Job Number: 190611
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Basement Impact Assessment

Property Details:
1 Dunolie Road
London
NW5 2XN

Rev	Date	Author	Checker	Comment
-	09-08-19	EP	GW	Draft issue
1	06/11/19	EP	GW	Final Issue

Contents

Basement Impact Assessment for Site Address

1. Non Technical Summary

1.1. Existing Property, Site & Neighbouring Sites

The site is located adjacent to 1 Dunollie Road which is an end of terrace property. The existing site is occupied by a single storey garage which is used as storage at present. This appears to be a temporary structure. The left and rear of the site is

1.2. Proposed Development

The proposed basement will occupy the majority of the site and will be part of a three storey house (basement, ground and first floor levels). The basement will also benefit from a lightwell to the front of the property. The existing single garage will be demolished.

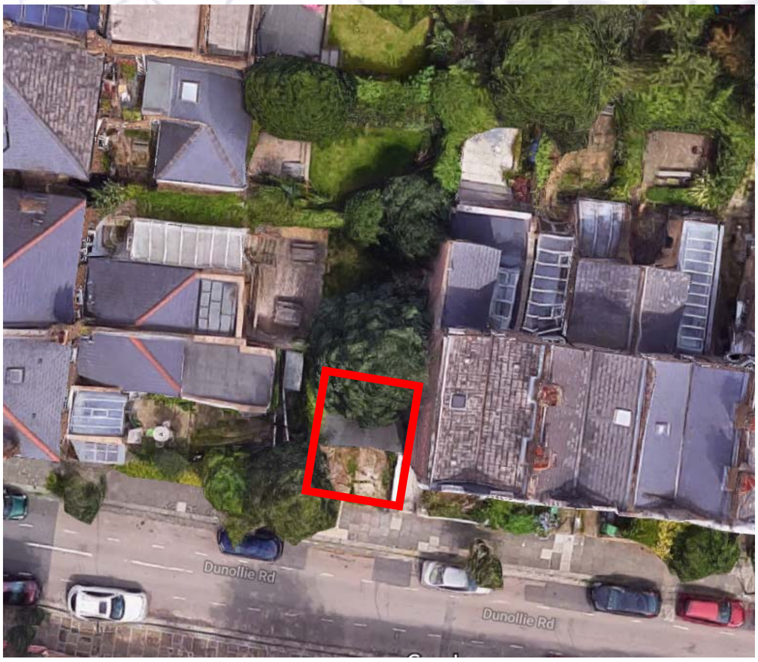


Figure 1: Map / Aerial view with approx. site area indicated in red box

1.3. Geology and Land Stability

This Summary has been taken from the Geologists report.

The BIA report considered relevant information from existing sources included in the 'Guidance for subterranean development' produced for the London Borough of Camden' (November 2010) and a Groundsure Enviro/Geo insight Report with historical maps and BGS records.

A ground investigation at the site was undertaken by Maund Geo-Consulting Ltd in June 2019 which comprised one borehole and two hand dug trial pits to expose party wall footings. The borehole (BH01) was drilled to 8.45m below ground level (bgl), while the trial pits (TP01 and TP02) were excavated to a depth of 0.5 and 0.75m bgl respectively.

The ground investigation confirmed the ground conditions as a thin layer of Made Ground of sandy gravel composition to a depth of approximately 0.4m which overlies firm to stiff silty clay of the London Clay Formation to a depth of at least 8.45 m bgl. Groundwater was not encountered during the ground investigation or from subsequent monitoring.

An assessment of land stability has been made from the excavation and construction of the basement. It has been calculated that heave in the centre of the basement is not expected to exceed 17 mm resulting from the excavation and construction. The foundation formation will be able to accommodate an imposed load from the retaining wall of 96 kPa with net settlement of < 25 mm.

A ground movement assessment was undertaken in relation to the proposed basement. It was determined that there is low impact from the basement construction with a Damage Assessment Category less than 1 or 'very slight' for 1 Dunollie Road. Other adjacent properties of 3 Dunollie Road and 22D Lady Margaret Road will not experience any significant impact.

1.4. Hydro-geology

This Summary has been taken from the Hydro-geologists report.

An assessment of hydrogeology has shown that the strata underlying site is considered non-productive strata of very low permeability and is not designated as an aquifer within Environment Agency (EA) guidelines. It is not anticipated that the development will have any significant impact on groundwater, which is over 5.88m below ground level.

1.5. Drainage, Surface Water & Flooding

The BIA has identified

- The construction of the basement will not have any significant impacts on the Surface water.

- Further assessment of the site and the development concluded that the basement will not increase the risk of flooding and further assessment of surface water and drainage is not necessary.
- The risk of flooding from excess surface water is not considered significant. There is a risk of flooding due to the failure of the pumping system but this can be reduced to acceptable levels with appropriate design and installation measures.

1.5.1. Drainage & SUDS Summary

There is not significant increase in the discharge of surface water into the existing sewer system. The use of complex SUDS features is therefore not considered applicable to a development of this scale. However, Croft has proposed the use of permeable paving to minimise the amount of surface water discharge into the sewer. This will act as a storage area for surface water allowing the water to recharge the ground water in the area.

Where basements below a garden are present, then a soil band will be provided. This will act as a storage area for surface water allowing the water to recharge the ground water in the area.

2. Report Authors and Qualifications

2.1. Geology, Land stability and Hydro Geology

To undertake the Land stability Geology and Hydro Geology Croft Structural Engineers has employed a suitably qualified professional, Mr Julian Maund of Maund Geo-Consulting Ltd BSc PhD CEng MIMMM CGeol FGS

2.2. Surface water and Flooding

Phil Henry
BEng MEng MICE

Chris Tomlin
MEng CEng MIStructE

3. Introduction

3.1. Site & location

The site is located

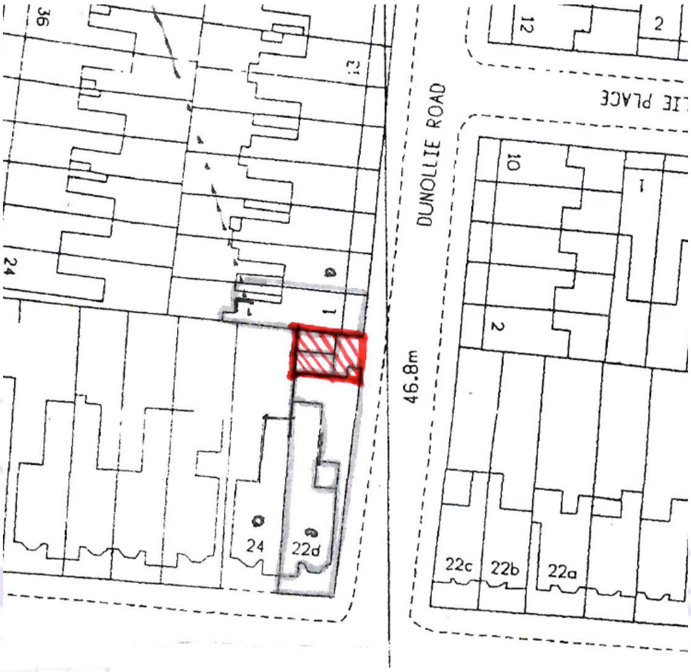


Figure 2: Plan view of site (approx. area outlined in red) and the surrounding properties

For further information refer to the Desk Study Section.

3.2. Proposed works

Refer to the Architects plans and Croft's Structural Scheme Appendix D to this BIA. The engineering statement & temporary works construction sequence is in the Appendices.

The proposed works involve the demolition of the existing garage and the construction of a three-storey property which will include a single storey basement, ground and first floor levels. The basement will have a lightwell to the front of the property.

4. Desk Study & Walk over Survey

4.1. General Desk Study

4.1.1. Site History

The Historical Map Review is located in the Geologist Desk Study.

4.1.2. Listed buildings

The existing building is not listed. Data from Historic England shows that there are no listed buildings close by.

Figure 3: historic England map

The site is within a conservation area as noted on the camden map below

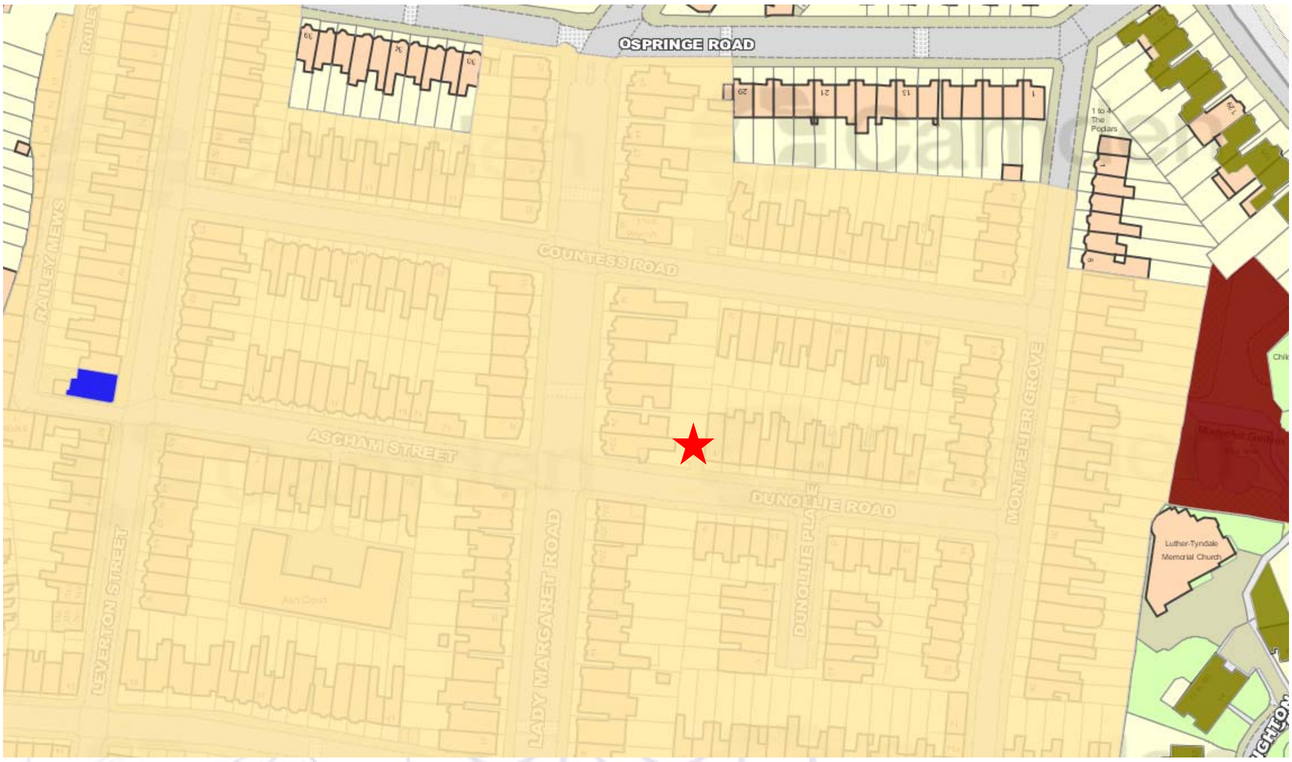


Figure 4: conservation map (www.camden.gov.uk)

4.1.3. London Under Ground and Network Rail Infrastructures

The site is more than 500m away from the nearest national rail line and the nearest subterranean train line. These are unlikely to be affected by the new basement.

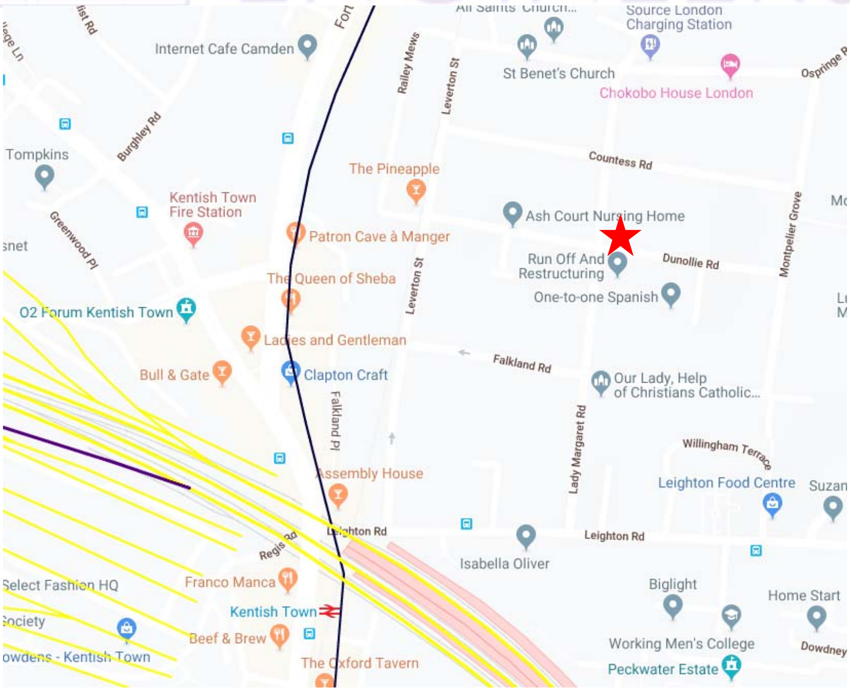


Figure 5: Extract from LUL map showing proximity of rail lines

4.1.4. Highways

The site is within 5m of the public highway. The front lightwell is within 5m of the pavement.

4.1.5. UK Power Network

There are no significant items of electrical infrastructure (such as pylons, substations or tunnels) in the immediate vicinity.

4.1.6. Utility Search

A utility search has been completed and is attached in the Appendices.

4.2. Walk Over Survey

A Structural Engineer from Croft Structural Engineers visited the site on the 21st June 2019.

4.2.1. Site and Existing Property

The existing site is occupied by a garage which is used as storage. The site to the left and rear is gardens to the adjacent properties. The existing garage will be demolished to allow for the new structure to be erected on site. To the front of the site there is the public pavement.

4.2.2. Proximity of Trees

The proposed basement may be within a tree protection zone of a Silver Birch which is located within the adjacent garden of 22 Lady Margaret Road. The distance from the tree trunk to the basement is at a minimum of 1.5m. An arboricultural survey is being obtained. Refer to Arboriculturalist report

There is an existing Sycamore tree stump on the boundary to the rear of the site with No. 24 Lady Margaret Road has been approved for removal.

4.2.3. Adjacent Properties

4.2.3.1. Nos 22d Lady Margaret Road – Property to Left

Property age : mid-Victorian (~150 years old)
Property use : Residential
Number of storeys : 3 storeys
Current Basement: The neighbouring property does not have a basement. The planning portal indicates that there are no plans for a basement.

Figure 6 Neighbouring property

4.2.3.2. Nos 1 Dunollie Road – Property to Right

Property age : mid-Victorian (~150 years old)
Property use : Residential
Number of storeys : 3 storeys
Current Basement: The neighbouring property does not have a basement. The planning portal indicates that there are no plans for a basement.

Figure 7 Neighbouring property

4.3. Surface Water and Drainage Walk Over Survey

4.3.1. Hardstanding

The site is curently occupied by garages and a hardstanding.

Figure 8: front elevation of site showing existing garages

4.3.2. Site Drainage

The surface water down pipes are located on the right-hand side of the garages. The downpipes discharge directly into flowerbeds.

4.3.3. Surface Water

No areas of surface water in the form of ponds lakes, streams or rivers were noted on the site.

4.3.4. Summary Surface Water and Drainage Walk

A walk over survey has confirmed that there are no surface water features, either within or close to the site. The survey has also confirmed that the site is covered with hard surfaces. Rainwater from these surfaces is likely to flow in the direction of the slope of the surrounding area.

Refer to Hydrogeology, Land Stability and Ground Movement Assessment by Maund Geo-consulting for additional information on issues relating to surface water and flooding.

4.4. Geology and Hydro geology : Ground Investigation

See MGC-FR-19-20-V1 Report. The ground investigation report, which has data from initial site investigations and data from subsequent monitoring, is available as a separate report.

A Ground investigation has been undertake to better determine the ground conditions. Water monitoring has been undertaken over time to investigate the Hydro geology.

5. Screening Stage

This stage identifies any areas for concern that should be investigated further.

5.1. Geology and Land Stability

See Report Completed by Maund Geo-Consulting

5.2. Hydro-geology

See Report Completed by Maund Geo-Consulting

5.3. Surface Flow and Flooding

Question 1: Is the site within the catchment of the pond chains on Hampstead Heath?

No. The site lies outside the areas denoted by Figure 14 of the GSD (extract shown below)

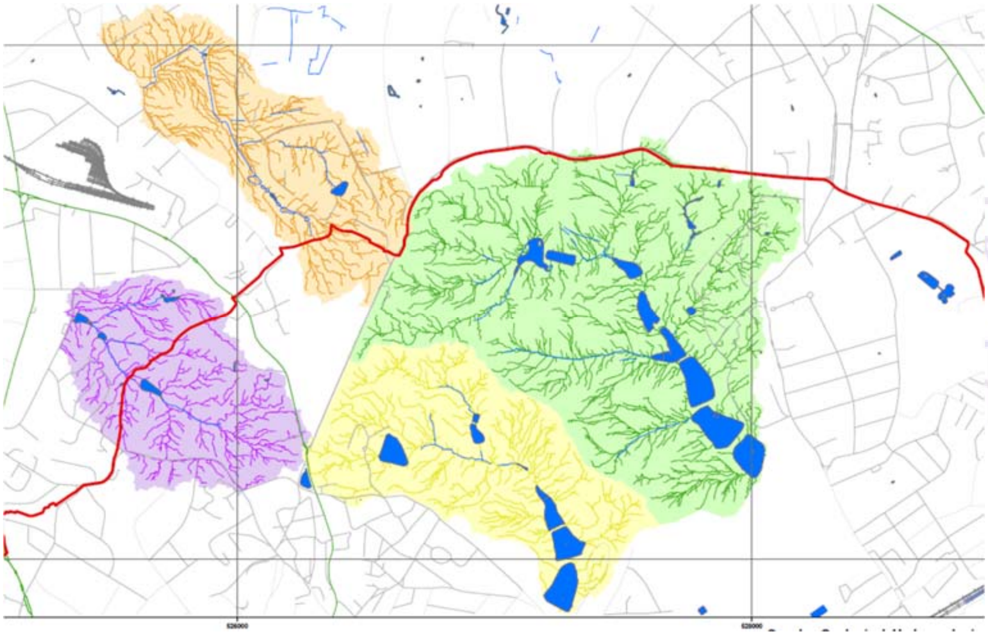


Figure 9: Extract from Figure 14 of the GSD (site lies to the south of the shaded areas)

Question 2. 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?

No – The surface water collects on hard surfaces of a domestic property. This will remain the same with the proposed development.

Question 3. Will the proposed basement development result in a change to the hard surfaced /paved external areas?

No – Currently the site is fully occupied by garages and hard-surfaced areas. This will remain the case with the proposed development.

Question 4. Will the proposed basement result in changes to the inflows (instantaneous and long term) of surface water being received by adjacent properties or downstream watercourses?

No – Currently the site is fully occupied by buildings and hard-surfaced areas. This will remain the case with the proposed development.

Question 5. Will the proposed basement result in changes to the quality of surface water being received by adjacent properties or downstream watercourses?

No. Collected surface water will be from building roofs and paving, as before. The quality of the water received downstream will therefore not change.

Question 6 : Is the site in an area identified to have surface water flood risk according to either the Local Flood Risk Management Strategy or the Strategic Flood Risk Assessment or is it at risk from flooding, for example because the proposed basement is below the static water level of nearby surface water feature?

The potential sources of flooding are summarised below:

Potential Source	**Potential Flood Risk at site?**	**Justification**
Fluvial flooding	No	EA Flood Mapping shows Flood Zone 1. Distance from nearest surface watercourse >1km
Tidal flooding	No	Site location is 'inland' and topography > 40mAOD.
Flooding from rising / high groundwater	No	The site is located on low permeability London Clay.
Surface water (pluvial) flooding	No	Site Address is not noted on the flooded street list and maps from 1975 or 2002
Flooding from infrastructure failure	No	Water-mains or sewers could potentially fail. However, these are owned by utility companies and are expected to have a high level of maintenance. The risk of flooding from these sources is therefore considered low.

Flooding from reservoirs, canals and other artificial sources	No	There are no reservoirs, canals or other artificial sources in the vicinity of the site that could give rise to a flood risk.

The answers to Questions 1-5 above indicate that the issues related to surface water flow and flooding are not significant. These questions therefore do not have to be carried forward to Scoping Stage.

Summary

In answering Question 6, a flood risk assessment is not considered necessary: the property is not on a street that has flooded in 1975 or 2002 and there are no risks to flooding that are greater than those inherent with all subterranean structures. However, the risks associated with infrastructure failure should be investigated further. The assessment, with regards to Surface Water Flow, should be carried forward to Scoping Stage.

6. Scoping Stage

6.1. Geology and Land Stability

See Report Completed by Maund Geo-Consulting

6.2. Hydro-geology

See Report Completed by Maund Geo-Consulting

6.3. Surface Flow and Flooding

6.3.1. Conceptual Model

The existing site is occupied by garages and hardstanding area. The garages will be demolished to give way for new basement and an above-ground structure. This does not significantly affect the surface water flow.

It is evident from the screening study that the only significant flood risk associated with the development is due to the failure of existing sewers in the vicinity of the site. These are inherent with all subterranean structures. No further study is required beyond the Scoping stage. However, the detailed design should consider how these risks can be mitigated. Proposals are presented in Section 9.

7. Site Investigation / Additional Assessments

Investigations for Land Stability and Hydrogeology are described within the BIA by Maund Geo-Consulting. This BIA proposed that an Arboricultural assessment should be carried out.

As mentioned previously, no further assessments are required for Surface Water and Flooding.

8. Construction Methodology and Engineering Statements

8.1. Outline Geotechnical Design Parameters

From the Geological report and soil investigation reasonably conservative geotechnical parameters have been determined, based on the soil investigation: design overall stability to K_a & K_p values.

K_a = 0.4217 , K_p = 2.3711

8.2. Hydro Static Pressure

Design temporary condition for water table level, If deeper than basement ignore.

Design permanent condition for water table level:

If deeper than existing, design reinforcement for water table at full basement depth to allow for local failure of water mains, drainage and storm water. Global uplift forces can be ignored when the water table is lower than the basement. BS8102 only indicates guidance.

8.2.1. Intended Use & Loadings

	UDL kN/m²	Concentrated Load kN
Domestic Single Dwellings	1.5	2.0

Below ground level, the reinforced concrete retaining walls are designed to carry the lateral loading applied from above.

The lateral earth pressure exerts a horizontal force on the retaining walls. The retaining walls will be checked for resistance to the overturning force this produces.

Lateral forces will be applied from:

- Soil loads
- Hydrostatic pressures
- Surcharge loading from behind the wall

These produce retaining wall thrust. This will be restrained by the opposing retaining wall.

8.2.1.1. Surcharge Loading

The following will be applied as surcharge loads to the front/ front lightwell retaining walls:

- 10kN/m² if within 45° of road
- 100kN point loads if under road or within 1.5m
- 5kN/m² if within 45° of Pavement

- Garden Surcharge 2.5kN/m^2 + 1 m of soil (if present above basement ceiling) 20kN/m^2
- Surcharge for adjacent property 1.5kN/m^2 + 4kN/m^2 for concrete ground bearing slab

Adjacent Properties:
All adjacent property footings within 45° to have additional geotechnical engineers' input. A line at 45º from the base of the neighbours' wall footing would be intersected by the basement retaining wall. This should be accounted for in the design.

8.3. Permanent Design Proposals

Reinforced concrete cantilevered retaining walls will form the new foundation of the property.

The design of the retaining walls was calculated using software by TEDDS. The software is specifically designed for retaining walls and ensures that the construction is kept to a limit to prevent damage to the adjacent property.

The overall stability of the walls is designed using K_a & K_p values, while the design of the wall structure uses K_0 values. This approach minimises the level of movement from the concrete affecting the adjacent properties.

The water table was recorded as low. The design of the walls considers long term scenarios. It is possible that a water main may break causing a local high water table. To account for this, the wall is designed for water 1m from the top of the wall.

The design also considers floatation as a risk. The design has accounted for the weight of the building and the uplift forces from the water. The weight of the building is greater than the uplift, resulting in a stable structure.

Appendix A shows the calculations of one of the most heavily loaded retaining walls. The most critical parameters have been used for this.

8.3.1. Temporary works

Walls are designed to be structurally stable with top and bottom propping. Temporary propping details will be required to be provided by the contractor and must be completed by a suitability qualified professional.

To demonstrate the feasibility of the works, a proposed basement construction sequence is in appendix E.

8.4. Ground Movement Assessment

See Geologists ground movement Assessment

8.5. Control of Construction Works

8.5.1. Control of Construction Works

A construction sequence has been formulated with Croft's experience of over 500 basements. The procedures described in this statement will mitigate the impacts that the construction of the basement will have on nearby properties.

To reduce the risk to the development:

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

With the measures listed above, the maximum level of cracking anticipated is 'Hairline' cracking. This can be repaired with normal decorative works. 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.

8.5.2. Noise and Nuisance Control

The contractor is to follow the good working practices and guidance laid down in the 'Considerate Constructors Scheme'.

The hours of working will be limited to those allowed; 8am to 5pm Monday to Friday and Saturday Morning 8am to 1pm.

None of the practices cause undue noise that one would typically expect from a construction site (a conveyor belt typically runs at around 70dB).

The site will be hoarded with 8' site hoarding to prevent access.

The hours of working will further be defined within the Party Wall Act.

The site is to be hoarded to minimise the level of direct noise from the site.

Working in the basement generally requires hand tools to be used. The level of noise generally will be no greater than that of digging of soil. The noise is reduced and muffled by the works being undertaken underground. The level of noise from basement construction works is lower than typical ground level construction due to this.

8.5.3. Monitoring

In order to safeguard the existing structures during underpinning and new basement construction, movement monitoring using total stations or similar is to be undertaken.

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

- Risk Assessment to determine level of monitoring
- Scope of Works
- Applicable standards
- Frequency of Monitoring
- Specification for Instrumentation
- Monitoring of Existing cracks
- Monitoring of movement
- Reporting

We would recommend that the monitoring frequency should follow:

Pre-construction: Monitored once.

During construction: Monitored after every pin is cast for first 4 no. pins to gauge effect of underpinning. If all is well, monitor after every other pin.

Post construction works: Monitored once.

Trigger values and contingency actions are noted in the table below. Monitoring locations are noted on the drawing which is included in the appendix F.

MOVEMENT		CATEGORY	ACTION
Vertical	Horizontal		
0mm-5mm	0-4mm	Green	No action required
5mm-8mm	4-6mm	AMBER	Detailed review of Monitoring: Check studs are OK and have not moved. Ensure site staff have not moved studs. If studs have moved reposition. Relevel to ensure results are correct and tolerance is not a concern. Inform Party Wall surveyors of amber readings. Double the monitoring for 2 further readings. If stable revert back. Carry out a local structural review and inspection.

			Preparation for the implementation of remedial measures should be required. Double number of lateral props
>8mm	>6mm	RED	Implement structural support as required; Cease works with the exception of necessary works for the safety and stability of the structure and personnel; Review monitoring data and implement revised method of works

Final trigger levels to be agreed with Party Wall Surveyor at Detailed Design Stage.

9. Basement Impact Assessment

9.1. Geology, Land stability and Hydro Geology

To undertake the Land stability Geology and Hydro Geology Croft Structural Engineers has employed a suitably qualified professional, Mr Julian Maund of Maund Geo-consulting. Refer to their report for the BIA on these items.

9.2. Surface Water & Flooding Assessment

As described previously, there are no impacts relating to surface water and flooding that need to be considered for further assessment. However, the planning requirements focus mainly on the potential for the development to cause flooding to other properties and assets; they do not assess the potential for flooding within the subject property.

There is a risk of flooding from failure of infrastructure, such as flooding due to unexpected failure of the drainage, water mains, etc. This risk is inherent in the construction of all subterranean structures. At detailed design stage, suitable features should be incorporated to reduce the hazards associated with this risk.

9.2.1. Internal Flooding: Mitigation Measures

To mitigate the risks associated with flooding, Croft would recommend the following mitigation measures:

- A pumping mechanism should be installed for the proposed basement. There is a likelihood that this may fail and allow excess water to accumulate. If this were to occur, the build-up of water would be gradual and noticeable before it becomes a significant life-threatening hazard.

- The pumping system should be a dual mechanism to maintain operation in the event of a failure. This should include a battery backup and a suitable alarm system for warning purposes. After the planning application is concluded, the design team should seek consent from Thames Water to pump and discharge water into the sewer.

- Route all electrical wiring at high level

- Ensure that the basement structure is adequately waterproofed during construction.

9.3. Drainage Assessment

The design of drainage and damp-proofing is not within the scope of this assessment and would normally be expected to be part of the structural Water proofer's remit at detailed design stage.

A common and anticipated detailed design stage approach is to use internal membranes (Delta or similar). These will be integral to the waterproofing of the basement. Any water from this will enter a drainage channel below the slab. This will be pumped and discharged into the existing sewer system.

It is recommended that a waterproofing specialist is employed to ensure all the water proofing requirements are met. The waterproofing specialist must name their structural waterproofer. The structural waterproofer must inspect the structural details and confirm that he is happy with the robustness.

Due to the segmental construction nature of the basement, it is not possible to waterproof the joints. All waterproofing must be made by the waterproofing specialist. They should review the Structural Engineer's design stage details and advise if water bars and stops are necessary.

The waterproofing designer must not assume that the structure is watertight. To help reduce water flow through the joints in the segmental pins, the following measures should be applied:

- All faces should be cleaned of all debris and detritus
- Faces between pins should be needle hammered to improve key for bonding
- All pipe work and other penetrations should have puddle flanges or hydrophilic strips

9.3.1. SUDS Assessment & Mitigation Measures

Existing Hard Standing = 50 m^2

Proposed Hardstanding = 50 m^2

Percentage Increase in hard standing = 0 %

There is no increase in hard surfaces compared to the existing hard surfaces present on site. There will be no additional discharge and the water will be pumped and discharged into the existing sewer system.

9.3.2. Drainage & SUDS Summary

There is no significant increase in the discharge of surface water into the existing sewer system. The use of complex SUDS features is therefore not considered applicable to a development of this scale.

9.3.3. Construction Mitigation Measures - Localised Dewatering

Monitor water levels 1 month prior to starting on site and throughout the construction process.

Localised dewatering to pins may be necessary.

Appendix A : Structural Calculations

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

The design must resist:

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

The final proposed scheme must:

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

As part of the planning Croft structural engineers has considered some of the pertinent parts of the basement structure to ensure that it can be constructed. The following calculations are not a full set of calculations for the final design which must be provided for building regulations.

WALL 1A (LIGHTWELL - WORST CASE)

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 2.9.08

Retaining wall details

Stem type	Cantilever
Stem height	h_{stem} = **2500** 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} = **1250** mm
Heel length	l_{heel} = **200** mm
Base thickness	t_{base} = **350** mm
Base density	γ_{base} = **25** kN/m^3
Height of retained soil	h_{ret} = **2500** mm
Angle of soil surface	β = **0** deg
Depth of cover	d_{cover} = **0** mm
Height of water	h_{water} = **1500** mm
Water density	γ_w = **9.8** kN/m^3

Retained soil properties

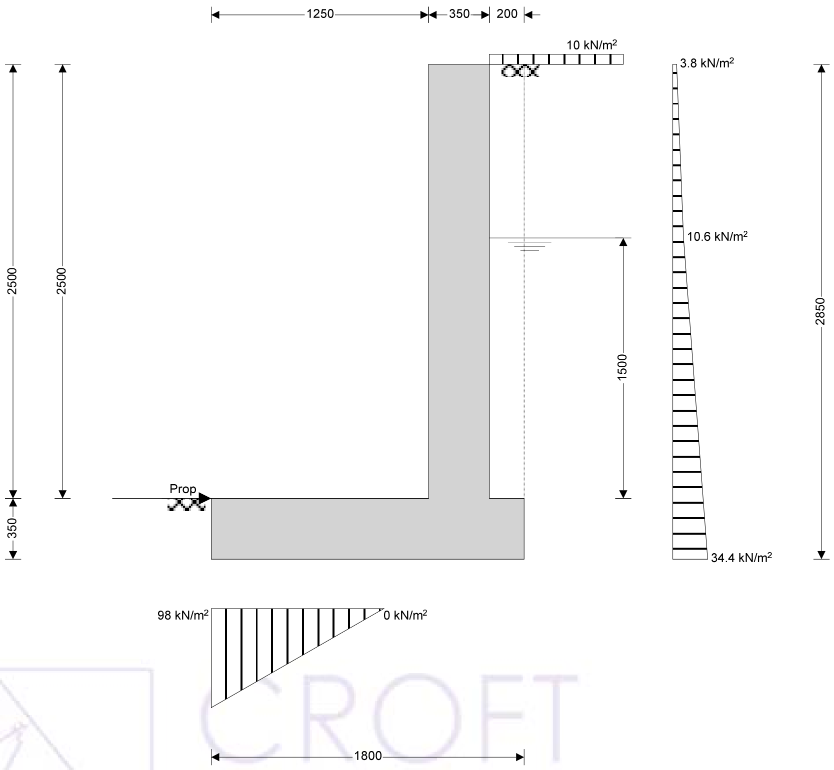
Soil type	Firm silty clay
Moist density	γ_{mr} = **18** kN/m^3
Saturated density	γ_{sr} = **18** kN/m^3
Characteristic effective shear resistance angle	$\phi'_{r.k}$ = **24** deg
Characteristic wall friction angle	$\delta_{r.k}$ = **11** deg

Base soil properties

Soil type	Firm silty clay
Soil density	γ_b = **18** kN/m^3
Characteristic effective shear resistance angle	$\phi'_{b.k}$ = **24** deg
Characteristic wall friction angle	$\delta_{b.k}$ = **11** deg
Characteristic base friction angle	$\delta_{bb.k}$ = **14.7** deg
Presumed bearing capacity	$P_{bearing}$ = **100** kN/m^2

Loading details

Variable surcharge load	$Surcharge_Q$ = **10** kN/m^2



General arrangement

Calculate retaining wall geometry

Base length	$l_{base} = l_{toe} + t_{stem} + l_{heel} = \mathbf{1800}$ mm
Saturated soil height	$h_{sat} = h_{water} + d_{cover} = \mathbf{1500}$ mm
Moist soil height	$h_{moist} = h_{ret} - h_{water} = \mathbf{1000}$ mm
Length of surcharge load	$l_{sur} = l_{heel} = \mathbf{200}$ mm
- Distance to vertical component	$x_{sur\_v} = l_{base} - l_{heel} / 2 = \mathbf{1700}$ mm
Effective height of wall	$h_{eff} = h_{base} + d_{cover} + h_{ret} = \mathbf{2850}$ mm
- Distance to horizontal component	$x_{sur\_h} = h_{eff} / 2 = \mathbf{1425}$ mm
Area of wall stem	$A_{stem} = h_{stem} \times t_{stem} = \mathbf{0.875}$ m^2
- Distance to vertical component	$x_{stem} = l_{toe} + t_{stem} / 2 = \mathbf{1425}$ mm
Area of wall base	$A_{base} = l_{base} \times t_{base} = \mathbf{0.63}$ m^2
- Distance to vertical component	$x_{base} = l_{base} / 2 = \mathbf{900}$ mm
Area of saturated soil	$A_{sat} = h_{sat} \times l_{heel} = \mathbf{0.3}$ m^2
- Distance to vertical component	$x_{sat\_v} = l_{base} - (h_{sat} \times l_{heel}^2 / 2) / A_{sat} = \mathbf{1700}$ mm
- Distance to horizontal component	$x_{sat\_h} = (h_{sat} + h_{base}) / 3 = \mathbf{617}$ mm
Area of water	$A_{water} = h_{sat} \times l_{heel} = \mathbf{0.3}$ m^2
- Distance to vertical component	$x_{water\_v} = l_{base} - (h_{sat} \times l_{heel}^2 / 2) / A_{sat} = \mathbf{1700}$ mm
- Distance to horizontal component	$x_{water\_h} = (h_{sat} + h_{base}) / 3 = \mathbf{617}$ mm
Area of moist soil	$A_{moist} = h_{moist} \times l_{heel} = \mathbf{0.2}$ m^2
- Distance to vertical component	$x_{moist\_v} = l_{base} - (h_{moist} \times l_{heel}^2 / 2) / A_{moist} = \mathbf{1700}$ mm
- Distance to horizontal component	$x_{moist\_h} = (h_{moist} \times (t_{base} + h_{sat} + h_{moist} / 3) / 2 + (h_{sat} + t_{base})^2/2) / (h_{sat} + t_{base} + h_{moist} / 2) = \mathbf{1193}$ mm

Using Coulomb theory

Active pressure coefficient	$K_A = \sin(\alpha + \phi'_{r.k})^2 / (\sin(\alpha)^2 \times \sin(\alpha - \delta_{r.k}) \times [1 + \sqrt{[\sin(\phi'_{r.k} + \delta_{r.k}) \times \sin(\phi'_{r.k} - \beta) / (\sin(\alpha - \delta_{r.k}) \times \sin(\alpha + \beta))]}]^2) = \mathbf{0.384}$

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Passive pressure coefficient	$K_P = \sin(90 - \phi'_{b.k})^2 / (\sin(90 + \delta_{b.k}) \times [1 - \sqrt{[\sin(\phi'_{b.k} + \delta_{b.k}) \times \sin(\phi'_{b.k}) / (\sin(90 + \delta_{b.k}))]}]^2)$ = **3.237**

Bearing pressure check

Vertical forces on wall

Wall stem	$F_{stem} = A_{stem} \times \gamma_{stem}$ = **21.9** kN/m
Wall base	$F_{base} = A_{base} \times \gamma_{base}$ = **15.8** kN/m
Surcharge load	$F_{sur\_v} = Surcharge_Q \times l_{heel}$ = **2** kN/m
Saturated retained soil	$F_{sat\_v} = A_{sat} \times (\gamma_{sr} - \gamma_w)$ = **2.5** kN/m
Water	$F_{water\_v} = A_{water} \times \gamma_w$ = **2.9** kN/m
Moist retained soil	$F_{moist\_v} = A_{moist} \times \gamma_{mr}$ = **3.6** kN/m
Total	$F_{total\_v} = F_{stem} + F_{base} + F_{sur\_v} + F_{sat\_v} + F_{water\_v} + F_{moist\_v}$ = **48.6** kN/m

Horizontal forces on wall

Surcharge load	$F_{sur\_h} = K_A \times \cos(\delta_{r.k}) \times Surcharge_Q \times h_{eff}$ = **10.7** kN/m
Saturated retained soil	$F_{sat\_h} = K_A \times \cos(\delta_{r.k}) \times (\gamma_{sr} - \gamma_w) \times (h_{sat} + h_{base})^2 / 2$ = **5.3** kN/m
Water	$F_{water\_h} = \gamma_w \times (h_{water} + d_{cover} + h_{base})^2 / 2$ = **16.8** kN/m
Moist retained soil	$F_{moist\_h} = K_A \times \cos(\delta_{r.k}) \times \gamma_{mr} \times ((h_{eff} - h_{sat} - h_{base})^2 / 2 + (h_{eff} - h_{sat} - h_{base}) \times (h_{sat} + h_{base}))$ = **16** kN/m
Base soil	$F_{pass\_h} = -K_P \times \cos(\delta_{b.k}) \times \gamma_b \times (d_{cover} + h_{base})^2 / 2$ = **-3.5** kN/m
Total	$F_{total\_h} = F_{sur\_h} + F_{sat\_h} + F_{water\_h} + F_{moist\_h} + F_{pass\_h}$ = **45.3** kN/m

Moments on wall

Wall stem	$M_{stem} = F_{stem} \times x_{stem}$ = **31.2** kNm/m
Wall base	$M_{base} = F_{base} \times x_{base}$ = **14.2** kNm/m
Surcharge load	$M_{sur} = F_{sur\_v} \times x_{sur\_v} - F_{sur\_h} \times x_{sur\_h}$ = **-11.9** kNm/m
Saturated retained soil	$M_{sat} = F_{sat\_v} \times x_{sat\_v} - F_{sat\_h} \times x_{sat\_h}$ = **0.9** kNm/m
Water	$M_{water} = F_{water\_v} \times x_{water\_v} - F_{water\_h} \times x_{water\_h}$ = **-5.3** kNm/m
Moist retained soil	$M_{moist} = F_{moist\_v} \times x_{moist\_v} - F_{moist\_h} \times x_{moist\_h}$ = **-12.9** kNm/m
Total	$M_{total} = M_{stem} + M_{base} + M_{sur} + M_{sat} + M_{water} + M_{moist}$ = **16.1** kNm/m

Check bearing pressure

Propping force	$F_{prop\_base} = F_{total\_h}$ = **45.3** kN/m
Distance to reaction	$\bar{x} = M_{total} / F_{total\_v}$ = **331** mm
Eccentricity of reaction	$e = \bar{x} - l_{base} / 2$ = **-569** mm
Loaded length of base	$l_{load} = 3 \times \bar{x}$ = **993** mm
Bearing pressure at toe	$q_{toe} = 2 \times F_{total\_v} / l_{load}$ = **98** kN/m^2
Bearing pressure at heel	q_{heel} = **0** kN/m^2
Factor of safety	$FoS_{bp} = P_{bearing} / \max(q_{toe}, q_{heel})$ = **1.021**

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 2.9.08

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

Concrete strength class	C30/37
Characteristic compressive cylinder strength	f_{ck} = **30** N/mm^2
Characteristic compressive cube strength	$f_{ck,cube}$ = **37** N/mm^2

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

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

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

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

Partial factor for concrete - Table 2.1N γ_C = **1.50**

Compressive strength coefficient - cl.3.1.6(1) α_{cc} = **0.85**

Design compressive concrete strength - exp.3.15 $f_{cd} = \alpha_{cc} \times f_{ck} / \gamma_C$ = **17.0** 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 $K_2 = 1.00 \times (0.6 + 0.0014/\varepsilon_{cu2})$ = **1.00**

Bending coefficient k_3 K_3 =**0.40**

Bending coefficient k_4 $K_4 = 1.00 \times (0.6 + 0.0014/\varepsilon_{cu2})$ =**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

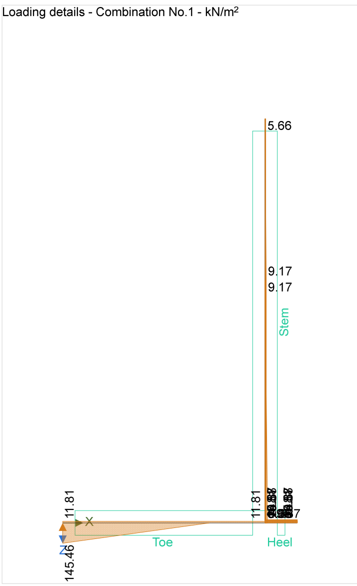
Cover to reinforcement

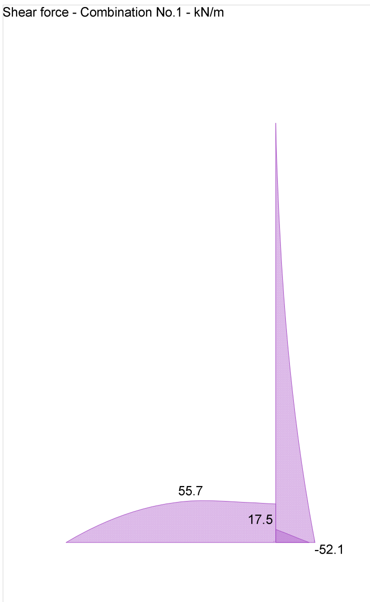
Front face of stem c_{sf} = **40** mm

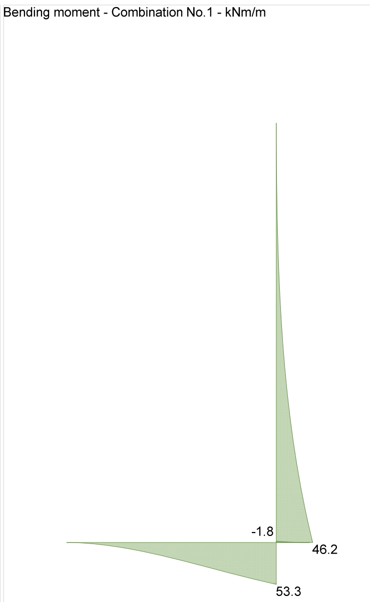
Rear face of stem c_{sr} = **50** mm

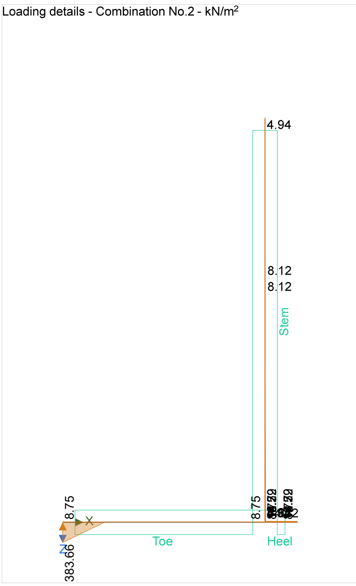
Top face of base c_{bt} = **50** mm

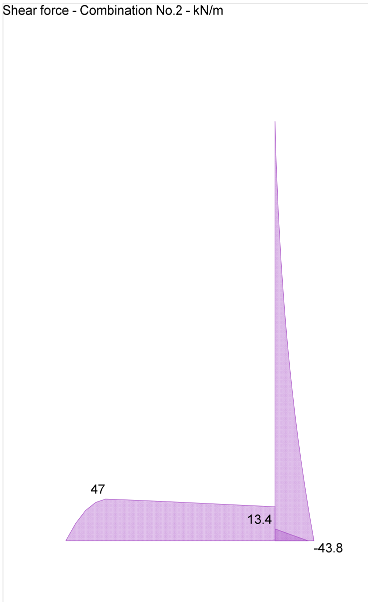
Bottom face of base c_{bb} = **75** mm

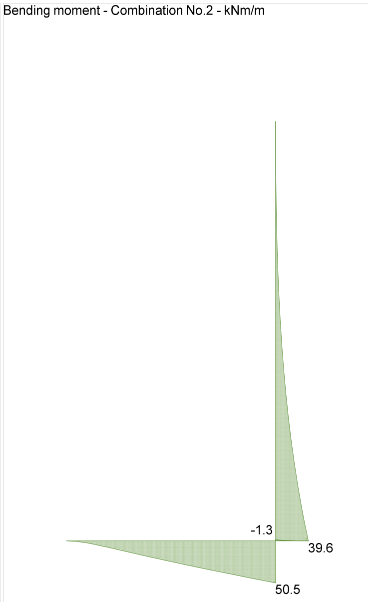












Check stem design at base of stem

Depth of section — h = **350** mm

Rectangular section in flexure - Section 6.1

Design bending moment combination 1 — M = **46.2** kNm/m

Depth to tension reinforcement — $d = h - c_{sr} - \phi_{sr} / 2$ = **294** mm

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

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

K' = **0.207**

K' > K - No compression reinforcement is required

Lever arm — $z = \min(0.5 + 0.5 \times (1 - 2 \times K / (\eta \times \alpha_{cc} / \gamma_C))^{0.5}, 0.95) \times d$ = **279** mm

Depth of neutral axis — $x = 2.5 \times (d - z)$ = **37** mm

Area of tension reinforcement required — $A_{sr.req} = M / (f_{yd} \times z)$ = **380** mm²/m

Tension reinforcement provided — 12 dia.bars @ 200 c/c

Area of tension reinforcement provided — $A_{sr.prov} = \pi \times \phi_{sr}^2 / (4 \times s_{sr})$ = **565** mm²/m

Minimum area of reinforcement - exp.9.1N — $A_{sr.min} = \max(0.26 \times f_{ctm} / f_{yk}, 0.0013) \times d$ = **443** mm²/m

Maximum area of reinforcement - cl.9.2.1.1(3) — $A_{sr.max} = 0.04 \times h$ = **14000** mm²/m

$\max(A_{sr.req}, A_{sr.min}) / A_{sr.prov}$ = **0.783**

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 — $\rho = A_{sr.req} / d$ = **0.001**

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

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

Reinforcement factor - exp.7.17 — $K_s = \min(500\ N/mm^2 / (f_{yk} \times A_{sr.req} / A_{sr.prov}), 1.5)$ = **1.487**

Limiting span to depth ratio - exp.7.16.a	min($K_s \times K_b \times [11 + 1.5 \times \sqrt{(f_{ck} / 1\ N/mm^2)} \times \rho_0 / \rho + 3.2 \times \sqrt{(f_{ck} / 1\ N/mm^2)} \times (\rho_0 / \rho - 1)^{3/2}], 40 \times K_b$) = **16**
Actual span to depth ratio	h_{stem} / d = **8.5**

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} = **28.2** kNm/m
Tensile stress in reinforcement	$\sigma_s = M_{sls} / (A_{sr.prov} \times z)$ = **178.5** N/mm²
Load duration	Long term
Load duration factor	k_t = **0.4**
Effective area of concrete in tension	$A_{c.eff}$ = min(2.5 × (h - d), (h - x) / 3, h / 2)
	$A_{c.eff}$ = **104417** mm²/m
Mean value of concrete tensile strength	$f_{ct.eff} = f_{ctm}$ = **2.9** N/mm²
Reinforcement ratio	$\rho_{p.eff} = A_{sr.prov} / A_{c.eff}$ = **0.005**
Modular ratio	$\alpha_e = E_s / E_{cm}$ = **6.091**
Bond property coefficient	k_1 = **0.8**
Strain distribution coefficient	k_2 = **0.5**
	k_3 = **3.4**
	k_4 = **0.425**
Maximum crack spacing - exp.7.11	$s_{r.max} = k_3 \times c_{sr} + k_1 \times k_2 \times k_4 \times \phi_{sr} / \rho_{p.eff}$ = **547** 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 = **0.293** mm
	w_k / w_{max} = **0.976**

PASS - Maximum crack width is less than limiting crack width

Rectangular section in shear - Section 6.2

Design shear force	V = **52.1** kN/m
	$C_{Rd,c} = 0.18 / \gamma_C$ = **0.120**
	k = min(1 + √(200 mm / d), 2) = **1.825**
Longitudinal reinforcement ratio	$\rho_l = \min(A_{sr.prov} / d, 0.02)$ = **0.002**
	$v_{min} = 0.035\ N^{1/2}/mm \times k^{3/2} \times f_{ck}^{0.5}$ = **0.473** N/mm²
Design shear resistance - exp.6.2a & 6.2b	$V_{Rd.c} = \max(C_{Rd.c} \times k \times (100\ N^2/mm^4 \times \rho_l \times f_{ck})^{1/3}, v_{min}) \times d$
	$V_{Rd.c}$ = **138.9** kN/m
	$V / V_{Rd.c}$ = **0.375**

PASS - Design shear resistance exceeds design shear force

Horizontal reinforcement parallel to face of stem - Section 9.6

Minimum area of reinforcement – cl.9.6.3(1)	$A_{sx.req} = \max(0.25 \times A_{sr.prov}, 0.001 \times t_{stem})$ = **350** mm²/m
Maximum spacing of reinforcement – cl.9.6.3(2)	s_{sx_max} = **400** mm
Transverse reinforcement provided	10 dia.bars @ 200 c/c
Area of transverse reinforcement provided	$A_{sx.prov} = \pi \times \phi_{sx}^2 / (4 \times s_{sx})$ = **393** mm²/m

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

Check base design at toe

Depth of section $h = \mathbf{350}$ mm

Rectangular section in flexure - Section 6.1

Design bending moment combination 1 $M = \mathbf{53.3}$ kNm/m

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

$K = M / (d^2 \times f_{ck}) = \mathbf{0.025}$

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

$K' = \mathbf{0.207}$

K' > K - No compression reinforcement is required

Lever arm $z = \min(0.5 + 0.5 \times (1 - 2 \times K / (\eta \times \alpha_{cc} / \gamma_C))^{0.5}, 0.95) \times d = \mathbf{254}$ mm

Depth of neutral axis $x = 2.5 \times (d - z) = \mathbf{33}$ mm

Area of tension reinforcement required $A_{bb.req} = M / (f_{yd} \times z) = \mathbf{483}$ mm^2/m

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

Area of tension reinforcement provided $A_{bb.prov} = \pi \times \phi_{bb}^2 / (4 \times s_{bb}) = \mathbf{1340}$ mm^2/m

Minimum area of reinforcement - exp.9.1N $A_{bb.min} = \max(0.26 \times f_{ctm} / f_{yk}, 0.0013) \times d = \mathbf{402}$ mm^2/m

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

$\max(A_{bb.req}, A_{bb.min}) / A_{bb.prov} = \mathbf{0.36}$

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 $w_{max} = \mathbf{0.3}$ mm

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

Serviceability bending moment $M_{sls} = \mathbf{37.9}$ kNm/m

Tensile stress in reinforcement $\sigma_s = M_{sls} / (A_{bb.prov} \times z) = \mathbf{111.3}$ N/mm^2

Load duration Long term

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

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

$A_{c.eff} = \mathbf{105542}$ mm^2/m

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

Reinforcement ratio $\rho_{p.eff} = A_{bb.prov} / A_{c.eff} = \mathbf{0.013}$

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

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

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

$k_3 = \mathbf{3.4}$

$k_4 = \mathbf{0.425}$

Maximum crack spacing - exp.7.11 $s_{r.max} = k_3 \times c_{bb} + k_1 \times k_2 \times k_4 \times \phi_{bb} / \rho_{p.eff} = \mathbf{469}$ mm

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

$w_k = \mathbf{0.157}$ mm

$w_k / w_{max} = \mathbf{0.522}$

PASS - Maximum crack width is less than limiting crack width

Rectangular section in shear - Section 6.2

Design shear force $V = \mathbf{55.7}$ kN/m

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

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

Longitudinal reinforcement ratio $\rho_l = \min(A_{bb.prov} / d, 0.02) = \mathbf{0.005}$

$v_{min} = 0.035 \text{ N}^{1/2}/\text{mm} \times k^{3/2} \times f_{ck}^{0.5} = \mathbf{0.488}$ N/mm²

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

$V_{Rd.c} = \mathbf{147.6}$ kN/m

$V / V_{Rd.c} = \mathbf{0.377}$

PASS - Design shear resistance exceeds design shear force

Check base design at heel

Depth of section $h = \mathbf{350}$ mm

Rectangular section in flexure - Section 6.1

Design bending moment combination 1 $M = \mathbf{1.8}$ kNm/m

Depth to tension reinforcement $d = h - c_{bt} - \phi_{bt} / 2 = \mathbf{294}$ mm

$K = M / (d^2 \times f_{ck}) = \mathbf{0.001}$

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

$K' = \mathbf{0.207}$

K' > K - No compression reinforcement is required

Lever arm $z = \min(0.5 + 0.5 \times (1 - 2 \times K / (\eta \times \alpha_{cc} / \gamma_C))^{0.5}, 0.95) \times d = \mathbf{279}$ mm

Depth of neutral axis $x = 2.5 \times (d – z) = \mathbf{37}$ mm

Area of tension reinforcement required $A_{bt.req} = M / (f_{yd} \times z) = \mathbf{14}$ mm²/m

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

Area of tension reinforcement provided $A_{bt.prov} = \pi \times \phi_{bt}^2 / (4 \times s_{bt}) = \mathbf{565}$ mm²/m

Minimum area of reinforcement - exp.9.1N $A_{bt.min} = \max(0.26 \times f_{ctm} / f_{yk}, 0.0013) \times d = \mathbf{443}$ mm²/m

Maximum area of reinforcement - cl.9.2.1.1(3) $A_{bt.max} = 0.04 \times h = \mathbf{14000}$ mm²/m

$\max(A_{bt.req}, A_{bt.min}) / A_{bt.prov} = \mathbf{0.783}$

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 $w_{max} = \mathbf{0.3}$ mm

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

Serviceability bending moment $M_{sls} = \mathbf{1.2}$ kNm/m

Tensile stress in reinforcement $\sigma_s = M_{sls} / (A_{bt.prov} \times z) = \mathbf{7.6}$ N/mm²

Load duration Long term

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

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

$A_{c.eff} = \mathbf{104417}$ mm²/m

Mean value of concrete tensile strength $f_{ct.eff} = f_{ctm} = \mathbf{2.9}$ N/mm²

Reinforcement ratio $\rho_{p.eff} = A_{bt.prov} / A_{c.eff} = \mathbf{0.005}$

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

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

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

$k_3 = \mathbf{3.4}$

$k_4 = \mathbf{0.425}$

Maximum crack spacing - exp.7.11 $s_{r.max} = k_3 \times c_{bt} + k_1 \times k_2 \times k_4 \times \phi_{bt} / \rho_{p.eff} = \mathbf{547}$ mm

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

$w_k = \mathbf{0.012}$ mm

$w_k / w_{max} = \mathbf{0.041}$

PASS - Maximum crack width is less than limiting crack width

Rectangular section in shear - Section 6.2

Design shear force $V = \mathbf{17.5}$ kN/m

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

$k = \min(1 + \sqrt{(200 \text{ mm} / d)}, 2) = \mathbf{1.825}$

Longitudinal reinforcement ratio $\rho_l = \min(A_{bt.prov} / d, 0.02) = \mathbf{0.002}$

$v_{min} = 0.035 \text{ N}^{1/2}/\text{mm} \times k^{3/2} \times f_{ck}^{0.5} = \mathbf{0.473}$ N/mm²

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

$V_{Rd.c} = \mathbf{138.9}$ kN/m

$V / V_{Rd.c} = \mathbf{0.126}$

PASS - Design shear resistance exceeds design shear force

Secondary transverse reinforcement to base - Section 9.3

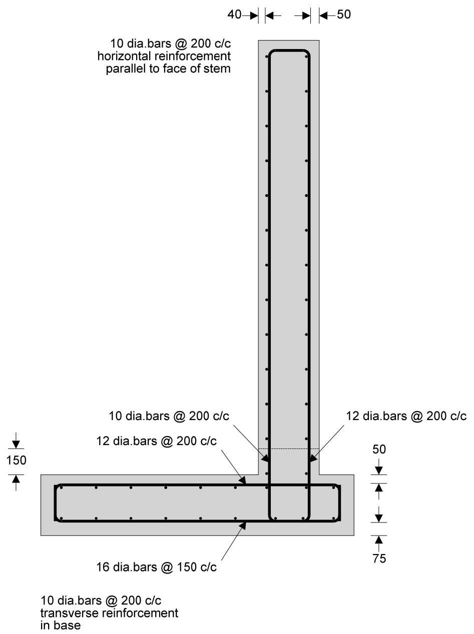
Minimum area of reinforcement – cl.9.3.1.1(2) $A_{bx.req} = 0.2 \times A_{bb.prov} = \mathbf{268}$ mm²/m

Maximum spacing of reinforcement – cl.9.3.1.1(3) $s_{bx\_max} = \mathbf{450}$ mm

Transverse reinforcement provided 10 dia.bars @ 200 c/c

Area of transverse reinforcement provided $A_{bx.prov} = \pi \times \phi_{bx}^2 / (4 \times s_{bx}) = \mathbf{393}$ mm²/m

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



Reinforcement details

Appendix B: Construction programme

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

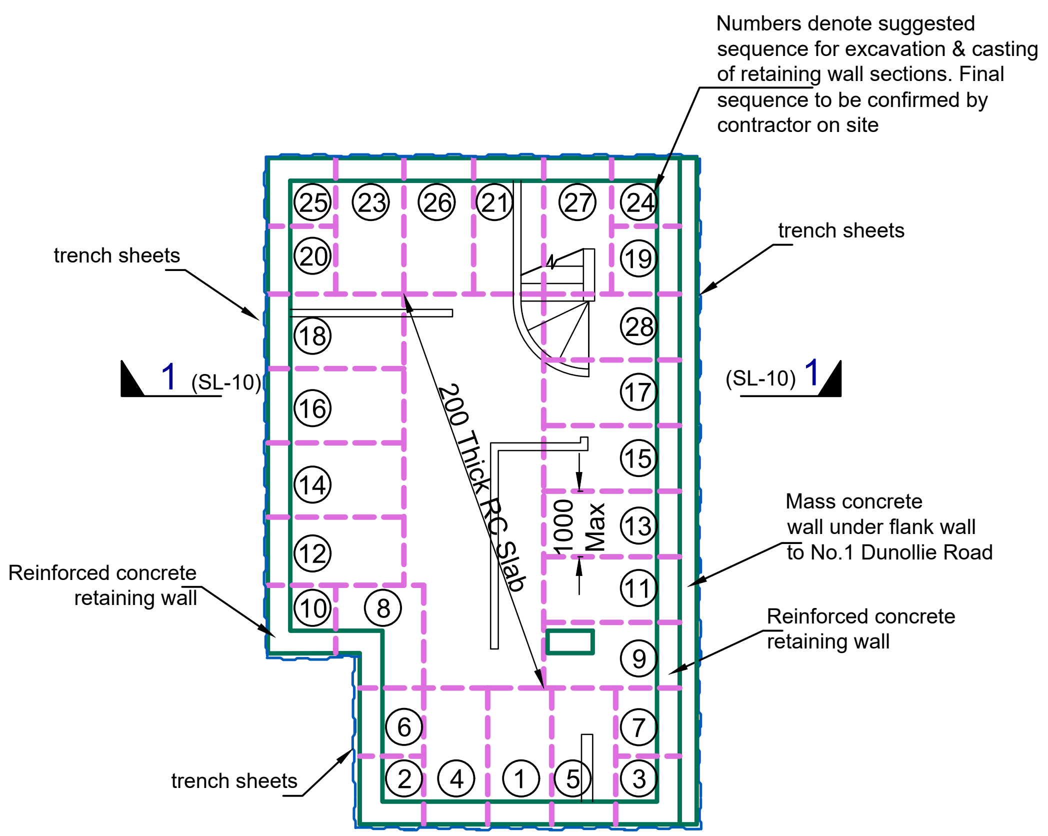
Outline construction Programme (For planning purposes only)																
	Months															
	1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16
Planning approval																
Derailed Design																
Tender																
Party Walls																
Monitoring of Adjacent structures																
Enabling works																
Basement Construction																
Superstructure construction																

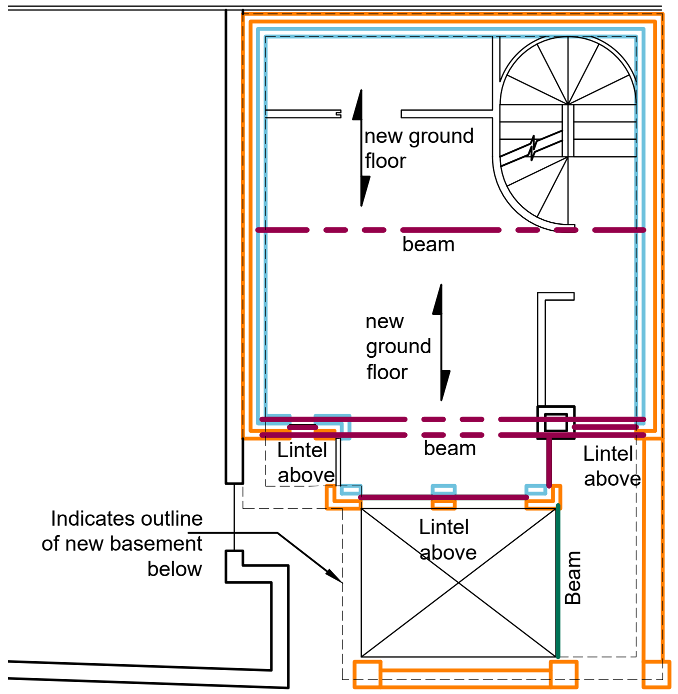
Appendix C : Utilities Searches

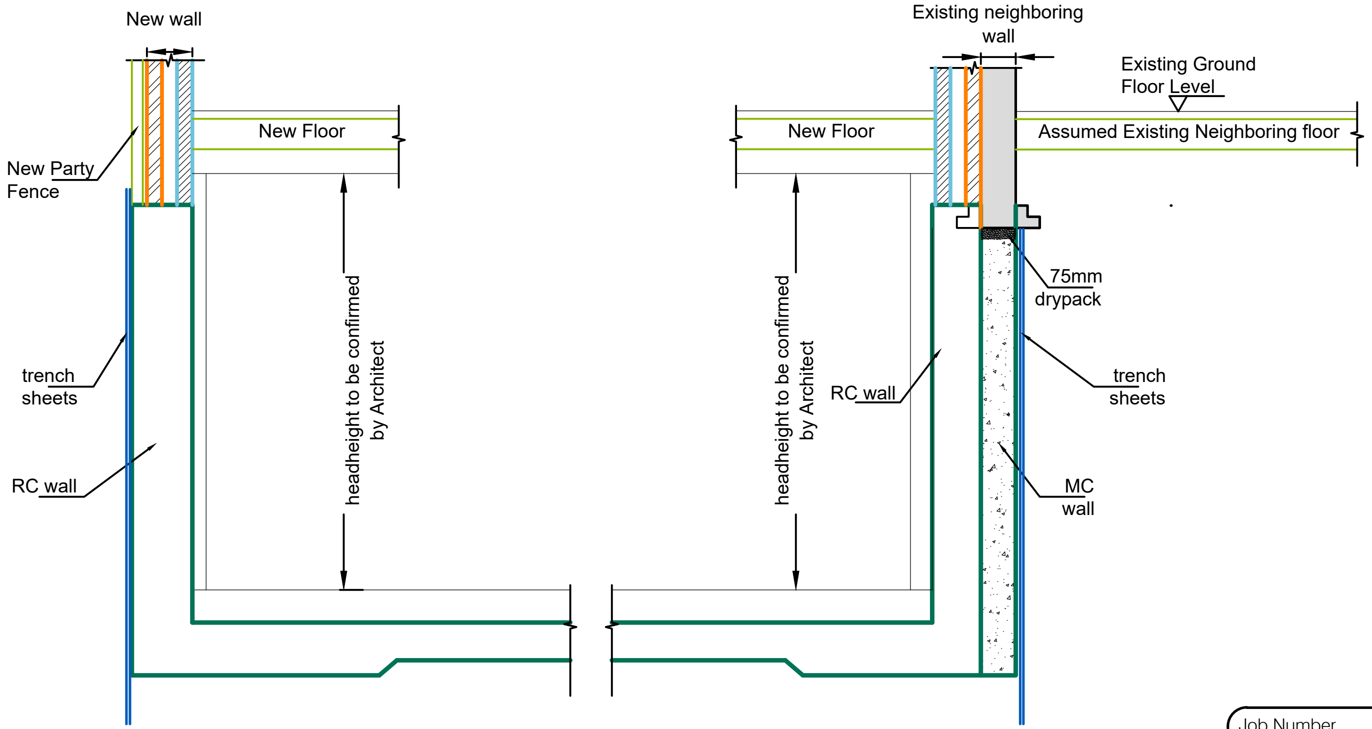
Appendix D : Structural Drawings

1:100 Basement Plan on A3 Showing Neighbouring basements if present
1:100 Ground Floor plan on A3 Showing Neighbouring property
1:50 Section on A3 Including section through Neighbouring Footings

Planning issue
Not for construction







Section 1-1
(Scale 1:50)

Rev	Date	Amendments
2	09/08/2019	Issued for planning minor amendments
1	10/07/2019	minor amendments
-	09/07/2019	Issued for planning

Job Number	Date
190611	July '19
Dwg Number: PL-10	Rev: 2
Drawn: SB	Ch'kd: EP
Scale: As shown @ A3	

Client: Ms Kerena Mond

Project: 1 Dunollie Road

Title : Basement and Ground Plan and Sections

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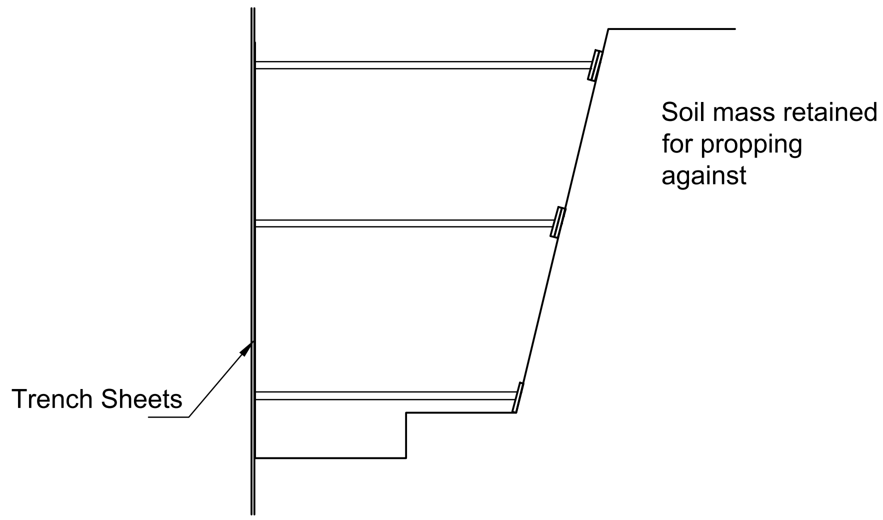
www.croftse.co.uk

Appendix E : Temporary Works Sequence

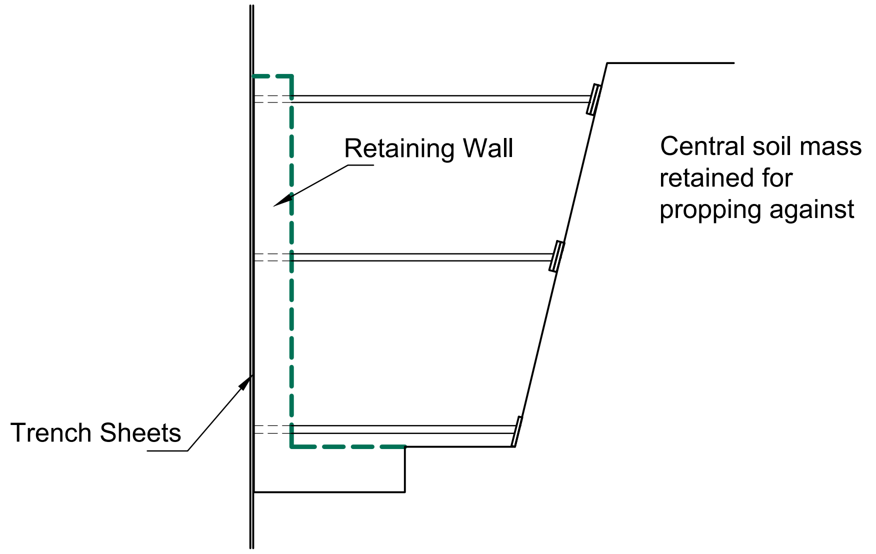
- Lateral propping
- Sequencing

Planning issue
Not for construction



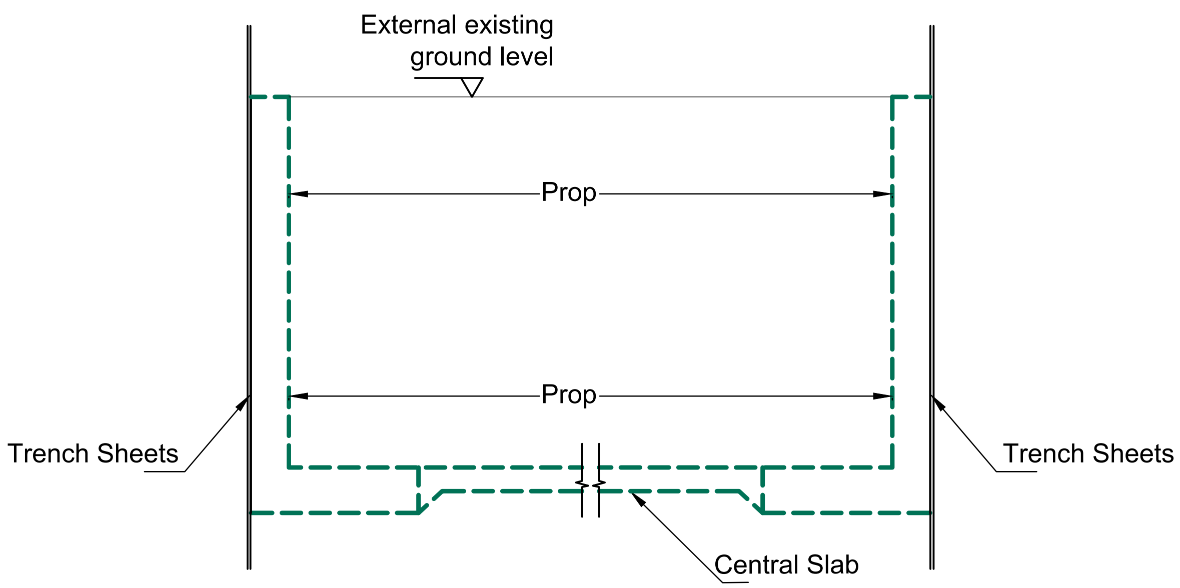
Phase 1

1. Excavate in 1m width maximum and install props and trench sheets and excavation progresses



Phase 2

1. Cast Retaining wall and Base.
2. Remove Props after concrete gain sufficient strength



Phase 3

1. Excavate and cast remainder of base slab. Install full length props while central soil mass is getting removed
2. Proceed with above ground construction

Rev	Date	Amendments
-	09/07/2019	Issued for planning

Job Number 190611	Date July '19
Dwg Number TW-11	Rev -
Drawn SB	Ch'kd EP
Scale As shown @ A3	

Client: Ms Kerena Mond

Project: 1 Dunollie Road

Title : Temporary works

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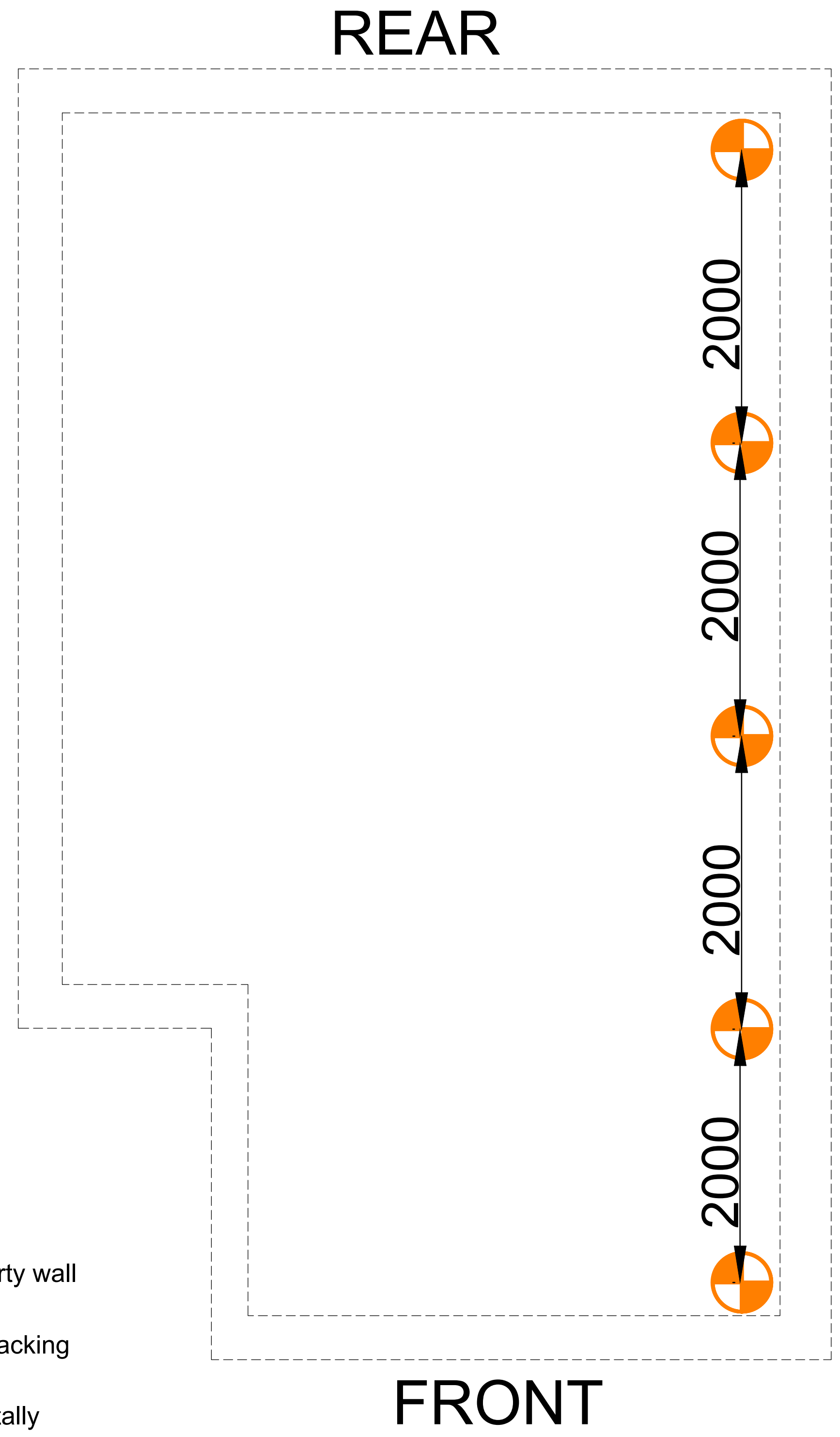
Appendix F : Monitoring locations

For Trigger values and frequency see BIA report



Key

Denotes position of Leveling Targets, fixed to party wall 500mm & 2000mm above Ground Floor Level.
Additional monitoring may be required for any cracking noted in the Party Wall Surveyor's survey.
Levels to be checked both vertically and horizontally

Ground Floor Layout

Rev	Date	Amendments
-	09-08-19	Issued for planning

Client: Ms Kerena Mond	Title : Movement monitoring layout		
Project: 1 Dunollie Road	Job nos 190611	Drawn EP	Date Aug 2019
	Dwg Nos SD-22	Rev -	Scale 1:50

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