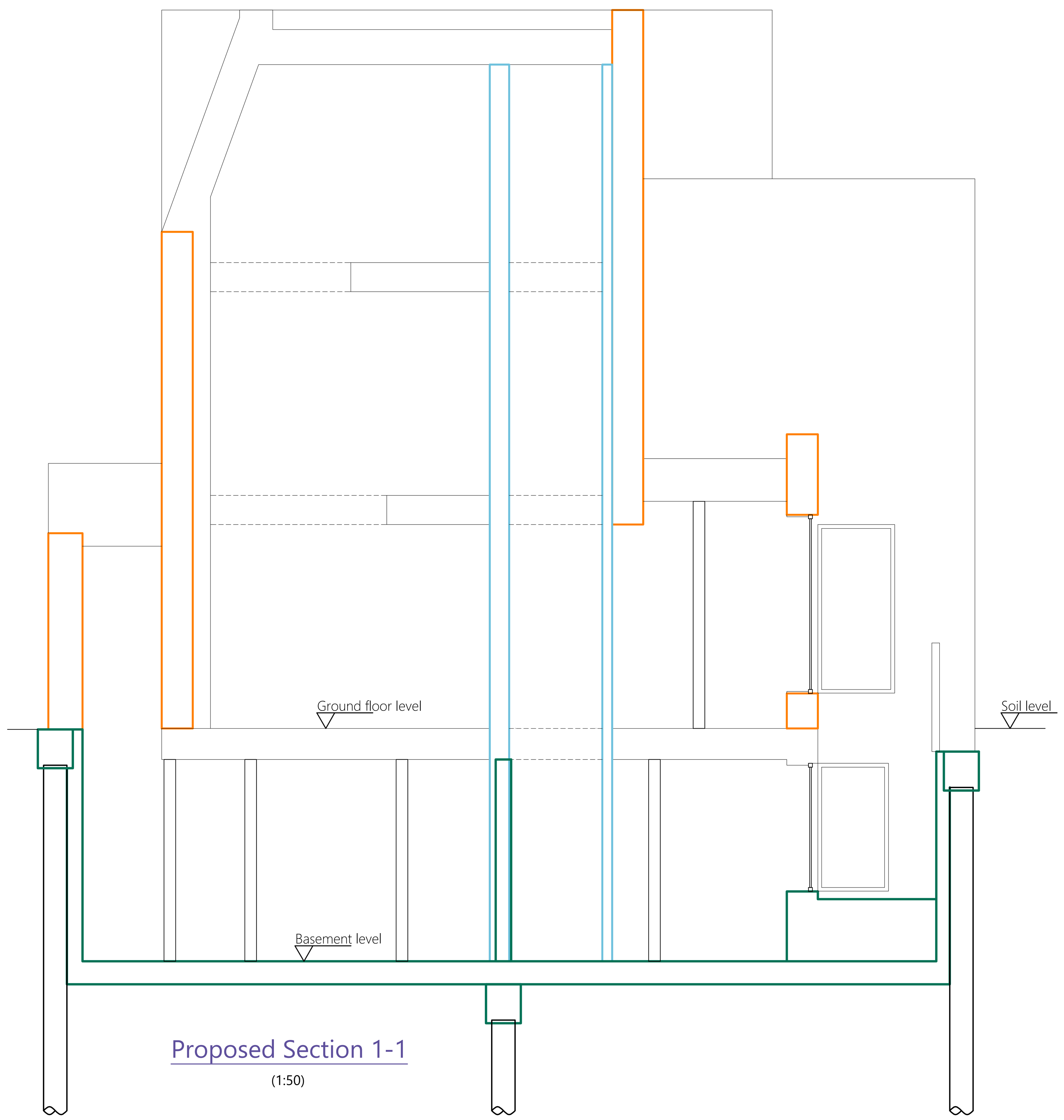
Phase 2

1. Bulk excavate 600mm of soil
2. Cut Edge Piles as required, cast Edge Beams, temporary Pile Caps and Slim Shore Props between them
3. Continue the excavation to the desired level
4. Cut middle piles as required, install Pile caps

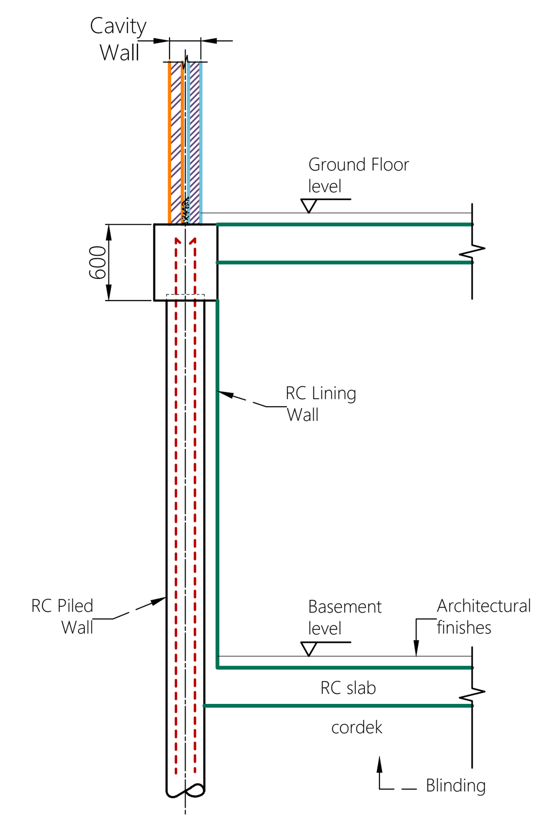


Phase 3

1. Cast min. 50mm thick concrete blinding and Place Cordek anti-heave protection.
2. Install Flat Slab, this to be anchored onto the piles.
3. Install RC Walls
4. Install Columns
5. Install Flat Slab at ground floor level (Slab to be cast sequentially with local removal of top propping).



Proposed Section 1-1
(1:50)



Typical RC Pile Wall Section
(1:50)

Rev	Date	By	Amendments
-	??/08/23	JH	First issue

CRØFT STRUCTURAL+ CIVIL
r/o 60 Saxon Rd,
London, SE25 5EH.
020 8684 4744
www.croftse.co.uk

Simon Kinsman

23A Hampton Road
Teddington TW11 0JN

Proposed Plans

Issued for **INFORMATION ONLY**

Job Number	Dwg Number	Rev
230705	SL-10	-
By	**Approved by**	**Scale**
JH	EP	1:50 @ A1