Basement Impact Assessment

in connection with proposed development at

Nos. 10-12 Kentish Town Road
Camden
London
NW1 9NX

for

Kentish Town Spaces (UK) Ltd

LBH4536 Ver 1.2

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ENGINEERING

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Executive Summary

It is proposed to construct a basement at this property.

Planning consent (2017/2852/P) was granted in August 2017, to construct a rear extension at first and second floor level.

It is now additionally proposed to construct a single storey basement beneath the entire footprint of the building.

This report provides an assessment of the potential impacts that the basement development may have upon the surrounding area, neighbouring structures and the local environment.

The ground conditions at the site comprise the London Clay Formation.

Hydrogeological Impacts

The BIA screening has not identified any potential groundwater issues.

Hydrological Impacts

The BIA screening has identified that Kentish Town Road flooded in 1975

An assessment of the risk has been undertaken and the surface water flood risk is to be mitigated by incorporating flood resistant measures into the building design. SUDS attenuation is to be included within the development in order to help reduce flood risk elsewhere.

Stability Impacts

The BIA screening has identified the need for extensive underpinning and ground movement assessments have been undertaken to demonstrate the acceptability of the impact of the proposed development upon neighbouring structures and the adjoining pavement and highway with a prediction of Maximum Burland Category 1 (Very Slight) damage.

In addition, it has been established that the front wall of the site lies very close to the TfL London Underground assets beneath Kentish Town Road. The potential impact upon these is the subject of a separate assessment and is being addressed separately and directly with TfL.

Conclusion

No adverse residual or cumulative stability, hydrological or hydrogeological impacts are expected as a result of this development. This BIA concludes that the proposed development will not cause harm to its neighbours or the wider environment.

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Foreword-Guidance Notes

GENERAL

This report has been prepared for a specific client and to meet a specific brief. The preparation of this report may have been affected by limitations of scope, resources or time scale required by the client. Should any part of this report be relied on by a third party, that party does so wholly at its own risk and LBH Wembley Engineering disclaims any liability to such parties.

The observations and conclusions described in this report are based solely upon the agreed scope of work. LBH Wembley Engineering has not performed any observations, investigations, studies or testing not specifically set out in the agreed scope of work and cannot accept any liability for the existence of any condition, the discovery of which would require performance of services beyond the agreed scope of work.

VALIDITY

Should the purpose for which the report is used, or the proposed use of the site change, this report may no longer be valid and any further use of or reliance upon the report in those circumstances shall be at the client's sole and own risk. The passage of time may result in changes in site conditions, regulatory or other legal provisions, technology or economic conditions which could render the report inaccurate or unreliable. The information and conclusions contained in this report should therefore not be relied upon in the future and any such reliance on the report in the future shall again be at the client's own and sole risk.

THIRD PARTY INFORMATION

The report may present an opinion based upon information received from third parties. However, no liability can be accepted for any inaccuracies or omissions in that information.

1. Introduction

1.1 Background

Planning consent (2017/2852/P) was granted, subject to a Section 106 Legal Agreement, by the London Borough of Camden in August 2017, to construct a rear extension at first and second floor level, for the purposes of converting Nos. 10-12 Kentish Town Road to a hotel. It is now additionally proposed to construct a single storey basement beneath the entire footprint of the building.

Excavation of the basement at this property has already commenced; hence this Basement Impact Assessment (BIA) has been prepared to support a retrospective planning application.. However, the BIA has been written from the perspective that the development has not yet begun.

1.2 Brief

LBH WEMBLEY have been appointed by Kentish Town Spaces (UK) Ltd to complete a BIA for submission to London Borough of Camden, in support of a planning application (2018/2425/P) for the proposed basement development.

This BIA has been prepared to satisfy the specific requirements of the 2017 Camden Planning Policy and Supplementary Planning Guidance CPG on Basements and Lightwells, and the associated 2010 Camden Geological, Hydrogeological and Hydrological Study.

1.3 Policy

The 2017 Camden Local Plan Policy A5 Basements reads as follows:

“The Council will only permit basement development where it is demonstrated to its satisfaction that the proposal would not cause harm to:

a) neighbouring properties;
b) the structural, ground, or water conditions of the area;
c) the character and amenity of the area;
d) the architectural character of the building; and
e) the significance of heritage assets.
In determining proposals for basements and other underground development, the Council will require an assessment of the scheme’s impact on drainage, flooding, groundwater conditions and structural stability in the form of a Basement Impact Assessment and where appropriate, a Basement Construction Plan.

The siting, location, scale and design of basements must have minimal impact on, and be subordinate to, the host building and property. Basement development should:

f) not comprise of more than one storey;
g) not be built under an existing basement;
h) not exceed 50% of each garden within the property;
i) be less than 1.5 times the footprint of the host building in area;
j) extend into the garden no further than 50% of the depth of the host building measured from the principal rear elevation;

k) not extend into or underneath the garden further than 50% of the depth of the garden;
l) be set back from neighbouring property boundaries where it extends beyond the footprint of the host building; and
m) avoid the loss of garden space or trees of townscape or amenity value.

Exceptions to f. to k. above may be made on large comprehensively planned sites.

The Council will require applicants to demonstrate that proposals for basements:

n. do not harm neighbouring properties, including requiring the provision of a Basement Impact Assessment which shows that the scheme poses a risk of damage to neighbouring properties no higher than Burland Scale 1 'very slight';
o. avoid adversely affecting drainage and run-off or causing other damage to the water environment;
p. avoid cumulative impacts;
q. do not harm the amenity of neighbours;
r. provide satisfactory landscaping, including adequate soil depth;
s. do not harm the appearance or setting of the property or the established character of the surrounding area;
t. protect important archaeological remains; and
u. do not prejudice the ability of the garden to support trees where they are part of the character of the area.

The Council will not permit basement schemes which include habitable rooms and other sensitive uses in areas prone to flooding.

We will generally require a Construction Management Plan for basement developments.

Given the complex nature of basement development, the Council encourages developers to offer security for expenses for basement development to adjoining neighbours."

The following policies in the Local Plan are also relevant to basement development and will be taken into account when assessing basement schemes:

- "Policy A2 Open space";
- "Policy A3 Biodiversity";
- "Policy D1 Design";
- "Policy D2 Heritage"; and
- "Policy CC3 Water and flooding".

In addition to the Local Plan Policy, Camden publishes Camden Planning Guidance on Basements and Lightwells. These CPG documents do not carry the same weight as the main Camden Development Plan documents (including the above Policy A5) but they are important supporting documents.

It is noted that the CPG Planning Guidance on Basements (formerly CPG4 2015) has been updated (March 2018) to reflect the Local Plan.

1.4 Report Structure

The report commences with a desk study and characterisation of the site, before progressing to BIA screening and scoping assessments, whereby consideration is given to identifying the potential hydrogeological, hydrological and stability impacts to be associated with the proposed development.

Following this the findings of an intrusive ground investigation are reported and a ground model is developed, followed by a discussion of the geotechnical issues. A discussion of the geotechnical issues is then put forward, followed by an outline construction methodology. An assessment of the potential ground movements affecting neighbouring structures is then provided.

Finally, an assessment of the potential impacts of the proposed scheme is presented.

1.5 Documents Consulted

The following documents have been consulted during the preparation of this document:

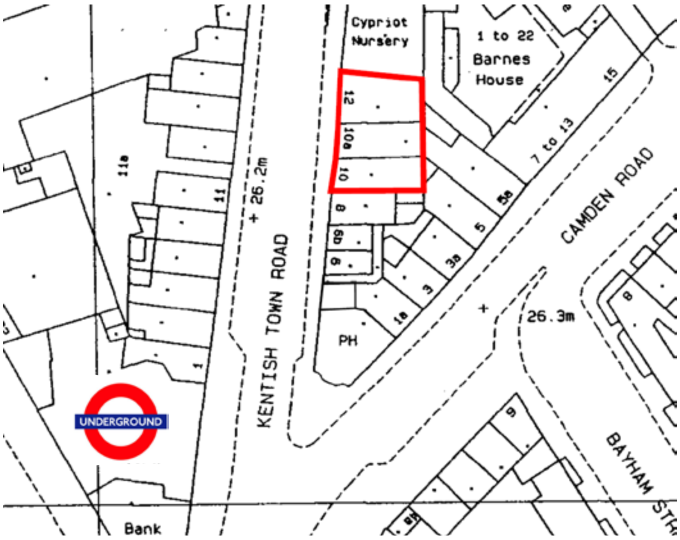
1. Proposed Ground Floor Plan by Ambigram Architects, dated 21st May 2018, Dwg No. P001, Rev. PL-0
2. Existing and Proposed Section by Ambigram Architects, dated 21st May 2018, Dwg No. P110, Rev. PL-0
3. Existing Ground Floor Plan by Ambigram Architects, dated 21st May 2018, Dwg No. E001, Rev. PL-0
4. Flood Risk Assessment & SUDS Strategy for Nos. 10-12 Kentish Town Road by LBH WEMBLEY, dated May 2018, ref: LBH4536fra Ver. 0.1
5. Records of Ground Investigation at No. 8 Kentish Town Road by Fastrack, dated May 2018, Ref: 11466 (appended)
6. Structural Calculations by Hinerti Ltd, dated July 2018 (appended)
7. Basement Impact Assessment Audit by Campbell Reith, dated October 2018, ref: 12985-03, rev: D1

2. The Site

2.1 Site Location

The site is situated on the eastern side of Kentish Town Road, approximately 40m to the northeast of Camden Town underground station.

The site may be located approximately by postcode NW1 9NX or by National Grid Reference 528960, 183990.



Location Plan

2.2 Topographical Setting

The site lies on a very gentle south-eastwards falling slope on the west bank of the now culverted River Fleet, which runs approximately 200m from the site.

2.3 Site Description

The site is occupied by three terraced properties that front onto the east side of Kentish Town Road.

No. 10 & 10A are mainly three storeys in height, but include single storey rear extensions. No. 12 is a two storey building. These buildings occupy the full length of the site.

Street level lies at approximately +26m OD and the existing ground floor level is raised by approximately 100mm from the street level to the front of the property.

The ground floor was previously occupied by a restaurant, while the upper floors have been vacant for a number of years.

To the north, the building adjoins a single storey gym (former nursery) at No. 16 Kentish Town Road. This building does not appear to have a basement.

To the south, the building adjoins a three storey terraced building at No. 8 Kentish Town Road. The ground floor is currently vacant, while residential flats occupy the upper floors.

To the rear the site backs onto three storey terraced buildings at Nos. 5 and 5A Camden Road. These buildings appear to have a cellar floors situated at roughly 1.5m below ground floor level.

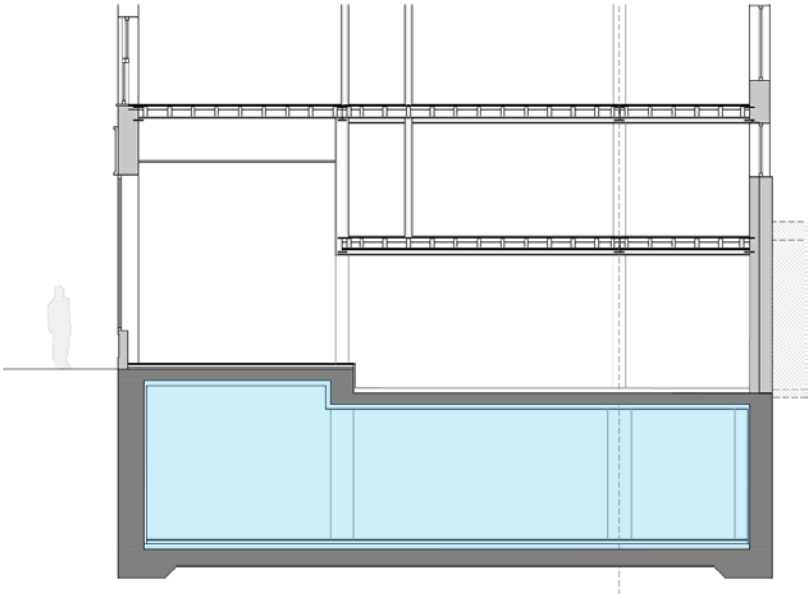
Part of Camden Town Underground Station lies beneath the pavement immediately adjacent to the site.

The crown of the tunnel appears to lie at approximately +13m OD.

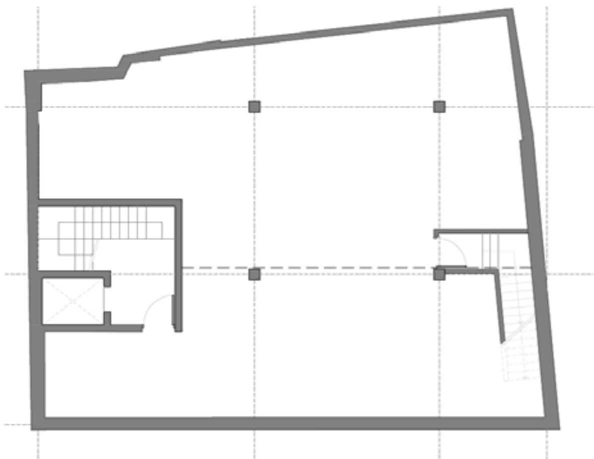
2.4 Proposed Development

It is proposed to construct a basement beneath the entire footprint of the building.

The proposed basement excavation will extend to approximately 4m (+22m OD) depth beneath the ground floor, with an indicated slab level at +22.34m OD.



Section showing proposed basement (highlighted in blue)



Plan showing proposed development

3. Desk Study

3.1 Site History

Earlier buildings on the site were demolished at the end of the 19th century and replaced by the existing row of terraced buildings.

At a similar time, although probably slightly earlier, the existing row of three storey terraced buildings that front onto Camden Road was also built.

The site and surrounding area has remained relatively unchanged since the early 1900s; however, in recent years, planning permission was granted to construct the existing mansard roof at No. 8 Kentish Town Road to provide additional residential accommodation. Earlier buildings on the site were demolished at the end of the 19th century and replaced by the existing row of terraced buildings.

3.2 Geological Information

The British Geological Survey (BGS) records indicate that the site is underlain by the London Clay Formation.

3.3 Hydrogeological Information

The London Clay Formation may be considered virtually impermeable; hence no significant groundwater flow is expected to occur beneath the site.

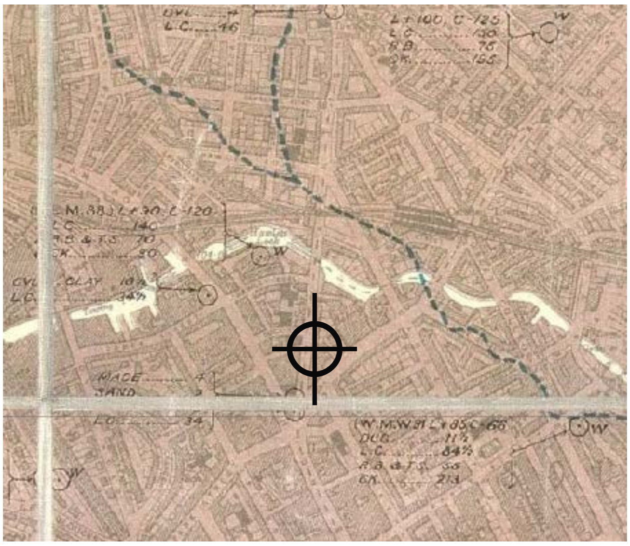


Figure 2: Camden 1920 Geological Map (CGHHS, 2010)
(dashed blue line shows the River Fleet)

3.4 Hydrological, Drainage and Flood Risk Information

Figure 2 of the CGHHS indicates that the River Fleet passes approximately 200m to the northeast of the site. There are no surface water features in the vicinity of the site.

The site is entirely hard-surfaced with the building occupying the entirely of the site.

Rainfall incident on the roof appears to be collected via pipework down the rear side of building, where it then discharges to a combined sewer that appears to run along the rear of the property.

Environment Agency (EA) surface water flood maps indicate that the site is at a very low risk of surface water flooding. However, according to the Flood in Camden 2003 report, Kentish Town Road is reported to have flooded in 1975.

Figure 6 of the Camden SFRA indicates that the site lies within a Critical Drainage Area (Group 3 003).

4. Screening & Scoping Assessments

The Screening & Scoping Assessments have been undertaken with reference to Appendices E and F of the CGHSS, which is a process for determining whether or not a BIA is usually required.

4.1 Screening Assessment

The Screening Assessment consists of a series of checklists that identifies any matters of concern relating to the following:

- Subterranean (groundwater) flow
- Surface flow and flooding
- Slope stability

4.1.1 Screening Checklist for Subterranean (Groundwater) Flow

Question	Response	Justification
Is the site is located directly above an aquifer?	**No**	The site is underlain by London Clay
Will the proposed basement extend beneath the water table surface?	**No**	No shallow groundwater is present beneath the site.
Is the site within 100m of a watercourse, well (used/disused) or potential spring line?	**No**	The nearest watercourse is the culverted River Fleet, approximately 200m to the northeast of the site.
Is the site within the catchment of the pond chains on Hampstead Heath?	**No**	CGHHS Fig. 14.
Will the proposed development result in a change in the area of hard-surfaced/paved areas?	**No**	Both the existing site and proposed development are entirely hard surfaced.
Will more surface water (e.g. rainfall and run-off) than at present will be discharged to the ground (e.g. via soakaways and/or SUDS)?	**No**	All surface water falling within the development will be attenuated and discharged to the Thames Water combined sewer.
Is the lowest point of the proposed excavation (allowing for any drainage and foundation space under the basement floor) close to or lower than the mean water level in any local pond?	**No**	CGHHS Fig. 12.

4.1.2 Screening Checklist for Surface Flow and Flooding

Question	Response	Justification
Is the site within the catchment area of the pond chains on Hampstead Heath?	**No**	CGHHS Fig. 14.
As part of the site drainage, will surface water flows (e.g. rainfall and run-off) be materially changed from the existing route?	**No**	The existing drainage arrangement will be maintained.
Will the proposed basement development result in a change in the proportion of hard-surfaced/paved areas?	**No**	Both the existing site and proposed development are entirely hard surfaced.
Will the proposed basement result in changes to the profile of the inflows (instantaneous and long-term) of surface-water being received by adjacent properties or downstream watercourses?	**No**	The existing drainage arrangement will be maintained.
Will the proposed basement result in changes to the quality of surface water being received by adjacent properties or downstream watercourses?	**No**	The existing drainage arrangement will be maintained.
Is the site in an area known to be at risk from surface water flooding, or is it at risk from flooding for example because the proposed basement is below the static water level of a nearby surface water feature?	**Yes**	Kentish Town Road is reported to have flooded in 1975

4.1.3 Screening Checklist for Stability

Question	Response	Justification
Does the existing site include slopes, natural or manmade, greater than 7 degrees?	**No**	There are no slopes greater than 7 degrees within the site.
Does the proposed re-profiling of landscaping at the site change slopes at the property boundary to more than 7 degrees?	**No**	No re-profiling is planned at the site.
Does the development neighbour land, including railway cuttings and the like, with a slope greater than 7 degrees?	**No**	There are no slopes greater than 7 degrees within the development land.
Is the site within a wider hillside setting in which the general slope is greater than 7 degrees?	**No**	Figure 6 of the CGHHS indicates that the general slope of the wider hillside is less than 7 degrees.

Is London Clay the shallowest strata at the site?	**Yes**	The site is underlain by London Clay Formation
Will trees be felled as part of the proposed development and/or are works proposed within tree protection zones where trees are to be retained?	**No**	There are no trees on the site.
Is there a history of seasonal shrink-swell subsidence in the local area, and/or evidence of such effects at the site?	**No**	
Is the site within 100m of a watercourse of a potential spring line?	**No**	The nearest watercourse is the culverted River Fleet, roughly 200m to the northeast of the site.
Is the site within an area of previously worked ground?	**No**	The British Geological Survey (BGS) records do not indicate that the site lies within an area of previously worked ground.
Is the site within an aquifer?	**No**	The site is underlain by London Clay
Will the proposed basement extend beneath the water table such that dewatering may be required during construction?	**No**	No shallow groundwater is present beneath the site.
Is the site within 50m of the Hampstead Heath ponds?	**No**	CGHHS Fig. 14.
Is the site within 5m of a highway or pedestrian right of way?	**Yes**	The proposed basement adjoins the pavement
Will the proposed basement significantly increase the differential depth of foundations relative to the neighbouring properties?	**Yes**	The proposed basement will increase the differential depth to foundations to No. 16 Kentish Town Road and No. 5a Camden Road.
Is the site over (or within the exclusion zone of) tunnels, e.g. railway lines?	**Yes**	The LUL Northern Line runs beneath the pavement to Kentish Town Road adjacent to the site.

4.2 Scoping Assessment

Where the checklist is answered with a “yes” or “unknown” to any of the questions posed in the flowcharts, these matters are carried forward to the scoping stage of the BIA process.

The scoping produces a statement which defines further the matters of concern identified in the screening stage. This defining should be in terms of ground processes, in order that a site specific BIA can be designed and executed (Section 6.3 of the CGHHS).

4.2.1 Scoping for Surface Flow and Flooding

- **Is the site in an area known to be at risk from surface water flooding, 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 guidance advises that a Flood Risk Assessment may be required.

4.2.2 Scoping for Stability

- **Is the London Clay the shallowest strata at the site?**

The guidance advises that of the at-surface soil strata present in LB Camden, the London Clay is the most prone to seasonal shrink-swell (subsidence and heave).

- **Is the site within 5m of a highway or pedestrian right of way?**

The guidance advises that excavation for a basement may result in damage to the road, pathway or any underground services buried in trenches beneath the road or pathway.

- **Will the proposed basement significantly increase the differential depth of foundations relative to neighbouring properties?**

The guidance advises that excavation for a basement may result in structural damage to neighbouring properties if there is a significant differential depth between adjacent foundations.

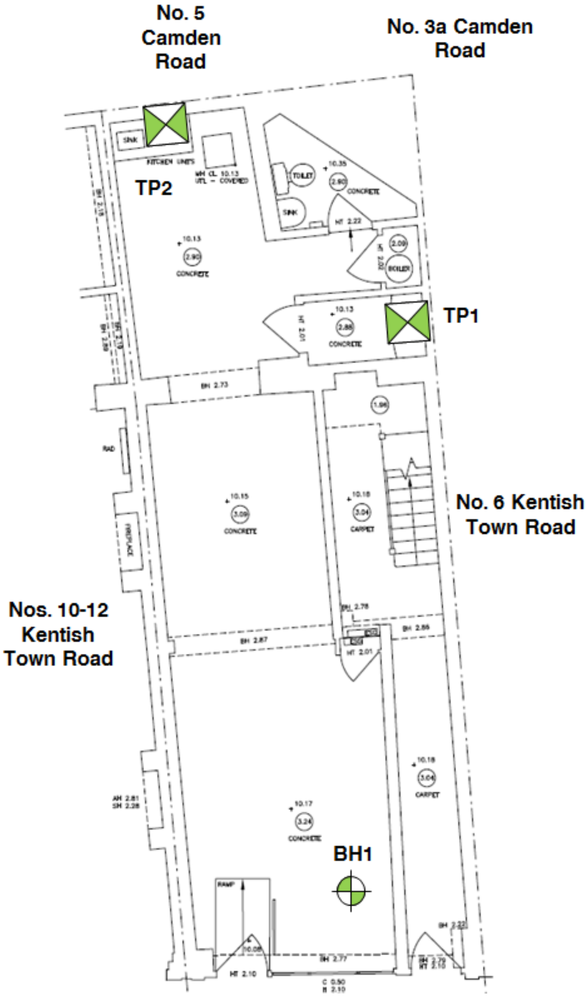
- **Is the site over (or within the exclusion zone of) any tunnels, e.g. railway lines?**

The guidance advises that excavation for a basement may result in damage to the tunnel.

5. Site Investigation

An intrusive site investigation, comprising a small diameter percussive diameter borehole and hand excavated trial pits was carried out in the adjacent property at No. 8 Kentish Town Road in May 2018.

The site plan to the right indicates the approximate positions of the exploratory positions, while the associated exploratory logs are appended.



Ground floor plan of showing exploratory positions in adjacent property at No. 8 Kentish Town Road

5.1 Ground Conditions

The ground investigation found the London Clay Formation to be present at shallow depth, and to consist of typical firm, becoming stiff, pale brown silty clay with occasional selenite crystals.

5.2 Groundwater

No groundwater table is present beneath the site.

6. Outline Basement Construction Methodology

6.1 Summary

The following outline methodology and sequence of works should be varied by the basement contractor or the structural engineer only by agreement with the basement design engineer and should be incorporated into the engineer's construction design and the contractor's method statements.

6.2 Methodology

The basement excavation will extend down into the London Clay Formation.

In the absence of any substantial groundwater inflows into the basement excavation, the basement perimeter walls will be formed by conventional underpinning and the construction of L-shaped reinforced cast in-situ concrete segments excavated and cast around the site in a 'hit and miss' sequence of 1m wide sections.

The depth of underpinning will be around 3m and a single stage of underpinning will therefore be used.

During the works, temporary propping will be utilised in order to ensure that lateral ground movements are minimised.

An upper row of props will be installed across the site between the newly underpinned walls prior to the main basement excavation, within trenches set at around 1m depth below ground level.

Initial excavation of the main basement area will then be undertaken by means of a series of discrete trenches, to allow the installation of a second row of low level props.

In the permanent situation the reinforced concrete underpins connected to the reinforced concrete floor slab will combine to form a rigid concrete box to support the vertical structural loading of the overlying building. Both the basement raft slab and the ground floor slab will act as props.

6.3 Site Set-Up

The site set-up will be detailed within the Construction Management Plan (CMP), which will also set out the traffic management measures for agreement with the council.

A skip will be placed to the front of the property with the siting of a compressor and materials in the same location. Hoarding will be erected around the skip and materials to ensure the protection of passers-by.

A conveyor belt will be installed initially sited towards the front of the property. A local excavation will be extended down in the central area of the site to allow the installation of the conveyor belt. The conveyor will extend up to feed the skip at ground level.

Spoil will be wheel barrowed from the excavation faces to the base of the conveyor belt. Spoil will be removed via the conveyor belt and deposited into the skip. The skip will be emptied using a grab lorry when it is full, or alternatively the skip will be exchanged.

6.4 Underpinning

The walls to the perimeter of the new basement will be wholly underpinned in reinforced concrete. During their construction the walls and bases will be propped in the temporary condition against an unexcavated dumpling of soil left in the central area of the site.

Underpinning sections will be excavated in short widths not exceeding 1000mm.

The sequence of the underpinning will be in an extended 1, 3, 5, 2, 4 & 6 type numbering sequence, such that any given underpin will be completed, dry packed, and a minimum period of 48 hours lapsed before and adjacent excavation is commenced to form another underpin.

In the event that the existing foundations to the wall are found to be unstable, sacrificial steel jacks will be installed underneath the existing foundation to prop the bottom few courses of bricks. These steel jacks will be left in place and will be incorporated into the concrete.

Each pin excavation will be undertaken only under the direct supervision of a suitably experienced and competent person. In the event that the vertical soil face to an underpin is judged to be potentially unstable, face support and lateral propping will be provided as required, using perforated plywood shutter sheeting supported by temporary walings and adjustable steel trench “acrow” props.

Where such sheeting is installed, great care will be taken to ensure that the concrete can flow through and fill any voids behind the shuttering. Any propping installed will be sacrificial and become part of the permanent works.

Subject to the requirements of the CMP, ready mixed concrete will be delivered to site and will be chuted into a catchment area within the excavated basement and placed by wheelbarrow or alternatively will be pumped.

Excavation for an underpin section will be excavated in a day, and the concrete to the base section of the underpin will poured by the end of the same day. The concrete to the stem of the underpin will be poured the following day. This will be poured up to within 50-75mm of the underside of the existing wall foundations.

On the following day, the gap between the concrete and the underside of the existing foundation will be dry packed with a rammed mixture of sharp sand and cement (ratio 3:1).

Two levels of temporary laterally propping of the new stem section will then be installed and maintained until the basement floor slab and the ground floor slab are cast and cured.

Once the dry pack has gained sufficient strength, any protrusions of the original footing into the site will be carefully trimmed back using hand tools to be flush with the face of the stem wall.

A minimum of 48 hours will be allowed before adjacent sections will be excavated to form a new underpin.

Adjacent underpins will be connected using B12 steel dowel bars 600mm long with 300mm embedment each side, at 200mm vertical centres.

6.5 Construction Sequence

1. Carry out excavation of initial pin (#1) to 200mm above base of existing foundations.
2. Excavate a shaft beneath the existing foundations to a depth of approx. 3.0m beneath foundations, ensuring the shaft is fully supported and propped to the full depth of the shaft.
3. Continue underpinning of perimeter walls in specified sequence until completed.

4. Install high level propping to underpinning in shallow trenches
5. Install low level propping to underpinning in deep trenches.
6. Main Basement Bulk excavation to be progressed down to the basement slab formation level.
7. The below-slab drainage for foul and ground water, sumps and pumps will be installed.
8. Slab reinforcement placed and basement slab cast.
9. Ground floor slab cast
10. Temporary propping removed.
11. Basement liner walls, membranes, cavity drainage, insulation and screeds to be installed.

6.6 Retaining Walls

The following parameters may be considered in the design of the retaining walls:-

Stratum	Bulk Unit Weight	Effective Cohesion	Effective Friction Angle
	(kN/m^3)	(c' - kN/m^2)	(ϕ'- degrees)
London Clay Formation	19.0	Zero	25

6.7 Waterproofing

Although no near-surface groundwater table is present at this site, there is potential for water to collect around the basement in the long term. Hence, the basement is to be fully waterproofed and designed to withstand hydrostatic pressures in accordance with BS8102:2009, Code of Practice for the Protection of Below-Ground Structures against Water from the Ground. An assumed hydrostatic level at 1m depth would be prudent for the purposes of assessing hydrostatic pressures, in order to allow for the possibility of surface water flooding due to a water main burst or similar.

6.8 Monitoring

A structural survey and monitoring scheme should be agreed with the Party Wall Surveyors and with TfL in order to provide an early warning of any movements and to allow the timely application of mitigation measures to prevent any unacceptable movements.

7. Ground Movement to Neighbouring Properties

Camden Council seeks to ensure that harm will not be caused to neighbouring properties by basement development.

Camden Local Plan (June 2017) states that the BIA must demonstrate that the proposed basement scheme has a risk of damage to the neighbouring properties no higher than Burland Scale 1 'Very Slight'.

Assessment of any potential impact upon the TfL London Underground assets beneath Kentish Town Road is being addressed separately and directly with TfL.

7.1 Structures Assessed for Ground Movement

7.1.1 No. 8 Kentish Town Road

No. 8 Kentish Town Road is a Victorian three storey terraced building that adjoins the site to the south.

This building does not have a basement.

7.1.2 No. 16 Kentish Town Road

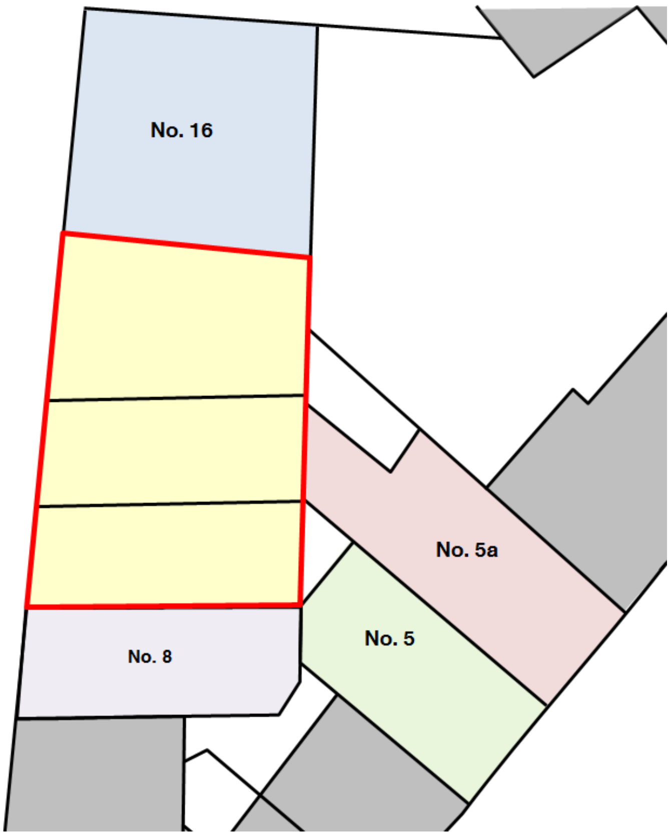
No. 16 Kentish Town Road is a late-20th Century single storey lightweight industrial-type building that adjoins the site to the north.

The building does not appear to have a basement.

7.1.3 No. 5 and 5a Camden Road

Nos. 5 and 5a Camden Road are part of a three storey terraced building, including a cellar that lies immediately to the southeast of the site.

The party walls to these buildings appear to be supported by strip foundations situated at approximately 1.8m depth below ground floor level.



Structures Assessed for Ground Movement (basement extent highlighted in yellow)

7.2 Modelled Ground Conditions

Excavation of the basement will result in net unloading of the clay leading to heave movement of the underlying soil in both the short and long term.

An analysis of the vertical movements has been carried out using the soil stiffness model detailed in the table below:

Stratum:	Undrained Elastic Modulus Eu (kN/m^2)	Drained Elastic Modulus E' (kN/m^2)
London Clay Formation	52,500kN/m^2 at surface increasing linearly to 232,500kN/m^2 at 30m depth	35,000 kN/m^2 at surface increasing linearly to 155,000kN/m^2 at 30m depth

Poisson's Ratios of 0.5 and 0.2 have been used for short term (undrained) and long term (drained) conditions respectively.

The analysis uses classic modified Boussinesq elastic theory, assuming uniform loading/unloading applied to a semi-infinite elastic half-space, using the above parameters for stratified homogeneity and with the introduction of an assumed rigid boundary at approximately 30m depth.

In order to represent a worst case scenario, the party walls to Nos. 8 & 16 Kentish Town Road, as well as No. 5a Camden Road are assumed to be is supported by footings situated at 1m depth below ground floor level.

7.3 Short Term Vertical Movements

There are two components of short term movement that will interact to affect the neighbouring structures.

These components are firstly progressive sagging movements of the underpinned walls due to imperfections in the underpinning process itself and then secondly elastic heave of the ground as a direct response to the unloading caused by excavation of the new basement. It is envisaged that the basement excavation will extend to approximately 4m depth beneath the existing ground floor level.

As a result, the potential effect of the excavation may be considered by a net unloading of -80kN/m^2 due to soil unloading.

7.3.1 Short Term Movement due to Underpinning

It is not possible to rigorously model the extent of party wall settlement arising from underpinning and experience indicates that amount of any movements are very much dependent on workmanship. However, it is suggested that given dry conditions and good workmanship, the amount of vertical movement of the party walls can reasonably be expected to be a maximum of 5mm per stage of underpinning. On the simplistic assumption of a 45 degree angle of support to any walls extending away in a direction perpendicular to the party walls, the scale of this vertical movement associated with the underpinning process itself is assumed to extend to a distance of 4m behind the wall.

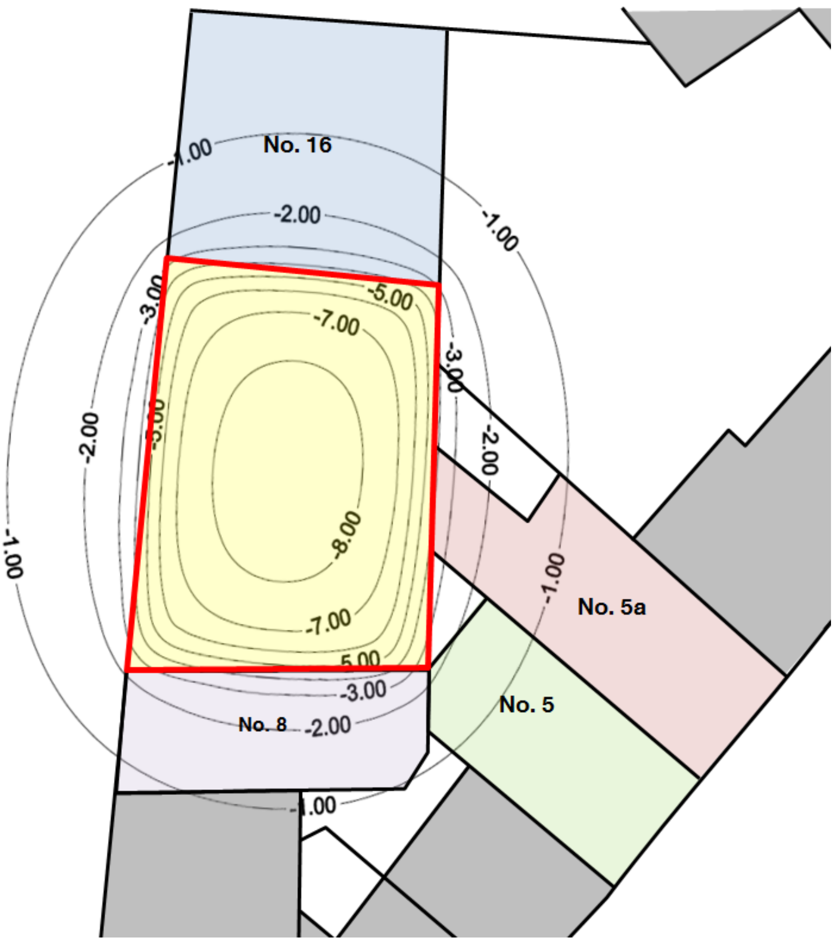
7.3.2 Short Term Movements due to Excavation heave

Approximately 10mm of short term soil heave is predicted at the centre of the basement excavation, reducing to less than 5mm beneath the party walls to No. 8 & 16 Kentish Town Road and No. 5a Camden Road.

7.4 Post Construction Vertical Movements

There will be a mismatch between the weight of soil that is removed and the weight of the new structure. In this situation, a component of long term heave that could proceed for decades is inevitable.

The results of heave analysis, as presented on the plan shown below, suggest that the scale of this additional long term heave will potentially amount to 10mm beneath the centre of the basement. This decreases to approximately 5mm beneath the party walls to No. 8 & 16 Kentish Town Road and No. 5a Camden Road. Less than 3mm of long term heave is predicted at No. 5 Camden Road.



Plan showing theoretical approximate post-construction heave (mm) due to basement excavation

7.5 Horizontal Movements

Horizontal soil movements are expected to occur due to yielding of the soil behind the underpinned wall during the basement excavation. For embedded retaining walls, this yielding has been found to extend to a distance approximately equivalent to four times the depth of excavation in front of the wall.

As a first approximation, the magnitude of the horizontal movement at the basement perimeter is assumed to be equal to the vertical movement of the underpinned wall, reducing to zero at a distance of 4 x 4m = 16m behind the wall.

On this basis 5mm of horizontal movement is predicted at the party walls, and these horizontal movements are assumed to decrease perpendicular from the underpinned wall on the basis of an assumed plane drawn upwards at an angle of 45° from the base of the excavation.

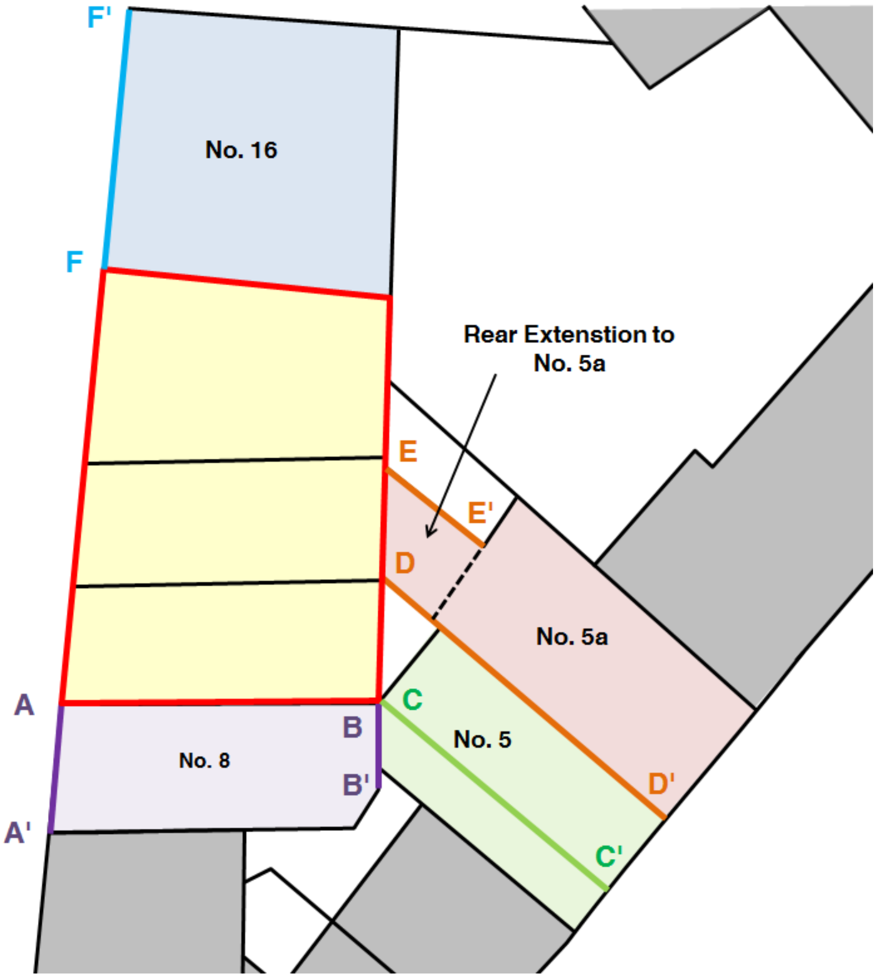
7.6 Impact on Neighbouring Structures

In practice although the various movements described above will interact so that the soil basement heave effects will tend to counteract the underpinning wall settlement movements, it is considered prudent to consider the worst case situation. Thus, the analysis of potential damage to neighbouring structures is based upon movement predictions that ignore basement soil heave.

The effect of these predicted vertical and horizontal deflections have been assessed using the Burland damage category assessment process, which is based upon consideration of a theoretical masonry panel of a given length (L) and height (H).

The potential degree of the predicted ground movements on the assessed structures can be estimated by the correlation of maximum horizontal strain, ε_h, with the maximum deflection ratio, Δ/L, where Δ is the vertical distortion over a the wall length under assessment. (Where the wall length L is actually less than the distance to the point at which zero vertical movement is assumed, a minimum distortion of 1mm is assumed.)

The potential degree of damage due to the proposed basement construction has been assessed for each neighbouring property using a series of sections and a summary for each is shown below.



Plan showing line of sections used for damage category assessment
(yellow colour indicates basement extent)

7.6.1 No. 8 Kentish Town Road – Long Party Wall Section (Section A-A')

The length of section is taken as 5.5m and the wall height as 10m.

The maximum horizontal strain, **δh (** L) is assessed as 0.03%, producing a maximum deflection ratio ∆ / L = -0.018, within a limiting tensile strain of 0.04%, for a Burland Category 0 "Negigible" condition.

7.6.2 No. 8 Kentish Town Road – Short Party Wall Section (Section B-B')

The length of section is taken as 3m and the wall height as 10m.

The maximum horizontal strain, **δh (** L) is assessed as 0.03%, producing a maximum deflection ratio ∆ / L = -0.033, within a limiting tensile strain of 0.05%, for a Burland Category 0 "Negigible" condition.

7.6.3 No. 5 Camden Road (Section C-C')

The length of section is taken as 12m and the wall height as 10m.

The maximum horizontal strain, **δh (** L) is assessed as 0.03%, producing a maximum deflection ratio Δ / L = -0.022, within a limiting tensile strain of 0.045%, for a Burland Category 0 “Negligible” condition.

7.6.4 No. 5a Camden Road – Main Building (Section D-D’)

The length of section is taken as 14m and the wall height as 9m.

The maximum horizontal strain, **δh (** L) is assessed as 0.03%, producing a maximum deflection ratio Δ / L = -0.020, within a limiting tensile strain of 0.05%, for a Burland Category 0 “Negligible” condition.

7.6.5 No. 5a Camden Road – Rear Extension (Section E-E’)

The length of section is taken as 5.5m and the wall height as 3m.

The maximum horizontal strain, **δh (** L) is assessed as 0.03%, producing a maximum deflection ratio Δ / L = -0.02, within a limiting tensile strain of 0.055%, for a Burland Category 1 “Very Slight” condition.

7.6.6 No. 16 Kentish Town Road – Main Building (Section F-F’)

The length of section is taken as 11m and the wall height as 3m.

The maximum horizontal strain, **δh (** L) is assessed as 0.03%, producing a maximum deflection ratio Δ / L = -0.023, within a limiting tensile strain of 0.065%, for a Burland Category 1 “Very Slight” condition.

7.6.7 Public Highway

The proposed basement lies directly adjacent to the pavement and there are various buried utilities located in this area.

However, given reasonable standards of workmanship during the underpinning works, negligible movement (<5mm settlement) is anticipated and this may be counteracted in practice by some small amounts of heave. The northern line Camden Town Station lies directly beneath the pavement in this area, and that is the subject of a separate assessment for TfL.

7.7 Structural Monitoring

The Camden Local Plan (June 2017) states that the BIA must demonstrate that the basement scheme has a risk of damage to the neighbouring properties no higher than Burland Scale 1 (very slight).

Given the possibility of up to Category 1 damage to neighbouring structures, structural monitoring of the relevant sections should be undertaken to ensure the movements remain within acceptable limits and to enable mitigation to be effectively implemented in the event of agreed trigger values for movement being exceeded.

During all underpinning works and basement excavation works, monitoring should be undertaken daily at the start and end of every work shift. At other times monitoring should be undertaken weekly to cover a period prior to commencement of any works and ceasing after completion of the works, by agreement of all interested parties.

Precise survey equipment should be used to record all vertical and horizontal components of movement (in three perpendicular directions) to a minimum accuracy of 1mm.

7.7.1 Criteria for assessment of Monitoring data and Comparison with Predicted Movements

The cumulative movements in any direction of any monitoring point are to be compared with the predicted movements at any stage and using the following decision table:

MONITORING CRITERIA		
Total movement less than 5mm in any direction		Green
Total movement in excess of 5mm in any direction or additional movement of 5mm in any direction	Notify Structural Engineer and Party Wall Surveyor	Red

7.7.2 Contingent Actions

Contingency actions should be undertaken using the following decision table:

CONTINGENT ACTIONS	
Green	None
Red	Cease work and Notify Structural Engineer and Party Wall Surveyor immediately. Commence backfilling / installation of additional propping. Undertake repeated monitoring as necessary to ensure that movement has ceased. Works to commence only once a revised construction methodology has been agreed with the Structural Engineer

8. Impact Assessment

The screening and scoping stages have identified potential effects of the development on those attributes or features of the geological, hydrogeological and hydrological environment. This stage is concerned with evaluating the direct and indirect implications of each of these potential impacts.

8.1 Potential Hydrogeological Impacts

No groundwater table is present at the site therefore, the development is not expected to have any impact upon groundwater flow and there is additionally expected to be no cumulative impact.

8.2 Potential Hydrological Impacts

There will be no change to the flood risk at the site or at neighbouring sites as a result of the proposed basement. A Flood Risk Assessment (FRA) & SUDS Statement is presented as a separate report (LBH4536fra Ver. 0.1).

Although the FRA indicates that the site is at a very low risk of sewer flooding, in order to ensure the basement is protected from sewer flooding, the basement drainage design will include a positive pumped chamber and non-return valve. As a result, any flood water will be directed away from the basement and will prevent the possibility of basement flooding through the drainage system.

8.3 Potential Stability Impacts

8.3.1 Public Highway

Negligible movement (<5mm settlement) is anticipated beneath the pavement and Kentish Town Road. The northern line Camden Town Station lies directly beneath the pavement in this area, and that is the subject of a separate assessment as described in 8.3.3 below.

8.3.2 London Clay

The London Clay soils beneath the site are suggested to be of high volume change potential.

However, the depth of the proposed construction will obviate concerns regarding potential seasonal shrink/swell movements.

8.3.3 Tunnels

An Asset Impact Assessment is currently being prepared for TfL to ensure that the proposed scheme will not adversely impact the underlying LUL northern line tunnel.

8.3.4 Ground Movements

The Local Plan states that proposed basements should pose a risk of damage to neighbouring properties no higher than Burland scale Category 1 ‘Very Slight’, and mitigation measures should be incorporated if the assessed damage is not acceptable.

The predicted building damage levels due to ground movements associated with the proposed development have been analysed and found to be acceptable

8.4 Residual Impacts

It is concluded that the proposed basement will have no residual unacceptable impacts upon the surrounding structures, infrastructure and environment. No cumulative impacts are envisaged.

9. Conclusion

No adverse residual or cumulative stability, hydrological or hydrogeological impacts are expected as a result of this development.

It is concluded that the proposed development will not cause harm to its neighbours or the wider environment and it has been demonstrated to comply with the requirements of Camden Local Plan Policy A5 in terms of protection of the local structural, hydrological and hydrogeological environment.

Appendix

Audit Query Tracker

Structural Calculations

Factual Site Investigation

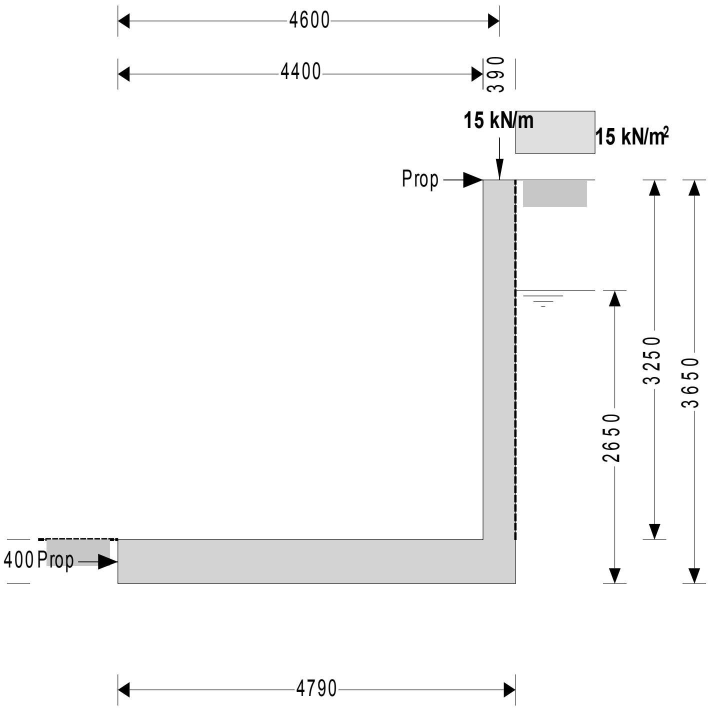
Outline Construction Programme

Audit Query Tracker

Audit Query No.	CRH Query	LBH Response	Location in BIA version 1.2	Status
1	Factual site investigation data should be presented	Factual SI is now attached	Appendix	Addressed
2	Confirm geotechnical interpretation including derivation of retaining wall design parameters and groundwater assumptions	These are now included	Sections 6.6 & 6.7	Addressed
3	An outline construction programme should be presented	A programme is now attached	Appendix	Addressed
4	Outline permanent and temporary works information should be provided	Calculations are now included	Appendix	Addressed
5	GMA strain calculations should be presented. It is stated that ground movements may impact the highway and Category 2 damage is predicted to 8 KTR. Mitigation measures should be provided.	The GMA has been reviewed and negligible impact on the highway is now predicted together with maximum Category 1 damage to 8KTR. This is compliant with the CPG and no mitigation is hence necessary.	Sections 7 and 8	Addressed
6	Thames Water indicates that there may be a risk from sewer flooding. The BIA should confirm that suitable mitigation is to be incorporated into the development.	This is now confirmed in the BIA	Section 8.2	Addressed
7	It is stated that ground movements may impact the highway and a structural monitoring scheme will be instigated to prevent any unacceptable damage. The assessment and proposed monitoring scheme should be provided.	This has been reviewed and given reasonable standards of workmanship during the underpinning works, negligible impact is now anticipated.	Section 8.3.1	Addressed

Calculations

RETAINING WALL ANALYSIS - FRONT 390MM



Wall details

Retaining wall type;	**Cantilever propped at both**
Height of retaining wall stem;	h_{stem} = **3250** mm
Thickness of wall stem;	t_{wall} = **390** mm
Length of toe;	l_{toe} = **4400** mm
Length of heel;	l_{heel} = **0** mm
Overall length of base;	$l_{base} = l_{toe} + l_{heel} + t_{wall}$ = **4790** mm
Thickness of base;	t_{base} = **400** mm
Depth of downstand;	d_{ds} = **0** mm
Position of downstand;	l_{ds} = **1900** mm
Thickness of downstand;	t_{ds} = **400** mm
Height of retaining wall;	$h_{wall} = h_{stem} + t_{base} + d_{ds}$ = **3650** mm
Depth of cover in front of wall;	d_{cover} = **0** mm
Depth of unplanned excavation;	d_{exc} = **0** mm
Height of ground water behind wall;	h_{water} = **2650** mm
Height of saturated fill above base;	$h_{sat} = \max(h_{water} - t_{base} - d_{ds}, 0\text{ mm})$ = **2250** mm
Density of wall construction;	γ_{wall} = **25.0** kN/m^3
Density of base construction;	γ_{base} = **25.0** kN/m^3
Angle of rear face of wall;	α = **90.0** deg
Angle of soil surface behind wall;	β = **0.0** deg
Effective height at virtual back of wall;	$h_{eff} = h_{wall} + l_{heel} \times \tan(\beta)$ = **3650** mm

Retained material details

Mobilisation factor;	M = **2.0**
Moist density of retained material;	γ_m = **19.0** kN/m^3
Saturated density of retained material;	γ_s = **19.0** kN/m^3

Design shear strength; $\phi' = \mathbf{25.0}$ deg

Angle of wall friction; $\delta = \mathbf{25.0}$ deg

Base material details

Moist density; $\gamma_{mb} = \mathbf{19.0}$ kN/m^3

Design shear strength; $\phi'_b = \mathbf{19.0}$ deg

Design base friction; $\delta_b = \mathbf{25.0}$ deg

Allowable bearing pressure; $P_{bearing} = \mathbf{150}$ kN/m^2

Using Coulomb theory

Active pressure coefficient for retained material

$$K_a = \sin(\alpha + \phi')^2 / (\sin(\alpha)^2 \times \sin(\alpha - \delta) \times [1 + \sqrt{(\sin(\phi' + \delta) \times \sin(\phi' - \beta) / (\sin(\alpha - \delta) \times \sin(\alpha + \beta)))}]^2) = \mathbf{0.355}$$

Passive pressure coefficient for base material

$$K_p = \sin(90 - \phi'_b)^2 / (\sin(90 - \delta_b) \times [1 - \sqrt{(\sin(\phi'_b + \delta_b) \times \sin(\phi'_b) / (\sin(90 + \delta_b)))}]^2) = \mathbf{3.938}$$

At-rest pressure

At-rest pressure for retained material; $K_0 = 1 - \sin(\phi') = \mathbf{0.577}$

Loading details

Surcharge load on plan; Surcharge = **15.0** kN/m^2

Applied vertical dead load on wall; $W_{dead} = \mathbf{10.0}$ kN/m

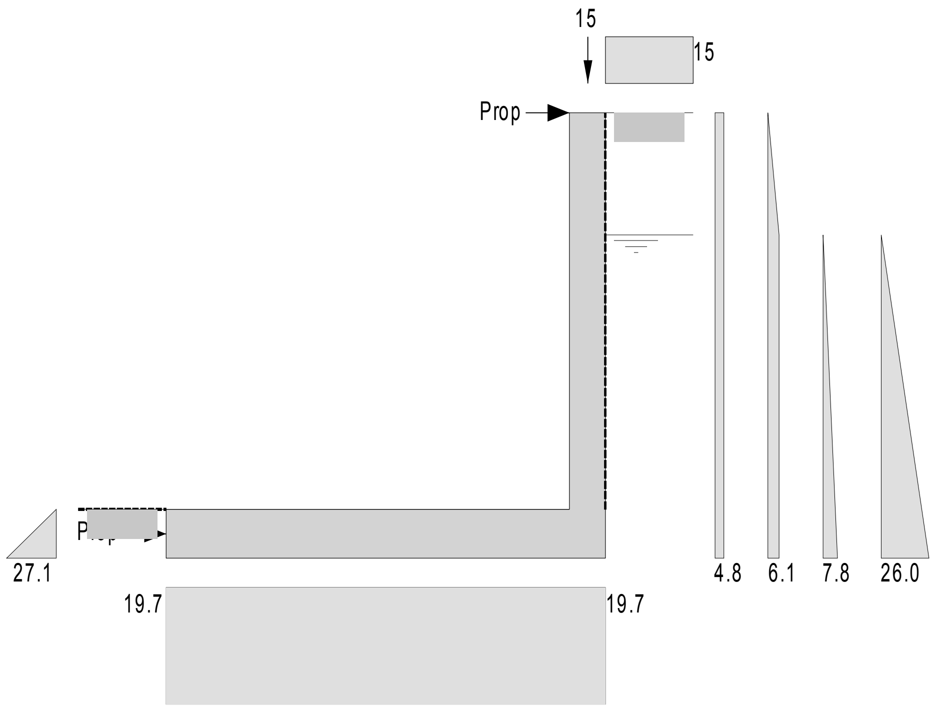
Applied vertical live load on wall; $W_{live} = \mathbf{5.0}$ kN/m

Position of applied vertical load on wall; $l_{load} = \mathbf{4600}$ mm

Applied horizontal dead load on wall; $F_{dead} = \mathbf{0.0}$ kN/m

Applied horizontal live load on wall; $F_{live} = \mathbf{0.0}$ kN/m

Height of applied horizontal load on wall; $h_{load} = \mathbf{0}$ mm



Loads shown in kN/m, pressures shown in kN/m^2

Vertical forces on wall

Wall stem; $w_{wall} = h_{stem} \times t_{wall} \times \gamma_{wall} = \mathbf{31.7}$ kN/m

Wall base; $w_{base} = l_{base} \times t_{base} \times \gamma_{base} = \mathbf{47.9}$ kN/m

Applied vertical load; $W_v = W_{dead} + W_{live} = \mathbf{15}$ kN/m

Total vertical load; $W_{total} = w_{wall} + w_{base} + W_v = \mathbf{94.6}$ kN/m

Horizontal forces on wall

Surcharge; $F_{sur} = K_a \times \cos(90 - \alpha + \delta) \times Surcharge \times h_{eff}$ = **17.6** kN/m

Moist backfill above water table; $F_{m\_a} = 0.5 \times K_a \times \cos(90 - \alpha + \delta) \times \gamma_m \times (h_{eff} - h_{water})^2$ = **3.1** kN/m

Moist backfill below water table; $F_{m\_b} = K_a \times \cos(90 - \alpha + \delta) \times \gamma_m \times (h_{eff} - h_{water}) \times h_{water}$ = **16.2** kN/m

Saturated backfill; $F_s = 0.5 \times K_a \times \cos(90 - \alpha + \delta) \times (\gamma_s - \gamma_{water}) \times h_{water}^2$ = **10.4** kN/m

Water; $F_{water} = 0.5 \times h_{water}^2 \times \gamma_{water}$ = **34.4** kN/m

Total horizontal load; $F_{total} = F_{sur} + F_{m\_a} + F_{m\_b} + F_s + F_{water}$ = **81.7** kN/m

Calculate total propping force

Passive resistance of soil in front of wall; $F_p = 0.5 \times K_p \times \cos(\delta_b) \times (d_{cover} + t_{base} + d_{ds} - d_{exc})^2 \times \gamma_{mb}$ = **5.4** kN/m

Propping force; $F_{prop} = \max(F_{total} - F_p - (W_{total} - W_{live}) \times \tan(\delta_b), 0\text{ kN/m})$

F_{prop} = **34.5** kN/m

Overturning moments

Surcharge; $M_{sur} = F_{sur} \times (h_{eff} - 2 \times d_{ds}) / 2$ = **32.2** kNm/m

Moist backfill above water table; $M_{m\_a} = F_{m\_a} \times (h_{eff} + 2 \times h_{water} - 3 \times d_{ds}) / 3$ = **9.1** kNm/m

Moist backfill below water table; $M_{m\_b} = F_{m\_b} \times (h_{water} - 2 \times d_{ds}) / 2$ = **21.5** kNm/m

Saturated backfill; $M_s = F_s \times (h_{water} - 3 \times d_{ds}) / 3$ = **9.2** kNm/m

Water; $M_{water} = F_{water} \times (h_{water} - 3 \times d_{ds}) / 3$ = **30.4** kNm/m

Total overturning moment; $M_{ot} = M_{sur} + M_{m\_a} + M_{m\_b} + M_s + M_{water}$ = **102.3** kNm/m

Restoring moments

Wall stem; $M_{wall} = w_{wall} \times (l_{toe} + t_{wall} / 2)$ = **145.6** kNm/m

Wall base; $M_{base} = w_{base} \times l_{base} / 2$ = **114.7** kNm/m

Design vertical dead load; $M_{dead} = W_{dead} \times l_{load}$ = **46** kNm/m

Total restoring moment; $M_{rest} = M_{wall} + M_{base} + M_{dead}$ = **306.3** kNm/m

Check bearing pressure

Total vertical reaction; $R = W_{total}$ = **94.6** kN/m

Distance to reaction; $x_{bar} = l_{base} / 2$ = **2395** mm

Eccentricity of reaction; $e = \text{abs}((l_{base} / 2) - x_{bar})$ = **0** mm

Reaction acts within middle third of base

Bearing pressure at toe; $p_{toe} = (R / l_{base}) - (6 \times R \times e / l_{base}^2)$ = **19.7** kN/m^2

Bearing pressure at heel; $p_{heel} = (R / l_{base}) + (6 \times R \times e / l_{base}^2)$ = **19.7** kN/m^2

PASS - Maximum bearing pressure is less than allowable bearing pressure

Calculate propping forces to top and base of wall

Propping force to top of wall

$F_{prop\_top} = (M_{ot} - M_{rest} + R \times l_{base} / 2 - F_{prop} \times t_{base} / 2) / (h_{stem} + t_{base} / 2)$ = **4.537** kN/m

Propping force to base of wall; $F_{prop\_base} = F_{prop} - F_{prop\_top}$ = **29.969** kN/m

RETAINING WALL DESIGN

Ultimate limit state load factors

Dead load factor; γ_{f_d} = **1.4**

Live load factor; γ_{f_l} = **1.6**

Earth and water pressure factor; γ_{f_e} = **1.4**

Factored vertical forces on wall

Wall stem; $w_{wall\_f} = \gamma_{f\_d} \times h_{stem} \times t_{wall} \times \gamma_{wall}$ = **44.4** kN/m

Wall base; $w_{base\_f} = \gamma_{f\_d} \times l_{base} \times t_{base} \times \gamma_{base}$ = **67.1** kN/m

Applied vertical load; $W_{v\_f} = \gamma_{f\_d} \times W_{dead} + \gamma_{f\_l} \times W_{live}$ = **22** kN/m

Total vertical load; $W_{total\_f} = w_{wall\_f} + w_{base\_f} + W_{v\_f}$ = **133.4** kN/m

Factored horizontal at-rest forces on wall

Surcharge; $F_{sur\_f} = \gamma_{f\_l} \times K_0 \times \text{Surcharge} \times h_{eff}$ = **50.6** kN/m

Moist backfill above water table; $F_{m\_a\_f} = \gamma_{f\_e} \times 0.5 \times K_0 \times \gamma_m \times (h_{eff} - h_{water})^2$ = **7.7** kN/m

Moist backfill below water table; $F_{m\_b\_f} = \gamma_{f\_e} \times K_0 \times \gamma_m \times (h_{eff} - h_{water}) \times h_{water}$ = **40.7** kN/m

Saturated backfill; $F_{s\_f} = \gamma_{f\_e} \times 0.5 \times K_0 \times (\gamma_s - \gamma_{water}) \times h_{water}^2$ = **26.1** kN/m

Water; $F_{water\_f} = \gamma_{f\_e} \times 0.5 \times h_{water}^2 \times \gamma_{water}$ = **48.2** kN/m

Total horizontal load; $F_{total\_f} = F_{sur\_f} + F_{m\_a\_f} + F_{m\_b\_f} + F_{s\_f} + F_{water\_f}$ = **173.3** kN/m

Calculate total propping force

Passive resistance of soil in front of wall; $F_{p\_f} = \gamma_{f\_e} \times 0.5 \times K_p \times \cos(\delta_b) \times (d_{cover} + t_{base} + d_{ds} - d_{exc})^2 \times \gamma_{mb}$
= **7.6** kN/m

Propping force; $F_{prop\_f} = \max(F_{total\_f} - F_{p\_f} - (W_{total\_f} - \gamma_{f\_l} \times W_{live}) \times \tan(\delta_b)$, 0
kN/m)

F_{prop_f} = **107.2** kN/m

Factored overturning moments

Surcharge; $M_{sur\_f} = F_{sur\_f} \times (h_{eff} - 2 \times d_{ds}) / 2$ = **92.3** kNm/m

Moist backfill above water table; $M_{m\_a\_f} = F_{m\_a\_f} \times (h_{eff} + 2 \times h_{water} - 3 \times d_{ds}) / 3$ = **22.9** kNm/m

Moist backfill below water table; $M_{m\_b\_f} = F_{m\_b\_f} \times (h_{water} - 2 \times d_{ds}) / 2$ = **53.9** kNm/m

Saturated backfill; $M_{s\_f} = F_{s\_f} \times (h_{water} - 3 \times d_{ds}) / 3$ = **23** kNm/m

Water; $M_{water\_f} = F_{water\_f} \times (h_{water} - 3 \times d_{ds}) / 3$ = **42.6** kNm/m

Total overturning moment; $M_{ot\_f} = M_{sur\_f} + M_{m\_a\_f} + M_{m\_b\_f} + M_{s\_f} + M_{water\_f}$ = **234.8** kNm/m

Restoring moments

Wall stem; $M_{wall\_f} = w_{wall\_f} \times (l_{toe} + t_{wall} / 2)$ = **203.8** kNm/m

Wall base; $M_{base\_f} = w_{base\_f} \times l_{base} / 2$ = **160.6** kNm/m

Design vertical load; $M_{v\_f} = W_{v\_f} \times l_{load}$ = **101.2** kNm/m

Total restoring moment; $M_{rest\_f} = M_{wall\_f} + M_{base\_f} + M_{v\_f}$ = **465.7** kNm/m

Factored bearing pressure

Total vertical reaction; $R_f = W_{total\_f}$ = **133.4** kN/m

Distance to reaction; $x_{bar\_f} = l_{base} / 2$ = **2395** mm

Eccentricity of reaction; $e_f = \text{abs}((l_{base} / 2) - x_{bar\_f})$ = **0** mm

Reaction acts within middle third of base

Bearing pressure at toe; $p_{toe\_f} = (R_f / l_{base}) - (6 \times R_f \times e_f / l_{base}^2)$ = **27.9** kN/m²

Bearing pressure at heel; $p_{heel\_f} = (R_f / l_{base}) + (6 \times R_f \times e_f / l_{base}^2)$ = **27.9** kN/m²

Rate of change of base reaction; $\text{rate} = (p_{toe\_f} - p_{heel\_f}) / l_{base}$ = **0.00** kN/m²/m

Bearing pressure at stem / toe; $p_{stem\_toe\_f} = \max(p_{toe\_f} - (\text{rate} \times l_{toe}), 0 \text{ kN/m}^2)$ = **27.9** kN/m²

Bearing pressure at mid stem; $p_{stem\_mid\_f} = \max(p_{toe\_f} - (\text{rate} \times (l_{toe} + t_{wall} / 2)), 0 \text{ kN/m}^2)$ = **27.9**
kN/m²

Bearing pressure at stem / heel; $p_{stem\_heel\_f} = \max(p_{toe\_f} - (rate \times (l_{toe} + t_{wall})), 0 \text{ kN/m}^2) = \mathbf{27.9}$ kN/m²

Calculate propping forces to top and base of wall

Propping force to top of wall

$F_{prop\_top\_f} = (M_{ot\_f} - M_{rest\_f} + R_f \times l_{base} / 2 - F_{prop\_f} \times t_{base} / 2) / (h_{stem} + t_{base} / 2) = \mathbf{19.489}$ kN/m

Propping force to base of wall; $F_{prop\_base\_f} = F_{prop\_f} - F_{prop\_top\_f} = \mathbf{87.695}$ kN/m

Design of reinforced concrete retaining wall toe

Material properties

Characteristic strength of concrete; $f_{cu} = \mathbf{40}$ N/mm²

Characteristic strength of reinforcement; $f_y = \mathbf{500}$ N/mm²

Base details

Minimum area of reinforcement; $k = \mathbf{0.13}$ %

Cover to reinforcement in toe; $c_{toe} = \mathbf{50}$ mm

Calculate shear for toe design

Shear from bearing pressure; $V_{toe\_bear} = (p_{toe\_f} + p_{stem\_toe\_f}) \times l_{toe} / 2 = \mathbf{122.6}$ kN/m

Shear from weight of base; $V_{toe\_wt\_base} = \gamma_{f\_d} \times \gamma_{base} \times l_{toe} \times t_{base} = \mathbf{61.6}$ kN/m

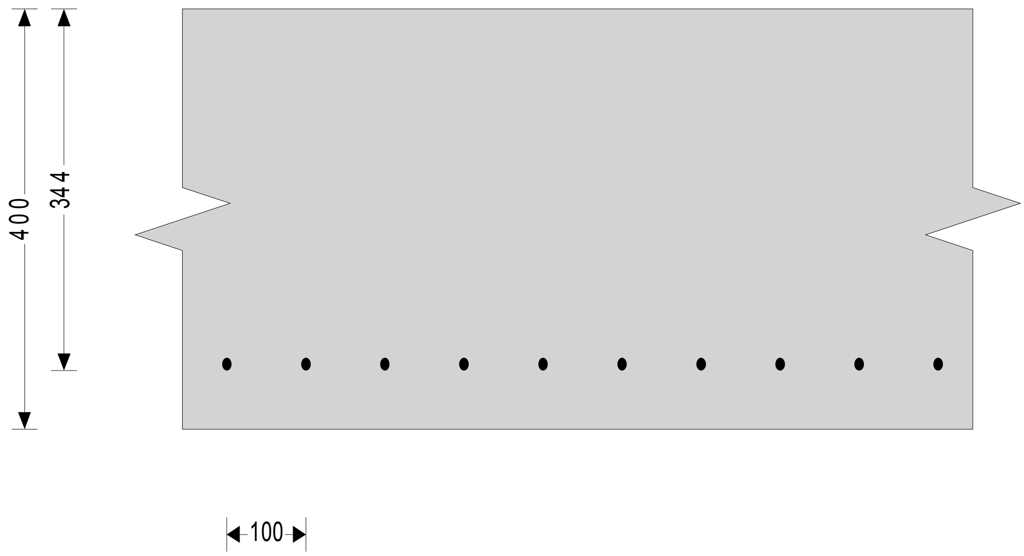
Total shear for toe design; $V_{toe} = V_{toe\_bear} - V_{toe\_wt\_base} = \mathbf{61}$ kN/m

Calculate moment for toe design

Moment from bearing pressure; $M_{toe\_bear} = (2 \times p_{toe\_f} + p_{stem\_mid\_f}) \times (l_{toe} + t_{wall} / 2)^2 / 6 = \mathbf{294.1}$ kNm/m

Moment from weight of base; $M_{toe\_wt\_base} = (\gamma_{f\_d} \times \gamma_{base} \times t_{base} \times (l_{toe} + t_{wall} / 2)^2 / 2) = \mathbf{147.8}$ kNm/m

Total moment for toe design; $M_{toe} = M_{toe\_bear} - M_{toe\_wt\_base} = \mathbf{146.3}$ kNm/m



Check toe in bending

Width of toe; $b = \mathbf{1000}$ mm/m

Depth of reinforcement; $d_{toe} = t_{base} - c_{toe} - (\phi_{toe} / 2) = \mathbf{344.0}$ mm

Constant; $K_{toe} = M_{toe} / (b \times d_{toe}^2 \times f_{cu}) = \mathbf{0.031}$

Compression reinforcement is not required

Lever arm; $z_{toe} = \min(0.5 + \sqrt{(0.25 - (\min(K_{toe}, 0.225) / 0.9))}, 0.95) \times d_{toe}$

$z_{toe} = \mathbf{327}$ mm

Area of tension reinforcement required; $A_{s\_toe\_des} = M_{toe} / (0.87 \times f_y \times z_{toe}) = \mathbf{1029}$ mm²/m

Minimum area of tension reinforcement; $A_{s\_toe\_min} = k \times b \times t_{base} = \mathbf{520}$ mm²/m

Area of tension reinforcement required; $A_{s\_toe\_req}$ = Max($A_{s\_toe\_des}$, $A_{s\_toe\_min}$) = **1029** mm^2/m
Reinforcement provided; **12 mm dia.bars @ 100 mm centres**
Area of reinforcement provided; $A_{s\_toe\_prov}$ = **1131** mm^2/m

PASS - Reinforcement provided at the retaining wall toe is adequate

Check shear resistance at toe

Design shear stress; $v_{toe} = V_{toe} / (b \times d_{toe})$ = **0.177** N/mm^2
Allowable shear stress; $v_{adm} = \min(0.8 \times \sqrt{(f_{cu} / 1 \text{ N/mm}^2)}, 5) \times 1 \text{ N/mm}^2$ = **5.000** N/mm^2

PASS - Design shear stress is less than maximum shear stress

From BS8110:Part 1:1997 – Table 3.8

Design concrete shear stress; v_{c_toe} = **0.530** N/mm^2

$v_{toe} < v_{c\_toe}$ - No shear reinforcement required

Design of reinforced concrete retaining wall stem

Material properties

Characteristic strength of concrete; f_{cu} = **40** N/mm^2
Characteristic strength of reinforcement; f_y = **500** N/mm^2

Wall details

Minimum area of reinforcement; k = **0.13** %
Cover to reinforcement in stem; c_{stem} = **40** mm
Cover to reinforcement in wall; c_{wall} = **40** mm

Factored horizontal at-rest forces on stem

Surcharge; $F_{s\_sur\_f} = \gamma_{f\_l} \times K_0 \times \text{Surcharge} \times (h_{eff} - t_{base} - d_{ds})$ = **45** kN/m
Moist backfill above water table; $F_{s\_m\_a\_f} = 0.5 \times \gamma_{f\_e} \times K_0 \times \gamma_m \times (h_{eff} - t_{base} - d_{ds} - h_{sat})^2$ = **7.7** kN/m
Moist backfill below water table; $F_{s\_m\_b\_f} = \gamma_{f\_e} \times K_0 \times \gamma_m \times (h_{eff} - t_{base} - d_{ds} - h_{sat}) \times h_{sat}$ = **34.6** kN/m
Saturated backfill; $F_{s\_s\_f} = 0.5 \times \gamma_{f\_e} \times K_0 \times (\gamma_s - \gamma_{water}) \times h_{sat}^2$ = **18.8** kN/m
Water; $F_{s\_water\_f} = 0.5 \times \gamma_{f\_e} \times \gamma_{water} \times h_{sat}^2$ = **34.8** kN/m

Calculate shear for stem design

Surcharge; $V_{s\_sur\_f} = 5 \times F_{s\_sur\_f} / 8$ = **28.1** kN/m
Moist backfill above water table; $V_{s\_m\_a\_f} = F_{s\_m\_a\_f} \times b_l \times ((5 \times L^2) - b_l^2) / (5 \times L^3)$ = **2.2** kN/m
Moist backfill below water table; $V_{s\_m\_b\_f} = F_{s\_m\_b\_f} \times (8 - (n^2 \times (4 - n))) / 8$ = **27.4** kN/m
Saturated backfill; $V_{s\_s\_f} = F_{s\_s\_f} \times (1 - (a_l^2 \times ((5 \times L) - a_l) / (20 \times L^3)))$ = **16.8** kN/m
Water; $V_{s\_water\_f} = F_{s\_water\_f} \times (1 - (a_l^2 \times ((5 \times L) - a_l) / (20 \times L^3)))$ = **31** kN/m
Total shear for stem design; $V_{stem} = V_{s\_sur\_f} + V_{s\_m\_a\_f} + V_{s\_m\_b\_f} + V_{s\_s\_f} + V_{s\_water\_f}$ = **105.5** kN/m

Calculate moment for stem design

Surcharge; $M_{s\_sur} = F_{s\_sur\_f} \times L / 8$ = **19.4** kNm/m
Moist backfill above water table; $M_{s\_m\_a} = F_{s\_m\_a\_f} \times b_l \times ((5 \times L^2) - (3 \times b_l^2)) / (15 \times L^2)$ = **2.4** kNm/m
Moist backfill below water table; $M_{s\_m\_b} = F_{s\_m\_b\_f} \times a_l \times (2 - n)^2 / 8$ = **17.6** kNm/m
Saturated backfill; $M_{s\_s} = F_{s\_s\_f} \times a_l \times ((3 \times a_l^2) - (15 \times a_l \times L) + (20 \times L^2)) / (60 \times L^2)$ = **8.3** kNm/m
Water; $M_{s\_water} = F_{s\_water\_f} \times a_l \times ((3 \times a_l^2) - (15 \times a_l \times L) + (20 \times L^2)) / (60 \times L^2)$ = **15.4** kNm/m
Total moment for stem design; $M_{stem} = M_{s\_sur} + M_{s\_m\_a} + M_{s\_m\_b} + M_{s\_s} + M_{s\_water}$ = **63.2** kNm/m

Calculate moment for wall design

Surcharge; $M_{w\_sur} = 9 \times F_{s\_sur\_f} \times L / 128 =$ **10.9** kNm/m

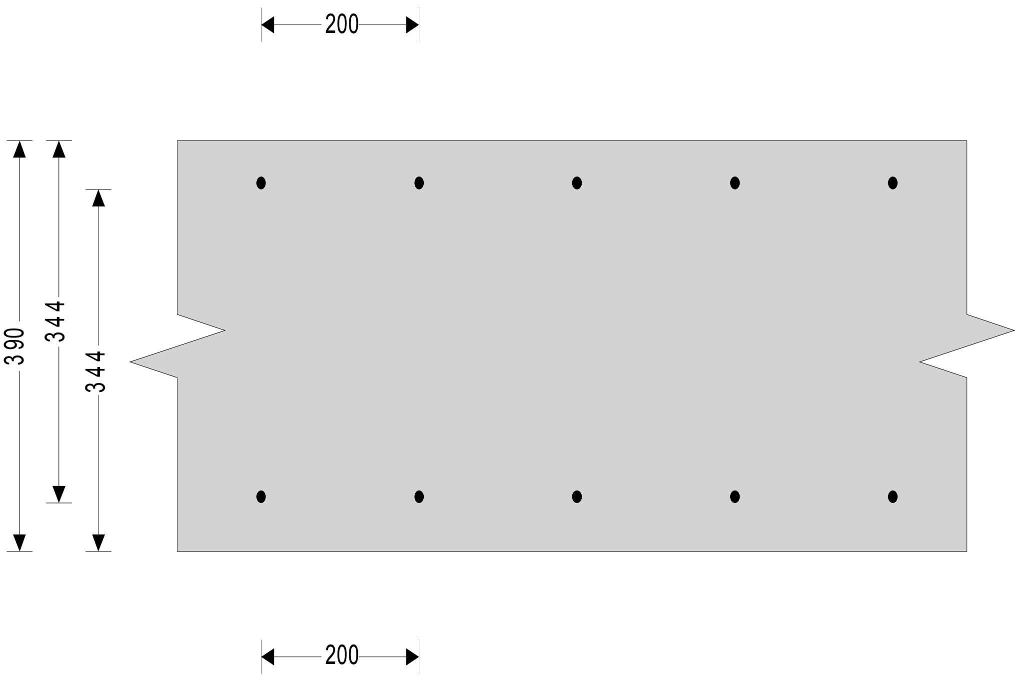
Moist backfill above water table; $M_{w\_m\_a} = F_{s\_m\_a\_f} \times 0.577 \times b_l \times [(b_l^3+5 \times a_l \times L^2)/(5 \times L^3)-0.577^2/3] =$ **2.7** kNm/m

Moist backfill below water table; $M_{w\_m\_b} = F_{s\_m\_b\_f} \times a_l \times [((8-n^2 \times (4-n))^2 /16)-4+n \times (4-n)]/8 =$ **9** kNm/m

Saturated backfill; $M_{w\_s} = F_{s\_s\_f} \times [a_l^2 \times x \times ((5 \times L)-a_l)/(20 \times L^3)-(x-b_l)^3 /(3 \times a_l^2)] =$ **3.1** kNm/m

Water; $M_{w\_water} = F_{s\_water\_f} \times [a_l^2 \times x \times ((5 \times L)-a_l)/(20 \times L^3)-(x-b_l)^3 /(3 \times a_l^2)] =$ **5.8** kNm/m

Total moment for wall design; $M_{wall} = M_{w\_sur} + M_{w\_m\_a} + M_{w\_m\_b} + M_{w\_s} + M_{w\_water} =$ **31.5** kNm/m



Check wall stem in bending

Width of wall stem; b = **1000** mm/m

Depth of reinforcement; $d_{stem} = t_{wall} - c_{stem} - (\phi_{stem} / 2) =$ **344.0** mm

Constant; $K_{stem} = M_{stem} / (b \times d_{stem}^2 \times f_{cu}) =$ **0.013**

Compression reinforcement is not required

Lever arm; $z_{stem} = \min(0.5 + \sqrt{(0.25 - (\min(K_{stem}, 0.225) / 0.9))},0.95) \times d_{stem}$

$z_{stem} =$ **327** mm

Area of tension reinforcement required; $A_{s\_stem\_des} = M_{stem} / (0.87 \times f_y \times z_{stem}) =$ **445** mm²/m

Minimum area of tension reinforcement; $A_{s\_stem\_min} = k \times b \times t_{wall} =$ **507** mm²/m

Area of tension reinforcement required; $A_{s\_stem\_req} = \mathrm{Max}(A_{s\_stem\_des}, A_{s\_stem\_min}) =$ **507** mm²/m

Reinforcement provided; **12 mm dia.bars @ 200 mm centres**

Area of reinforcement provided; $A_{s\_stem\_prov} =$ **565** mm²/m

PASS - Reinforcement provided at the retaining wall stem is adequate

Check shear resistance at wall stem

Design shear stress; $v_{stem} = V_{stem} / (b \times d_{stem}) =$ **0.307** N/mm²

Allowable shear stress; $v_{adm} = \min(0.8 \times \sqrt{(f_{cu} / 1\ N/mm^2)}, 5) \times 1\ N/mm^2 =$ **5.000** N/mm²

PASS - Design shear stress is less than maximum shear stress

From BS8110:Part 1:1997 – Table 3.8

Design concrete shear stress; v_{c_stem} = **0.420** N/mm^2

$v_{stem} < v_{c\_stem}$ - No shear reinforcement required

Check mid height of wall in bending

Depth of reinforcement; $d_{wall} = t_{wall} - c_{wall} - (\phi_{wall} / 2)$ = **344.0** mm

Constant; $K_{wall} = M_{wall} / (b \times d_{wall}^2 \times f_{cu})$ = **0.007**

Compression reinforcement is not required

Lever arm; $z_{wall} = \text{Min}(0.5 + \sqrt{(0.25 - (\min(K_{wall}, 0.225) / 0.9))}, 0.95) \times d_{wall}$

z_{wall} = **327** mm

Area of tension reinforcement required; $A_{s\_wall\_des} = M_{wall} / (0.87 \times f_y \times z_{wall})$ = **222** mm^2/m

Minimum area of tension reinforcement; $A_{s\_wall\_min} = k \times b \times t_{wall}$ = **507** mm^2/m

Area of tension reinforcement required; $A_{s\_wall\_req} = \text{Max}(A_{s\_wall\_des}, A_{s\_wall\_min})$ = **507** mm^2/m

Reinforcement provided; **12 mm dia.bars @ 200 mm centres**

Area of reinforcement provided; $A_{s\_wall\_prov}$ = **565** mm^2/m

PASS - Reinforcement provided to the retaining wall at mid height is adequate

Check retaining wall deflection

Basic span/effective depth ratio; $ratio_{bas}$ = **20**

Design service stress; $f_s = 2 \times f_y \times A_{s\_stem\_req} / (3 \times A_{s\_stem\_prov})$ = **298.9** N/mm^2

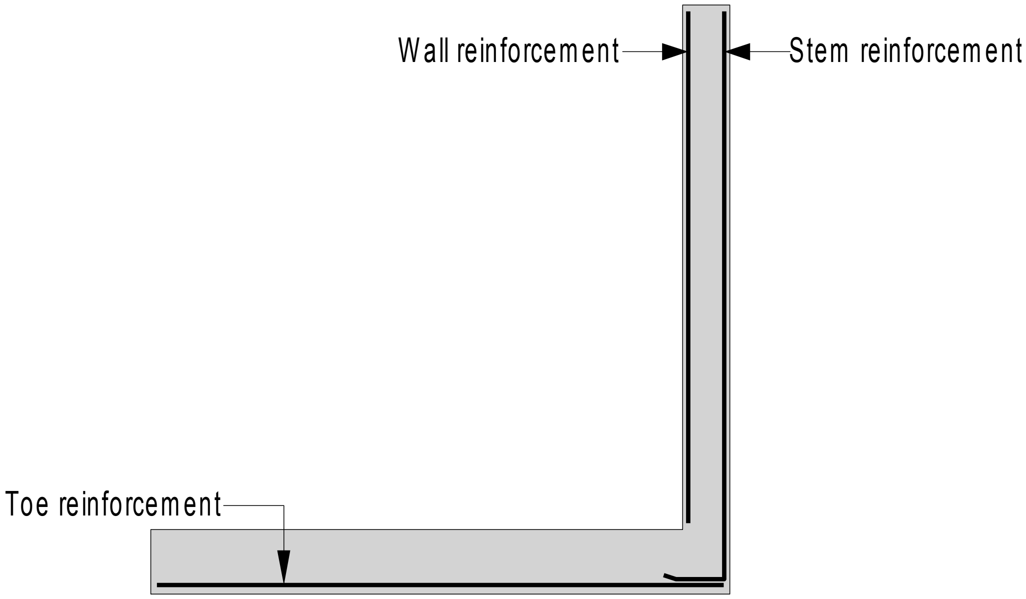
Modification factor; $factor_{tens} = \min(0.55 + (477 \text{ N/mm}^2 - f_s)/(120 \times (0.9 \text{ N/mm}^2 + (M_{stem}/(b \times d_{stem}^2)))),2)$ = **1.59**

Maximum span/effective depth ratio; $ratio_{max} = ratio_{bas} \times factor_{tens}$ = **31.70**

Actual span/effective depth ratio; $ratio_{act} = h_{stem} / d_{stem}$ = **9.45**

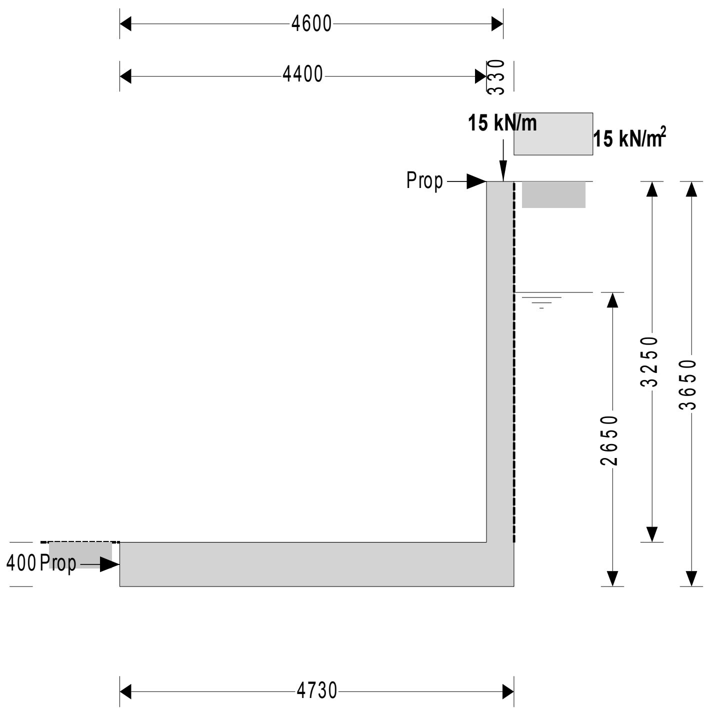
PASS - Span to depth ratio is acceptable

Indicative retaining wall reinforcement diagram



Toe bars - 12 mm dia.@ 100 mm centres - (1131 mm^2/m)
Wall bars - 12 mm dia.@ 200 mm centres - (565 mm^2/m)
Stem bars - 12 mm dia.@ 200 mm centres - (565 mm^2/m)

RETAINING WALL ANALYSIS – FRONT 330MM



Wall details

Retaining wall type;	**Cantilever propped at both**
Height of retaining wall stem;	h_{stem} = **3250** mm
Thickness of wall stem;	t_{wall} = **330** mm
Length of toe;	l_{toe} = **4400** mm
Length of heel;	l_{heel} = **0** mm
Overall length of base;	$l_{base} = l_{toe} + l_{heel} + t_{wall}$ = **4730** mm
Thickness of base;	t_{base} = **400** mm
Depth of downstand;	d_{ds} = **0** mm
Position of downstand;	l_{ds} = **1900** mm
Thickness of downstand;	t_{ds} = **400** mm
Height of retaining wall;	$h_{wall} = h_{stem} + t_{base} + d_{ds}$ = **3650** mm
Depth of cover in front of wall;	d_{cover} = **0** mm
Depth of unplanned excavation;	d_{exc} = **0** mm
Height of ground water behind wall;	h_{water} = **2650** mm
Height of saturated fill above base;	$h_{sat} = \max(h_{water} - t_{base} - d_{ds}, 0 \text{ mm})$ = **2250** mm
Density of wall construction;	γ_{wall} = **25.0** kN/m^3
Density of base construction;	γ_{base} = **25.0** kN/m^3
Angle of rear face of wall;	α = **90.0** deg
Angle of soil surface behind wall;	β = **0.0** deg
Effective height at virtual back of wall;	$h_{eff} = h_{wall} + l_{heel} \times \tan(\beta)$ = **3650** mm

Retained material details

Mobilisation factor;	M = **2.0**
Moist density of retained material;	γ_m = **19.0** kN/m^3
Saturated density of retained material;	γ_s = **19.0** kN/m^3
Design shear strength;	ϕ' = **25.0** deg
Angle of wall friction;	δ = **25.0** deg

Base material details

Moist density; γ_{mb} = **19.0** kN/m^3

Design shear strength; ϕ'_b = **19.0** deg

Design base friction; δ_b = **25.0** deg

Allowable bearing pressure; $P_{bearing}$ = **150** kN/m^2

Using Coulomb theory

Active pressure coefficient for retained material

$$K_a = \sin(\alpha + \phi')^2 / (\sin(\alpha)^2 \times \sin(\alpha - \delta) \times [1 + \sqrt{(\sin(\phi' + \delta) \times \sin(\phi' - \beta) / (\sin(\alpha - \delta) \times \sin(\alpha + \beta)))}]^2) = \mathbf{0.355}$$

Passive pressure coefficient for base material

$$K_p = \sin(90 - \phi'_b)^2 / (\sin(90 - \delta_b) \times [1 - \sqrt{(\sin(\phi'_b + \delta_b) \times \sin(\phi'_b) / (\sin(90 + \delta_b)))}]^2) = \mathbf{3.938}$$

At-rest pressure

At-rest pressure for retained material; $K_0 = 1 - \sin(\phi')$ = **0.577**

Loading details

Surcharge load on plan; Surcharge = **15.0** kN/m^2

Applied vertical dead load on wall; W_{dead} = **10.0** kN/m

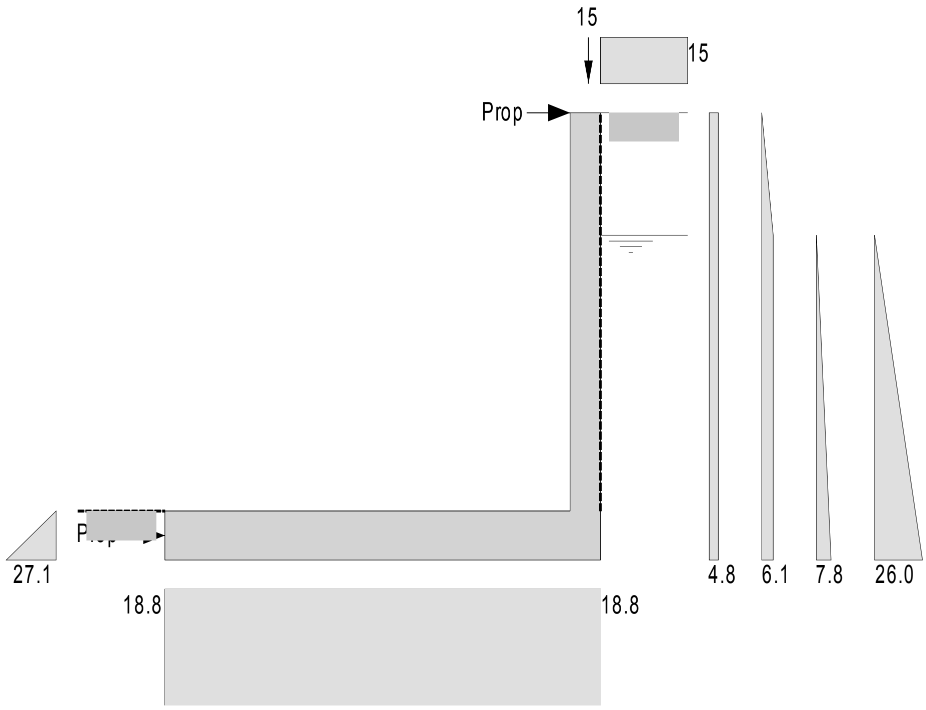
Applied vertical live load on wall; W_{live} = **5.0** kN/m

Position of applied vertical load on wall; l_{load} = **4600** mm

Applied horizontal dead load on wall; F_{dead} = **0.0** kN/m

Applied horizontal live load on wall; F_{live} = **0.0** kN/m

Height of applied horizontal load on wall; h_{load} = **0** mm



Loads shown in kN/m, pressures shown in kN/m^2

Vertical forces on wall

Wall stem; $w_{wall} = h_{stem} \times t_{wall} \times \gamma_{wall}$ = **26.8** kN/m

Wall base; $w_{base} = l_{base} \times t_{base} \times \gamma_{base}$ = **47.3** kN/m

Applied vertical load; $W_v = W_{dead} + W_{live}$ = **15** kN/m

Total vertical load; $W_{total} = w_{wall} + w_{base} + W_v$ = **89.1** kN/m

Horizontal forces on wall

Surcharge; $F_{sur} = K_a \times \cos(90 - \alpha + \delta) \times \text{Surcharge} \times h_{eff}$ = **17.6** kN/m

Moist backfill above water table; $F_{m\_a} = 0.5 \times K_a \times \cos(90 - \alpha + \delta) \times \gamma_m \times (h_{eff} - h_{water})^2$ = **3.1** kN/m

Moist backfill below water table; $F_{m\_b} = K_a \times \cos(90 - \alpha + \delta) \times \gamma_m \times (h_{eff} - h_{water}) \times h_{water}$ = **16.2** kN/m

Saturated backfill; $F_s = 0.5 \times K_a \times \cos(90 - \alpha + \delta) \times (\gamma_s - \gamma_{water}) \times h_{water}^2$ = **10.4** kN/m

Water; $F_{water} = 0.5 \times h_{water}^2 \times \gamma_{water}$ = **34.4** kN/m

Total horizontal load; $F_{total} = F_{sur} + F_{m\_a} + F_{m\_b} + F_s + F_{water}$ = **81.7** kN/m

Calculate total propping force

Passive resistance of soil in front of wall; $F_p = 0.5 \times K_p \times \cos(\delta_b) \times (d_{cover} + t_{base} + d_{ds} - d_{exc})^2 \times \gamma_{mb}$ = **5.4** kN/m

Propping force; $F_{prop} = \max(F_{total} - F_p - (W_{total} - W_{live}) \times \tan(\delta_b), 0 \text{ kN/m})$

F_{prop} = **37.1** kN/m

Overturning moments

Surcharge; $M_{sur} = F_{sur} \times (h_{eff} - 2 \times d_{ds}) / 2$ = **32.2** kNm/m

Moist backfill above water table; $M_{m\_a} = F_{m\_a} \times (h_{eff} + 2 \times h_{water} - 3 \times d_{ds}) / 3$ = **9.1** kNm/m

Moist backfill below water table; $M_{m\_b} = F_{m\_b} \times (h_{water} - 2 \times d_{ds}) / 2$ = **21.5** kNm/m

Saturated backfill; $M_s = F_s \times (h_{water} - 3 \times d_{ds}) / 3$ = **9.2** kNm/m

Water; $M_{water} = F_{water} \times (h_{water} - 3 \times d_{ds}) / 3$ = **30.4** kNm/m

Total overturning moment; $M_{ot} = M_{sur} + M_{m\_a} + M_{m\_b} + M_s + M_{water}$ = **102.3** kNm/m

Restoring moments

Wall stem; $M_{wall} = w_{wall} \times (l_{toe} + t_{wall} / 2)$ = **122.4** kNm/m

Wall base; $M_{base} = w_{base} \times l_{base} / 2$ = **111.9** kNm/m

Design vertical dead load; $M_{dead} = W_{dead} \times l_{load}$ = **46** kNm/m

Total restoring moment; $M_{rest} = M_{wall} + M_{base} + M_{dead}$ = **280.3** kNm/m

Check bearing pressure

Total vertical reaction; $R = W_{total}$ = **89.1** kN/m

Distance to reaction; $x_{bar} = l_{base} / 2$ = **2365** mm

Eccentricity of reaction; $e = \mathrm{abs}((l_{base} / 2) - x_{bar})$ = **0** mm

Reaction acts within middle third of base

Bearing pressure at toe; $p_{toe} = (R / l_{base}) - (6 \times R \times e / l_{base}^2)$ = **18.8** kN/m^2

Bearing pressure at heel; $p_{heel} = (R / l_{base}) + (6 \times R \times e / l_{base}^2)$ = **18.8** kN/m^2

PASS - Maximum bearing pressure is less than allowable bearing pressure

Calculate propping forces to top and base of wall

Propping force to top of wall

$F_{prop\_top} = (M_{ot} - M_{rest} + R \times l_{base} / 2 - F_{prop} \times t_{base} / 2) / (h_{stem} + t_{base} / 2)$ = **7.367** kN/m

Propping force to base of wall; $F_{prop\_base} = F_{prop} - F_{prop\_top}$ = **29.692** kN/m

RETAINING WALL DESIGN

TEDDS calculation version 1.2.01.06

Ultimate limit state load factors

Dead load factor; γ_{f_d} = **1.4**

Live load factor; γ_{f_l} = **1.6**

Earth and water pressure factor; γ_{f_e} = **1.4**

Factored vertical forces on wall

Wall stem; $w_{wall\_f} = \gamma_{f\_d} \times h_{stem} \times t_{wall} \times \gamma_{wall}$ = **37.5** kN/m

Wall base; $w_{base\_f} = \gamma_{f\_d} \times l_{base} \times t_{base} \times \gamma_{base}$ = **66.2** kN/m

Applied vertical load; $W_{v\_f} = \gamma_{f\_d} \times W_{dead} + \gamma_{f\_l} \times W_{live}$ = **22** kN/m

Total vertical load; $W_{total\_f} = w_{wall\_f} + w_{base\_f} + W_{v\_f}$ = **125.8** kN/m

Factored horizontal at-rest forces on wall

Surcharge; $F_{sur\_f} = \gamma_{f\_l} \times K_0 \times \text{Surcharge} \times h_{eff}$ = **50.6** kN/m

Moist backfill above water table; $F_{m\_a\_f} = \gamma_{f\_e} \times 0.5 \times K_0 \times \gamma_m \times (h_{eff} - h_{water})^2$ = **7.7** kN/m

Moist backfill below water table; $F_{m\_b\_f} = \gamma_{f\_e} \times K_0 \times \gamma_m \times (h_{eff} - h_{water}) \times h_{water}$ = **40.7** kN/m

Saturated backfill; $F_{s\_f} = \gamma_{f\_e} \times 0.5 \times K_0 \times (\gamma_s - \gamma_{water}) \times h_{water}^2$ = **26.1** kN/m

Water; $F_{water\_f} = \gamma_{f\_e} \times 0.5 \times h_{water}^2 \times \gamma_{water}$ = **48.2** kN/m

Total horizontal load; $F_{total\_f} = F_{sur\_f} + F_{m\_a\_f} + F_{m\_b\_f} + F_{s\_f} + F_{water\_f}$ = **173.3** kN/m

Calculate total propping force

Passive resistance of soil in front of wall; $F_{p\_f} = \gamma_{f\_e} \times 0.5 \times K_p \times \cos(\delta_b) \times (d_{cover} + t_{base} + d_{ds} - d_{exc})^2 \times \gamma_{mb}$
= **7.6** kN/m

Propping force; $F_{prop\_f} = \max(F_{total\_f} - F_{p\_f} - (W_{total\_f} - \gamma_{f\_l} \times W_{live}) \times \tan(\delta_b)$, 0
kN/m)

F_{prop_f} = **110.8** kN/m

Factored overturning moments

Surcharge; $M_{sur\_f} = F_{sur\_f} \times (h_{eff} - 2 \times d_{ds}) / 2$ = **92.3** kNm/m

Moist backfill above water table; $M_{m\_a\_f} = F_{m\_a\_f} \times (h_{eff} + 2 \times h_{water} - 3 \times d_{ds}) / 3$ = **22.9** kNm/m

Moist backfill below water table; $M_{m\_b\_f} = F_{m\_b\_f} \times (h_{water} - 2 \times d_{ds}) / 2$ = **53.9** kNm/m

Saturated backfill; $M_{s\_f} = F_{s\_f} \times (h_{water} - 3 \times d_{ds}) / 3$ = **23** kNm/m

Water; $M_{water\_f} = F_{water\_f} \times (h_{water} - 3 \times d_{ds}) / 3$ = **42.6** kNm/m

Total overturning moment; $M_{ot\_f} = M_{sur\_f} + M_{m\_a\_f} + M_{m\_b\_f} + M_{s\_f} + M_{water\_f}$ = **234.8** kNm/m

Restoring moments

Wall stem; $M_{wall\_f} = w_{wall\_f} \times (l_{toe} + t_{wall} / 2)$ = **171.4** kNm/m

Wall base; $M_{base\_f} = w_{base\_f} \times l_{base} / 2$ = **156.6** kNm/m

Design vertical load; $M_{v\_f} = W_{v\_f} \times l_{load}$ = **101.2** kNm/m

Total restoring moment; $M_{rest\_f} = M_{wall\_f} + M_{base\_f} + M_{v\_f}$ = **429.2** kNm/m

Factored bearing pressure

Total vertical reaction; $R_f = W_{total\_f}$ = **125.8** kN/m

Distance to reaction; $x_{bar\_f} = l_{base} / 2$ = **2365** mm

Eccentricity of reaction; $e_f = \text{abs}((l_{base} / 2) - x_{bar\_f})$ = **0** mm

Reaction acts within middle third of base

Bearing pressure at toe; $p_{toe\_f} = (R_f / l_{base}) - (6 \times R_f \times e_f / l_{base}^2)$ = **26.6** kN/m^2

Bearing pressure at heel; $p_{heel\_f} = (R_f / l_{base}) + (6 \times R_f \times e_f / l_{base}^2)$ = **26.6** kN/m^2

Rate of change of base reaction; rate = $(p_{toe\_f} - p_{heel\_f}) / l_{base}$ = **0.00** kN/m^2/m

Bearing pressure at stem / toe; $p_{stem\_toe\_f} = \max(p_{toe\_f} - (\text{rate} \times l_{toe})$, 0 kN/m^2) = **26.6** kN/m^2

Bearing pressure at mid stem; $p_{stem\_mid\_f} = \max(p_{toe\_f} - (\text{rate} \times (l_{toe} + t_{wall} / 2))$, 0 kN/m^2) = **26.6**
kN/m^2

Bearing pressure at stem / heel; $p_{stem\_heel\_f} = \max(p_{toe\_f} - (\text{rate} \times (l_{toe} + t_{wall})), 0\ \text{kN/m}^2) =$ **26.6** kN/m^2

Calculate propping forces to top and base of wall

Propping force to top of wall

$F_{prop\_top\_f} = (M_{ot\_f} - M_{rest\_f} + R_f \times l_{base} / 2 - F_{prop\_f} \times t_{base} / 2) / (h_{stem} + t_{base} / 2) =$ **23.442** kN/m

Propping force to base of wall; $F_{prop\_base\_f} = F_{prop\_f} - F_{prop\_top\_f} =$ **87.315** kN/m

<u>Design of reinforced concrete retaining wall toe</u>

Material properties

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

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

Base details

Minimum area of reinforcement; k = **0.13** %

Cover to reinforcement in toe; $c_{toe} =$ **50** mm

Calculate shear for toe design

Shear from bearing pressure; $V_{toe\_bear} = (p_{toe\_f} + p_{stem\_toe\_f}) \times l_{toe} / 2 =$ **117** kN/m

Shear from weight of base; $V_{toe\_wt\_base} = \gamma_{f\_d} \times \gamma_{base} \times l_{toe} \times t_{base} =$ **61.6** kN/m

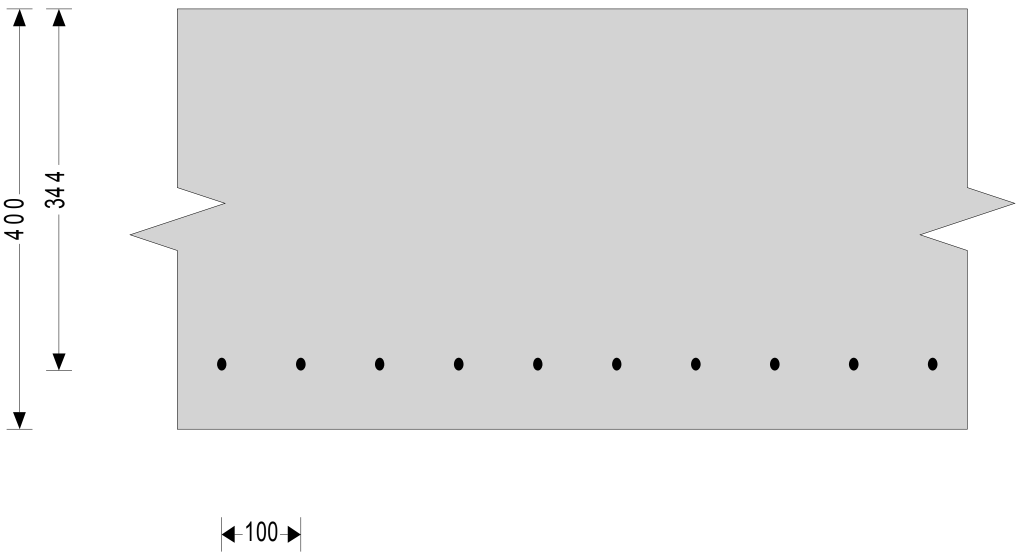
Total shear for toe design; $V_{toe} = V_{toe\_bear} - V_{toe\_wt\_base} =$ **55.4** kN/m

Calculate moment for toe design

Moment from bearing pressure; $M_{toe\_bear} = (2 \times p_{toe\_f} + p_{stem\_mid\_f}) \times (l_{toe} + t_{wall} / 2)^2 / 6 =$ **277** kNm/m

Moment from weight of base; $M_{toe\_wt\_base} = (\gamma_{f\_d} \times \gamma_{base} \times t_{base} \times (l_{toe} + t_{wall} / 2)^2 / 2) =$ **145.9** kNm/m

Total moment for toe design; $M_{toe} = M_{toe\_bear} - M_{toe\_wt\_base} =$ **131.2** kNm/m



Check toe in bending

Width of toe; b = **1000** mm/m

Depth of reinforcement; $d_{toe} = t_{base} - c_{toe} - (\phi_{toe} / 2) =$ **344.0** mm

Constant; $K_{toe} = M_{toe} / (b \times d_{toe}^2 \times f_{cu}) =$ **0.028**

Compression reinforcement is not required

Lever arm; $z_{toe} = \min(0.5 + \sqrt{(0.25 - (\min(K_{toe}, 0.225) / 0.9))}, 0.95) \times d_{toe}$

$z_{toe} =$ **327** mm

Area of tension reinforcement required; $A_{s\_toe\_des} = M_{toe} / (0.87 \times f_y \times z_{toe}) =$ **923** mm^2/m

Minimum area of tension reinforcement; $A_{s\_toe\_min} = k \times b \times t_{base} =$ **520** mm^2/m

Area of tension reinforcement required; $A_{s\_toe\_req}$ = Max($A_{s\_toe\_des}$, $A_{s\_toe\_min}$) = **923** mm²/m

Reinforcement provided; **12 mm dia.bars @ 100 mm centres**

Area of reinforcement provided; $A_{s\_toe\_prov}$ = **1131** mm²/m

PASS - Reinforcement provided at the retaining wall toe is adequate

Check shear resistance at toe

Design shear stress; $v_{toe} = V_{toe} / (b \times d_{toe})$ = **0.161** N/mm²

Allowable shear stress; $v_{adm} = \min(0.8 \times \sqrt{(f_{cu} / 1\ N/mm^2)}, 5) \times 1\ N/mm^2$ = **5.000** N/mm²

PASS - Design shear stress is less than maximum shear stress

From BS8110:Part 1:1997 – Table 3.8

Design concrete shear stress; v_{c_toe} = **0.530** N/mm²

$v_{toe} < v_{c\_toe}$ - No shear reinforcement required

<u>Design of reinforced concrete retaining wall stem</u>

Material properties

Characteristic strength of concrete; f_{cu} = **40** N/mm²

Characteristic strength of reinforcement; f_y = **500** N/mm²

Wall details

Minimum area of reinforcement; k = **0.13** %

Cover to reinforcement in stem; c_{stem} = **40** mm

Cover to reinforcement in wall; c_{wall} = **40** mm

Factored horizontal at-rest forces on stem

Surcharge; $F_{s\_sur\_f} = \gamma_{f\_l} \times K_0 \times Surcharge \times (h_{eff} - t_{base} - d_{ds})$ = **45** kN/m

Moist backfill above water table; $F_{s\_m\_a\_f} = 0.5 \times \gamma_{f\_e} \times K_0 \times \gamma_m \times (h_{eff} - t_{base} - d_{ds} - h_{sat})^2$ = **7.7** kN/m

Moist backfill below water table; $F_{s\_m\_b\_f} = \gamma_{f\_e} \times K_0 \times \gamma_m \times (h_{eff} - t_{base} - d_{ds} - h_{sat}) \times h_{sat}$ = **34.6** kN/m

Saturated backfill; $F_{s\_s\_f} = 0.5 \times \gamma_{f\_e} \times K_0 \times (\gamma_s - \gamma_{water}) \times h_{sat}^2$ = **18.8** kN/m

Water; $F_{s\_water\_f} = 0.5 \times \gamma_{f\_e} \times \gamma_{water} \times h_{sat}^2$ = **34.8** kN/m

Calculate shear for stem design

Surcharge; $V_{s\_sur\_f} = 5 \times F_{s\_sur\_f} / 8$ = **28.1** kN/m

Moist backfill above water table; $V_{s\_m\_a\_f} = F_{s\_m\_a\_f} \times b_l \times ((5 \times L^2) - b_l^2) / (5 \times L^3)$ = **2.2** kN/m

Moist backfill below water table; $V_{s\_m\_b\_f} = F_{s\_m\_b\_f} \times (8 - (n^2 \times (4 - n))) / 8$ = **27.4** kN/m

Saturated backfill; $V_{s\_s\_f} = F_{s\_s\_f} \times (1 - (a_l^2 \times ((5 \times L) - a_l) / (20 \times L^3)))$ = **16.8** kN/m

Water; $V_{s\_water\_f} = F_{s\_water\_f} \times (1 - (a_l^2 \times ((5 \times L) - a_l) / (20 \times L^3)))$ = **31** kN/m

Total shear for stem design; $V_{stem} = V_{s\_sur\_f} + V_{s\_m\_a\_f} + V_{s\_m\_b\_f} + V_{s\_s\_f} + V_{s\_water\_f}$ = **105.5** kN/m

Calculate moment for stem design

Surcharge; $M_{s\_sur} = F_{s\_sur\_f} \times L / 8$ = **19.4** kNm/m

Moist backfill above water table; $M_{s\_m\_a} = F_{s\_m\_a\_f} \times b_l \times ((5 \times L^2) - (3 \times b_l^2)) / (15 \times L^2)$ = **2.4** kNm/m

Moist backfill below water table; $M_{s\_m\_b} = F_{s\_m\_b\_f} \times a_l \times (2 - n)^2 / 8$ = **17.6** kNm/m

Saturated backfill; $M_{s\_s} = F_{s\_s\_f} \times a_l \times ((3 \times a_l^2) - (15 \times a_l \times L) + (20 \times L^2)) / (60 \times L^2)$ = **8.3** kNm/m

Water; $M_{s\_water} = F_{s\_water\_f} \times a_l \times ((3 \times a_l^2) - (15 \times a_l \times L) + (20 \times L^2)) / (60 \times L^2)$ = **15.4** kNm/m

Total moment for stem design; $M_{stem} = M_{s\_sur} + M_{s\_m\_a} + M_{s\_m\_b} + M_{s\_s} + M_{s\_water}$ = **63.2** kNm/m

Calculate moment for wall design

Surcharge; $M_{w\_sur} = 9 \times F_{s\_sur\_f} \times L / 128 =$ **10.9** kNm/m

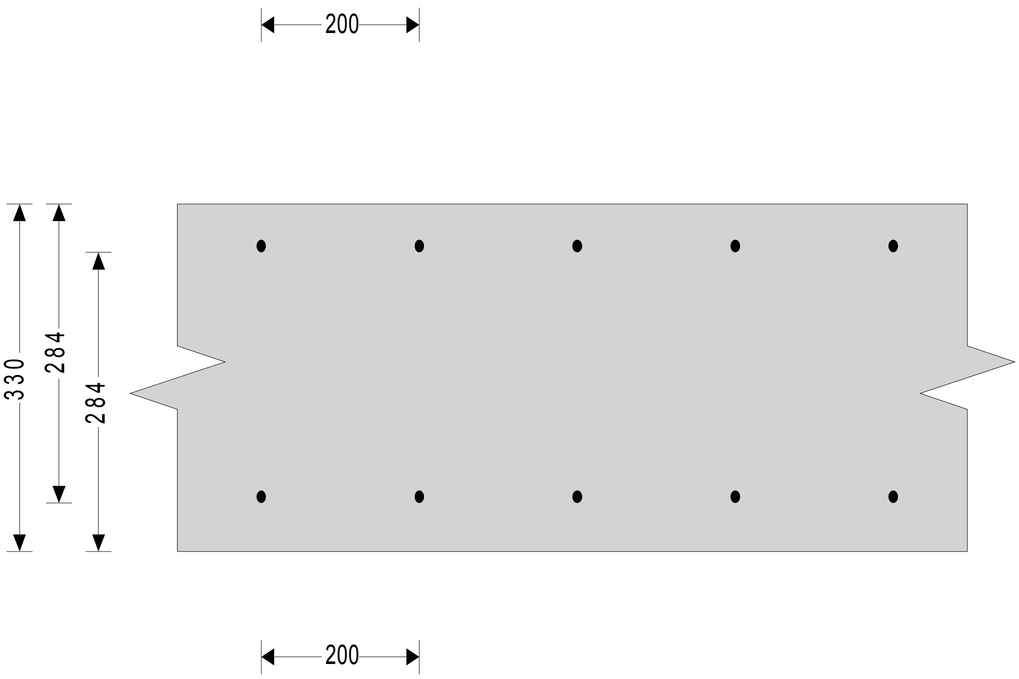
Moist backfill above water table; $M_{w\_m\_a} = F_{s\_m\_a\_f} \times 0.577 \times b_l \times [(b_l^3+5 \times a_l \times L^2)/(5 \times L^3)-0.577^2/3] =$ **2.7** kNm/m

Moist backfill below water table; $M_{w\_m\_b} = F_{s\_m\_b\_f} \times a_l \times [((8-n^2 \times (4-n))^2 /16)-4+n \times (4-n)]/8 =$ **9** kNm/m

Saturated backfill; $M_{w\_s} = F_{s\_s\_f} \times [a_l^2 \times x \times ((5 \times L)-a_l)/(20 \times L^3)-(x-b_l)^3 /(3 \times a_l^2)] =$ **3.1** kNm/m

Water; $M_{w\_water} = F_{s\_water\_f} \times [a_l^2 \times x \times ((5 \times L)-a_l)/(20 \times L^3)-(x-b_l)^3 /(3 \times a_l^2)] =$ **5.8** kNm/m

Total moment for wall design; $M_{wall} = M_{w\_sur} + M_{w\_m\_a} + M_{w\_m\_b} + M_{w\_s} + M_{w\_water} =$ **31.5** kNm/m



Check wall stem in bending

Width of wall stem; b = **1000** mm/m

Depth of reinforcement; $d_{stem} = t_{wall} - c_{stem} - (\phi_{stem} / 2) =$ **284.0** mm

Constant; $K_{stem} = M_{stem} / (b \times d_{stem}^2 \times f_{cu}) =$ **0.020**

Compression reinforcement is not required

Lever arm; $z_{stem} = \min(0.5 + \sqrt{(0.25 - (\min(K_{stem}, 0.225) / 0.9))},0.95) \times d_{stem}$

$z_{stem} =$ **270** mm

Area of tension reinforcement required; $A_{s\_stem\_des} = M_{stem} / (0.87 \times f_y \times z_{stem}) =$ **539** mm²/m

Minimum area of tension reinforcement; $A_{s\_stem\_min} = k \times b \times t_{wall} =$ **429** mm²/m

Area of tension reinforcement required; $A_{s\_stem\_req} = \mathrm{Max}(A_{s\_stem\_des}, A_{s\_stem\_min}) =$ **539** mm²/m

Reinforcement provided; **12 mm dia.bars @ 200 mm centres**

Area of reinforcement provided; $A_{s\_stem\_prov} =$ **565** mm²/m

PASS - Reinforcement provided at the retaining wall stem is adequate

Check shear resistance at wall stem

Design shear stress; $v_{stem} = V_{stem} / (b \times d_{stem}) =$ **0.371** N/mm²

Allowable shear stress; $v_{adm} = \min(0.8 \times \sqrt{(f_{cu} / 1 \text{ N/mm}^2)}, 5) \times 1 \text{ N/mm}^2 =$ **5.000** N/mm²

PASS - Design shear stress is less than maximum shear stress

From BS8110:Part 1:1997 – Table 3.8

Design concrete shear stress; v_{c_stem} = **0.470** N/mm^2

v_{stem} < v_{c_stem} - No shear reinforcement required

Check mid height of wall in bending

Depth of reinforcement; $d_{wall} = t_{wall} – c_{wall} – (\phi_{wall} / 2)$ = **284.0** mm

Constant; $K_{wall} = M_{wall} / (b \times d_{wall}^2 \times f_{cu})$ = **0.010**

Compression reinforcement is not required

Lever arm; $z_{wall} = Min(0.5 + \sqrt{(0.25 - (min(K_{wall}, 0.225) / 0.9))},0.95) \times d_{wall}$

z_{wall} = **270** mm

Area of tension reinforcement required; $A_{s\_wall\_des} = M_{wall} / (0.87 \times f_y \times z_{wall})$ = **268** mm^2/m

Minimum area of tension reinforcement; $A_{s\_wall\_min} = k \times b \times t_{wall}$ = **429** mm^2/m

Area of tension reinforcement required; $A_{s\_wall\_req} = Max(A_{s\_wall\_des}, A_{s\_wall\_min})$ = **429** mm^2/m

Reinforcement provided; **12 mm dia.bars @ 200 mm centres**

Area of reinforcement provided; $A_{s\_wall\_prov}$ = **565** mm^2/m

PASS - Reinforcement provided to the retaining wall at mid height is adequate

Check retaining wall deflection

Basic span/effective depth ratio; $ratio_{bas}$ = **20**

Design service stress; $f_s = 2 \times f_y \times A_{s\_stem\_req} / (3 \times A_{s\_stem\_prov})$ = **317.5** N/mm^2

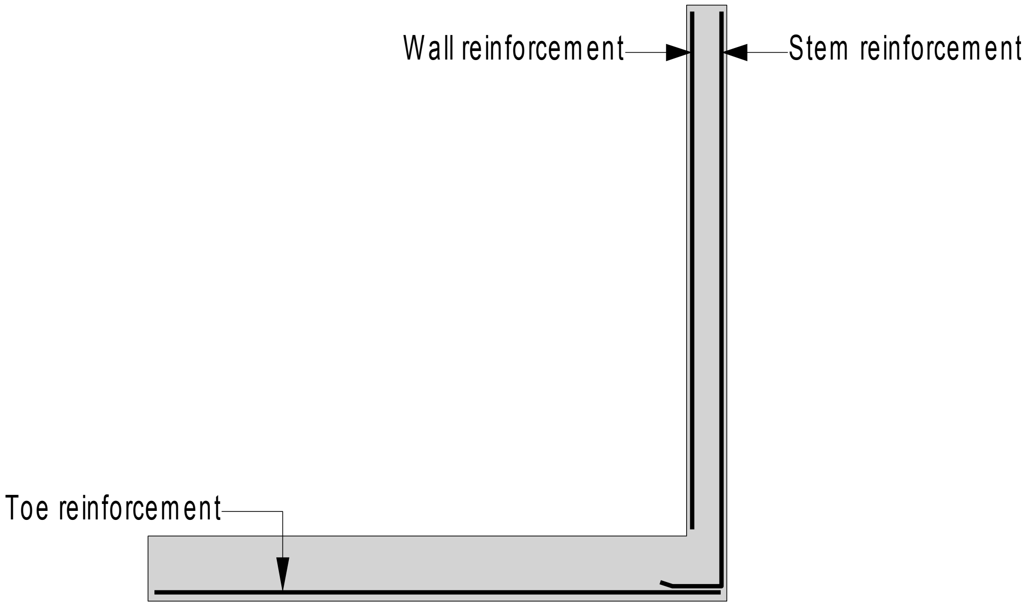
Modification factor; $factor_{tens}$ = min(0.55 + (477 N/mm^2 - f_s)/(120 × (0.9 N/mm^2 + (M_{stem}/(b × d_{stem}^2)))),2) = **1.34**

Maximum span/effective depth ratio; $ratio_{max} = ratio_{bas} \times factor_{tens}$ = **26.79**

Actual span/effective depth ratio; $ratio_{act} = h_{stem} / d_{stem}$ = **11.44**

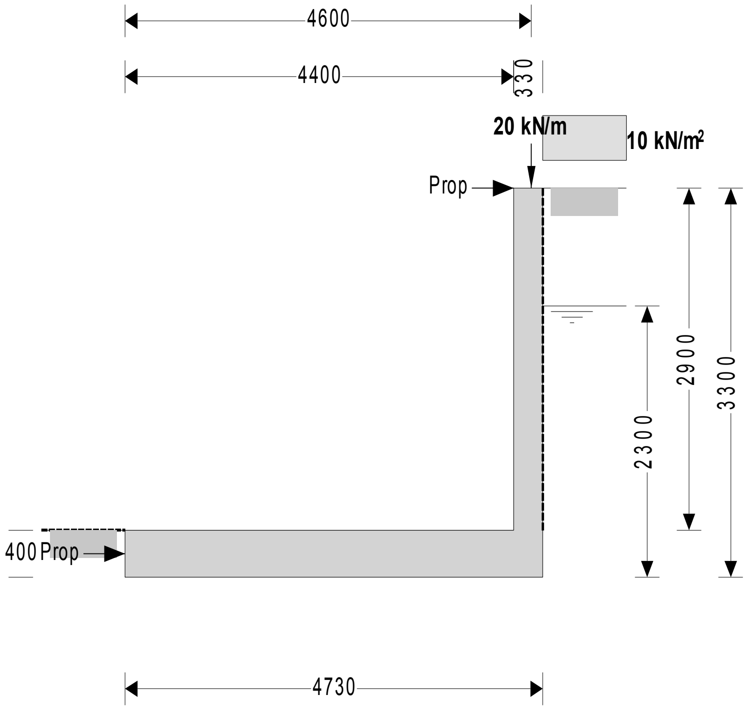
PASS - Span to depth ratio is acceptable

Indicative retaining wall reinforcement diagram



Toe bars - 12 mm dia.@ 100 mm centres - (1131 mm^2/m)
Wall bars - 12 mm dia.@ 200 mm centres - (565 mm^2/m)
Stem bars - 12 mm dia.@ 200 mm centres - (565 mm^2/m)

RETAINING WALL ANALYSIS – BACK 330MM



Wall details

Retaining wall type;	**Cantilever propped at both**
Height of retaining wall stem;	h_{stem} = **2900** mm
Thickness of wall stem;	t_{wall} = **330** mm
Length of toe;	l_{toe} = **4400** mm
Length of heel;	l_{heel} = **0** mm
Overall length of base;	$l_{base} = l_{toe} + l_{heel} + t_{wall}$ = **4730** mm
Thickness of base;	t_{base} = **400** mm
Depth of downstand;	d_{ds} = **0** mm
Position of downstand;	l_{ds} = **1900** mm
Thickness of downstand;	t_{ds} = **400** mm
Height of retaining wall;	$h_{wall} = h_{stem} + t_{base} + d_{ds}$ = **3300** mm
Depth of cover in front of wall;	d_{cover} = **0** mm
Depth of unplanned excavation;	d_{exc} = **0** mm
Height of ground water behind wall;	h_{water} = **2300** mm
Height of saturated fill above base;	$h_{sat} = \max(h_{water} - t_{base} - d_{ds}, 0\text{ mm})$ = **1900** mm
Density of wall construction;	γ_{wall} = **25.0** kN/m^3
Density of base construction;	γ_{base} = **25.0** kN/m^3
Angle of rear face of wall;	α = **90.0** deg
Angle of soil surface behind wall;	β = **0.0** deg
Effective height at virtual back of wall;	$h_{eff} = h_{wall} + l_{heel} \times \tan(\beta)$ = **3300** mm

Retained material details

Mobilisation factor;	M = **2.0**
Moist density of retained material;	γ_m = **19.0** kN/m^3
Saturated density of retained material;	γ_s = **19.0** kN/m^3
Design shear strength;	ϕ' = **25.0** deg
Angle of wall friction;	δ = **25.0** deg

Base material details

Moist density; γ_{mb} = **19.0** kN/m^3

Design shear strength; ϕ'_b = **19.0** deg

Design base friction; δ_b = **25.0** deg

Allowable bearing pressure; $P_{bearing}$ = **150** kN/m^2

Using Coulomb theory

Active pressure coefficient for retained material

$$K_a = \sin(\alpha + \phi')^2 / (\sin(\alpha)^2 \times \sin(\alpha - \delta) \times [1 + \sqrt{}(\sin(\phi' + \delta) \times \sin(\phi' - \beta) / (\sin(\alpha - \delta) \times \sin(\alpha + \beta)))]^2) = \mathbf{0.355}$$

Passive pressure coefficient for base material

$$K_p = \sin(90 - \phi'_b)^2 / (\sin(90 - \delta_b) \times [1 - \sqrt{}(\sin(\phi'_b + \delta_b) \times \sin(\phi'_b) / (\sin(90 + \delta_b)))]^2) = \mathbf{3.938}$$

At-rest pressure

At-rest pressure for retained material; $K_0 = 1 - \sin(\phi')$ = **0.577**

Loading details

Surcharge load on plan; Surcharge = **10.0** kN/m^2

Applied vertical dead load on wall; W_{dead} = **10.0** kN/m

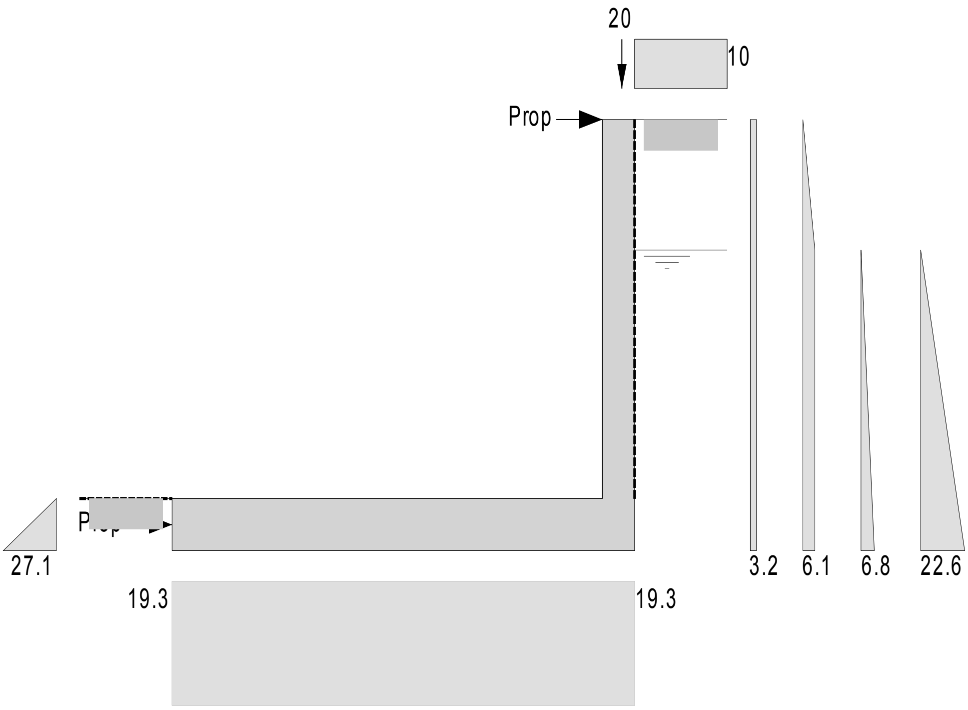
Applied vertical live load on wall; W_{live} = **10.0** kN/m

Position of applied vertical load on wall; l_{load} = **4600** mm

Applied horizontal dead load on wall; F_{dead} = **0.0** kN/m

Applied horizontal live load on wall; F_{live} = **0.0** kN/m

Height of applied horizontal load on wall; h_{load} = **0** mm



Loads shown in kN/m, pressures shown in kN/m^2

Vertical forces on wall

Wall stem; $w_{wall} = h_{stem} \times t_{wall} \times \gamma_{wall}$ = **23.9** kN/m

Wall base; $w_{base} = l_{base} \times t_{base} \times \gamma_{base}$ = **47.3** kN/m

Applied vertical load; $W_v = W_{dead} + W_{live}$ = **20** kN/m

Total vertical load; $W_{total} = w_{wall} + w_{base} + W_v$ = **91.2** kN/m

Horizontal forces on wall

Surcharge; $F_{sur} = K_a \times \cos(90 - \alpha + \delta) \times \text{Surcharge} \times h_{eff}$ = **10.6** kN/m

Moist backfill above water table; $F_{m\_a} = 0.5 \times K_a \times \cos(90 - \alpha + \delta) \times \gamma_m \times (h_{eff} - h_{water})^2$ = **3.1** kN/m

Moist backfill below water table; $F_{m\_b} = K_a \times \cos(90 - \alpha + \delta) \times \gamma_m \times (h_{eff} - h_{water}) \times h_{water}$ = **14.1** kN/m

Saturated backfill; $F_s = 0.5 \times K_a \times \cos(90 - \alpha + \delta) \times (\gamma_s - \gamma_{water}) \times h_{water}^2$ = **7.8** kN/m

Water; $F_{water} = 0.5 \times h_{water}^2 \times \gamma_{water}$ = **25.9** kN/m

Total horizontal load; $F_{total} = F_{sur} + F_{m\_a} + F_{m\_b} + F_s + F_{water}$ = **61.5** kN/m

Calculate total propping force

Passive resistance of soil in front of wall; $F_p = 0.5 \times K_p \times \cos(\delta_b) \times (d_{cover} + t_{base} + d_{ds} - d_{exc})^2 \times \gamma_{mb}$ = **5.4** kN/m

Propping force; $F_{prop} = \max(F_{total} - F_p - (W_{total} - W_{live}) \times \tan(\delta_b), 0 \text{ kN/m})$

F_{prop} = **18.2** kN/m

Overturning moments

Surcharge; $M_{sur} = F_{sur} \times (h_{eff} - 2 \times d_{ds}) / 2$ = **17.5** kNm/m

Moist backfill above water table; $M_{m\_a} = F_{m\_a} \times (h_{eff} + 2 \times h_{water} - 3 \times d_{ds}) / 3$ = **8.1** kNm/m

Moist backfill below water table; $M_{m\_b} = F_{m\_b} \times (h_{water} - 2 \times d_{ds}) / 2$ = **16.2** kNm/m

Saturated backfill; $M_s = F_s \times (h_{water} - 3 \times d_{ds}) / 3$ = **6** kNm/m

Water; $M_{water} = F_{water} \times (h_{water} - 3 \times d_{ds}) / 3$ = **19.9** kNm/m

Total overturning moment; $M_{ot} = M_{sur} + M_{m\_a} + M_{m\_b} + M_s + M_{water}$ = **67.6** kNm/m

Restoring moments

Wall stem; $M_{wall} = w_{wall} \times (l_{toe} + t_{wall} / 2)$ = **109.2** kNm/m

Wall base; $M_{base} = w_{base} \times l_{base} / 2$ = **111.9** kNm/m

Design vertical dead load; $M_{dead} = W_{dead} \times l_{load}$ = **46** kNm/m

Total restoring moment; $M_{rest} = M_{wall} + M_{base} + M_{dead}$ = **267.1** kNm/m

Check bearing pressure

Total vertical reaction; $R = W_{total}$ = **91.2** kN/m

Distance to reaction; $x_{bar} = l_{base} / 2$ = **2365** mm

Eccentricity of reaction; $e = \text{abs}((l_{base} / 2) - x_{bar})$ = **0** mm

Reaction acts within middle third of base

Bearing pressure at toe; $p_{toe} = (R / l_{base}) - (6 \times R \times e / l_{base}^2)$ = **19.3** kN/m^2

Bearing pressure at heel; $p_{heel} = (R / l_{base}) + (6 \times R \times e / l_{base}^2)$ = **19.3** kN/m^2

PASS - Maximum bearing pressure is less than allowable bearing pressure

Calculate propping forces to top and base of wall

Propping force to top of wall

$F_{prop\_top} = (M_{ot} - M_{rest} + R \times l_{base} / 2 - F_{prop} \times t_{base} / 2) / (h_{stem} + t_{base} / 2)$ = **4.083** kN/m

Propping force to base of wall; $F_{prop\_base} = F_{prop} - F_{prop\_top}$ = **14.124** kN/m

RETAINING WALL DESIGN

TEDDS calculation version 1.2.01.06

Ultimate limit state load factors

Dead load factor; γ_{f_d} = **1.4**

Live load factor; γ_{f_l} = **1.6**

Earth and water pressure factor; γ_{f_e} = **1.4**

Factored vertical forces on wall

Wall stem; $w_{wall\_f} = \gamma_{f\_d} \times h_{stem} \times t_{wall} \times \gamma_{wall}$ = **33.5** kN/m

Wall base; $w_{base\_f} = \gamma_{f\_d} \times l_{base} \times t_{base} \times \gamma_{base}$ = **66.2** kN/m

Applied vertical load; $W_{v\_f} = \gamma_{f\_d} \times W_{dead} + \gamma_{f\_l} \times W_{live}$ = **30** kN/m

Total vertical load; $W_{total\_f} = w_{wall\_f} + w_{base\_f} + W_{v\_f}$ = **129.7** kN/m

Factored horizontal at-rest forces on wall

Surcharge; $F_{sur\_f} = \gamma_{f\_l} \times K_0 \times Surcharge \times h_{eff}$ = **30.5** kN/m

Moist backfill above water table; $F_{m\_a\_f} = \gamma_{f\_e} \times 0.5 \times K_0 \times \gamma_m \times (h_{eff} - h_{water})^2$ = **7.7** kN/m

Moist backfill below water table; $F_{m\_b\_f} = \gamma_{f\_e} \times K_0 \times \gamma_m \times (h_{eff} - h_{water}) \times h_{water}$ = **35.3** kN/m

Saturated backfill; $F_{s\_f} = \gamma_{f\_e} \times 0.5 \times K_0 \times (\gamma_s - \gamma_{water}) \times h_{water}^2$ = **19.6** kN/m

Water; $F_{water\_f} = \gamma_{f\_e} \times 0.5 \times h_{water}^2 \times \gamma_{water}$ = **36.3** kN/m

Total horizontal load; $F_{total\_f} = F_{sur\_f} + F_{m\_a\_f} + F_{m\_b\_f} + F_{s\_f} + F_{water\_f}$ = **129.5** kN/m

Calculate total propping force

Passive resistance of soil in front of wall; $F_{p\_f} = \gamma_{f\_e} \times 0.5 \times K_p \times \cos(\delta_b) \times (d_{cover} + t_{base} + d_{ds} - d_{exc})^2 \times \gamma_{mb}$ = **7.6** kN/m

Propping force; $F_{prop\_f} = \max(F_{total\_f} - F_{p\_f} - (W_{total\_f} - \gamma_{f\_l} \times W_{live}) \times \tan(\delta_b), 0$ kN/m)

F_{prop_f} = **68.8** kN/m

Factored overturning moments

Surcharge; $M_{sur\_f} = F_{sur\_f} \times (h_{eff} - 2 \times d_{ds}) / 2$ = **50.3** kNm/m

Moist backfill above water table; $M_{m\_a\_f} = F_{m\_a\_f} \times (h_{eff} + 2 \times h_{water} - 3 \times d_{ds}) / 3$ = **20.2** kNm/m

Moist backfill below water table; $M_{m\_b\_f} = F_{m\_b\_f} \times (h_{water} - 2 \times d_{ds}) / 2$ = **40.6** kNm/m

Saturated backfill; $M_{s\_f} = F_{s\_f} \times (h_{water} - 3 \times d_{ds}) / 3$ = **15.1** kNm/m

Water; $M_{water\_f} = F_{water\_f} \times (h_{water} - 3 \times d_{ds}) / 3$ = **27.9** kNm/m

Total overturning moment; $M_{ot\_f} = M_{sur\_f} + M_{m\_a\_f} + M_{m\_b\_f} + M_{s\_f} + M_{water\_f}$ = **154.1** kNm/m

Restoring moments

Wall stem; $M_{wall\_f} = w_{wall\_f} \times (l_{toe} + t_{wall} / 2)$ = **152.9** kNm/m

Wall base; $M_{base\_f} = w_{base\_f} \times l_{base} / 2$ = **156.6** kNm/m

Design vertical load; $M_{v\_f} = W_{v\_f} \times l_{load}$ = **138** kNm/m

Total restoring moment; $M_{rest\_f} = M_{wall\_f} + M_{base\_f} + M_{v\_f}$ = **447.5** kNm/m

Factored bearing pressure

Total vertical reaction; $R_f = W_{total\_f}$ = **129.7** kN/m

Distance to reaction; $x_{bar\_f} = l_{base} / 2$ = **2365** mm

Eccentricity of reaction; $e_f = \mathrm{abs}((l_{base} / 2) - x_{bar\_f})$ = **0** mm

Reaction acts within middle third of base

Bearing pressure at toe; $p_{toe\_f} = (R_f / l_{base}) - (6 \times R_f \times e_f / l_{base}^2)$ = **27.4** kN/m^2

Bearing pressure at heel; $p_{heel\_f} = (R_f / l_{base}) + (6 \times R_f \times e_f / l_{base}^2)$ = **27.4** kN/m^2

Rate of change of base reaction; $rate = (p_{toe\_f} - p_{heel\_f}) / l_{base}$ = **0.00** kN/m^2/m

Bearing pressure at stem / toe; $p_{stem\_toe\_f} = \max(p_{toe\_f} - (rate \times l_{toe}), 0$ kN/m^2) = **27.4** kN/m^2

Bearing pressure at mid stem; $p_{stem\_mid\_f} = \max(p_{toe\_f} - (rate \times (l_{toe} + t_{wall} / 2)), 0$ kN/m^2) = **27.4** kN/m^2

Bearing pressure at stem / heel; $p_{stem\_heel\_f} = \max(p_{toe\_f} - (\text{rate} \times (l_{toe} + t_{wall})), 0\ \text{kN/m}^2) =$ **27.4** kN/m²

Calculate propping forces to top and base of wall

Propping force to top of wall

$F_{prop\_top\_f} = (M_{ot\_f} - M_{rest\_f} + R_f \times l_{base} / 2 - F_{prop\_f} \times t_{base} / 2) / (h_{stem} + t_{base} / 2) =$ **-0.144** kN/m

Propping force to base of wall; $F_{prop\_base\_f} = F_{prop\_f} - F_{prop\_top\_f} =$ **68.987** kN/m

Design of reinforced concrete retaining wall toe

Material properties

Characteristic strength of concrete; $f_{cu} =$ **40** N/mm²

Characteristic strength of reinforcement; $f_y =$ **500** N/mm²

Base details

Minimum area of reinforcement; k = **0.13** %

Cover to reinforcement in toe; $c_{toe} =$ **50** mm

Calculate shear for toe design

Shear from bearing pressure; $V_{toe\_bear} = (p_{toe\_f} + p_{stem\_toe\_f}) \times l_{toe} / 2 =$ **120.7** kN/m

Shear from weight of base; $V_{toe\_wt\_base} = \gamma_{f\_d} \times \gamma_{base} \times l_{toe} \times t_{base} =$ **61.6** kN/m

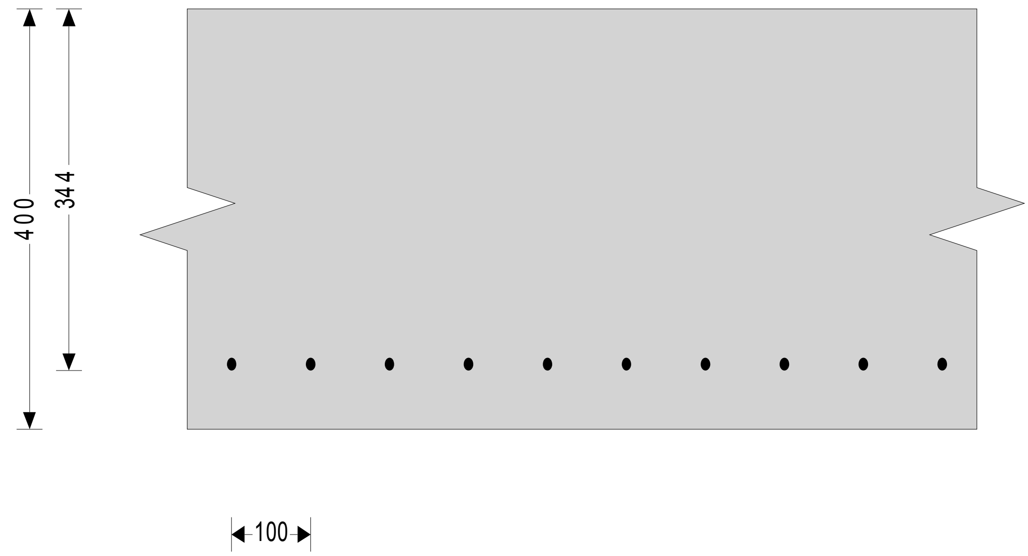
Total shear for toe design; $V_{toe} = V_{toe\_bear} - V_{toe\_wt\_base} =$ **59.1** kN/m

Calculate moment for toe design

Moment from bearing pressure; $M_{toe\_bear} = (2 \times p_{toe\_f} + p_{stem\_mid\_f}) \times (l_{toe} + t_{wall} / 2)^2 / 6 =$ **285.7** kNm/m

Moment from weight of base; $M_{toe\_wt\_base} = (\gamma_{f\_d} \times \gamma_{base} \times t_{base} \times (l_{toe} + t_{wall} / 2)^2 / 2) =$ **145.9** kNm/m

Total moment for toe design; $M_{toe} = M_{toe\_bear} - M_{toe\_wt\_base} =$ **139.9** kNm/m



Check toe in bending

Width of toe; b = **1000** mm/m

Depth of reinforcement; $d_{toe} = t_{base} - c_{toe} - (\phi_{toe} / 2) =$ **344.0** mm

Constant; $K_{toe} = M_{toe} / (b \times d_{toe}^2 \times f_{cu}) =$ **0.030**

Compression reinforcement is not required

Lever arm; $z_{toe} = \min(0.5 + \sqrt{(0.25 - (\min(K_{toe}, 0.225) / 0.9))}, 0.95) \times d_{toe}$

$z_{toe} =$ **327** mm

Area of tension reinforcement required; $A_{s\_toe\_des} = M_{toe} / (0.87 \times f_y \times z_{toe}) =$ **984** mm²/m

Minimum area of tension reinforcement; $A_{s\_toe\_min} = k \times b \times t_{base} =$ **520** mm²/m

Area of tension reinforcement required; $A_{s\_toe\_req} = Max(A_{s\_toe\_des}, A_{s\_toe\_min}) = \mathbf{984}$ mm^2/m
Reinforcement provided; **12 mm dia.bars @ 100 mm centres**
Area of reinforcement provided; $A_{s\_toe\_prov} = \mathbf{1131}$ mm^2/m

PASS - Reinforcement provided at the retaining wall toe is adequate

Check shear resistance at toe

Design shear stress; $v_{toe} = V_{toe} / (b \times d_{toe}) = \mathbf{0.172}$ N/mm^2
Allowable shear stress; $v_{adm} = min(0.8 \times \sqrt{}(f_{cu} / 1\ N/mm^2), 5) \times 1\ N/mm^2 = \mathbf{5.000}$
N/mm^2

PASS - Design shear stress is less than maximum shear stress

From BS8110:Part 1:1997 – Table 3.8

Design concrete shear stress; $v_{c\_toe} = \mathbf{0.530}$ N/mm^2

$v_{toe} < v_{c\_toe}$ - No shear reinforcement required

<u>Design of reinforced concrete retaining wall stem</u>

Material properties

Characteristic strength of concrete; $f_{cu} = \mathbf{40}$ N/mm^2
Characteristic strength of reinforcement; $f_y = \mathbf{500}$ N/mm^2

Wall details

Minimum area of reinforcement; k = **0.13** %
Cover to reinforcement in stem; $c_{stem} = \mathbf{40}$ mm
Cover to reinforcement in wall; $c_{wall} = \mathbf{40}$ mm

Factored horizontal at-rest forces on stem

Surcharge; $F_{s\_sur\_f} = \gamma_{f\_l} \times K_0 \times Surcharge \times (h_{eff} - t_{base} - d_{ds}) = \mathbf{26.8}$ kN/m
Moist backfill above water table; $F_{s\_m\_a\_f} = 0.5 \times \gamma_{f\_e} \times K_0 \times \gamma_m \times (h_{eff} - t_{base} - d_{ds} - h_{sat})^2 = \mathbf{7.7}$
kN/m
Moist backfill below water table; $F_{s\_m\_b\_f} = \gamma_{f\_e} \times K_0 \times \gamma_m \times (h_{eff} - t_{base} - d_{ds} - h_{sat}) \times h_{sat} = \mathbf{29.2}$
kN/m
Saturated backfill; $F_{s\_s\_f} = 0.5 \times \gamma_{f\_e} \times K_0 \times (\gamma_s - \gamma_{water}) \times h_{sat}^2 = \mathbf{13.4}$ kN/m
Water; $F_{s\_water\_f} = 0.5 \times \gamma_{f\_e} \times \gamma_{water} \times h_{sat}^2 = \mathbf{24.8}$ kN/m

Calculate shear for stem design

Surcharge; $V_{s\_sur\_f} = 5 \times F_{s\_sur\_f} / 8 = \mathbf{16.7}$ kN/m
Moist backfill above water table; $V_{s\_m\_a\_f} = F_{s\_m\_a\_f} \times b_l \times ((5 \times L^2) - b_l^2) / (5 \times L^3) = \mathbf{2.4}$ kN/m
Moist backfill below water table; $V_{s\_m\_b\_f} = F_{s\_m\_b\_f} \times (8 - (n^2 \times (4 - n))) / 8 = \mathbf{23.6}$ kN/m
Saturated backfill; $V_{s\_s\_f} = F_{s\_s\_f} \times (1 - (a_l^2 \times ((5 \times L) - a_l) / (20 \times L^3))) = \mathbf{12.1}$
kN/m
Water; $V_{s\_water\_f} = F_{s\_water\_f} \times (1 - (a_l^2 \times ((5 \times L) - a_l) / (20 \times L^3))) = \mathbf{22.3}$
kN/m
Total shear for stem design; $V_{stem} = V_{s\_sur\_f} + V_{s\_m\_a\_f} + V_{s\_m\_b\_f} + V_{s\_s\_f} + V_{s\_water\_f} = \mathbf{77.2}$
kN/m

Calculate moment for stem design

Surcharge; $M_{s\_sur} = F_{s\_sur\_f} \times L / 8 = \mathbf{10.4}$ kNm/m
Moist backfill above water table; $M_{s\_m\_a} = F_{s\_m\_a\_f} \times b_l \times ((5 \times L^2) - (3 \times b_l^2)) / (15 \times L^2) = \mathbf{2.4}$
kNm/m
Moist backfill below water table; $M_{s\_m\_b} = F_{s\_m\_b\_f} \times a_l \times (2 - n)^2 / 8 = \mathbf{13.4}$ kNm/m
Saturated backfill; $M_{s\_s} = F_{s\_s\_f} \times a_l \times ((3 \times a_l^2) - (15 \times a_l \times L) + (20 \times L^2)) / (60 \times L^2) = \mathbf{5.3}$
kNm/m
Water; $M_{s\_water} = F_{s\_water\_f} \times a_l \times ((3 \times a_l^2) - (15 \times a_l \times L) + (20 \times L^2)) / (60 \times L^2) =$
9.7 kNm/m
Total moment for stem design; $M_{stem} = M_{s\_sur} + M_{s\_m\_a} + M_{s\_m\_b} + M_{s\_s} + M_{s\_water} = \mathbf{41.2}$ kNm/m

Calculate moment for wall design

Surcharge; $M_{w\_sur} = 9 \times F_{s\_sur\_f} \times L / 128 =$ **5.8** kNm/m

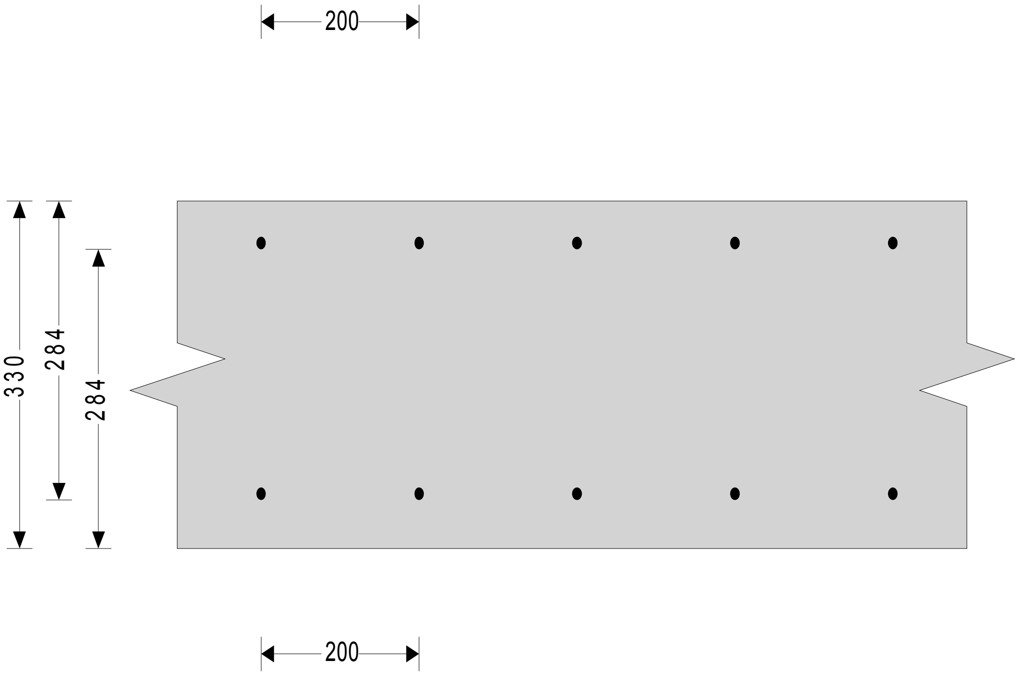
Moist backfill above water table; $M_{w\_m\_a} = F_{s\_m\_a\_f} \times 0.577 \times b_l \times [(b_l^3+5 \times a_l \times L^2)/(5 \times L^3)-0.577^2/3] =$ **2.5** kNm/m

Moist backfill below water table; $M_{w\_m\_b} = F_{s\_m\_b\_f} \times a_l \times [((8-n^2 \times (4-n))^2 /16)-4+n \times (4-n)]/8 =$ **6.7** kNm/m

Saturated backfill; $M_{w\_s} = F_{s\_s\_f} \times [a_l^2 \times x \times ((5 \times L)-a_l)/(20 \times L^3)-(x-b_l)^3 /(3 \times a_l^2)] =$ **1.9** kNm/m

Water; $M_{w\_water} = F_{s\_water\_f} \times [a_l^2 \times x \times ((5 \times L)-a_l)/(20 \times L^3)-(x-b_l)^3 /(3 \times a_l^2)] =$ **3.5** kNm/m

Total moment for wall design; $M_{wall} = M_{w\_sur} + M_{w\_m\_a} + M_{w\_m\_b} + M_{w\_s} + M_{w\_water} =$ **20.5** kNm/m



Check wall stem in bending

Width of wall stem; b = **1000** mm/m

Depth of reinforcement; $d_{stem} = t_{wall} - c_{stem} - (\phi_{stem} / 2) =$ **284.0** mm

Constant; $K_{stem} = M_{stem} / (b \times d_{stem}^2 \times f_{cu}) =$ **0.013**

Compression reinforcement is not required

Lever arm; $z_{stem} = \min(0.5 + \sqrt{(0.25 - (\min(K_{stem}, 0.225) / 0.9))},0.95) \times d_{stem}$

$z_{stem} =$ **270** mm

Area of tension reinforcement required; $A_{s\_stem\_des} = M_{stem} / (0.87 \times f_y \times z_{stem}) =$ **351** mm²/m

Minimum area of tension reinforcement; $A_{s\_stem\_min} = k \times b \times t_{wall} =$ **429** mm²/m

Area of tension reinforcement required; $A_{s\_stem\_req} = \mathrm{Max}(A_{s\_stem\_des}, A_{s\_stem\_min}) =$ **429** mm²/m

Reinforcement provided; **12 mm dia.bars @ 200 mm centres**

Area of reinforcement provided; $A_{s\_stem\_prov} =$ **565** mm²/m

PASS - Reinforcement provided at the retaining wall stem is adequate

Check shear resistance at wall stem

Design shear stress; $v_{stem} = V_{stem} / (b \times d_{stem}) =$ **0.272** N/mm²

Allowable shear stress; $v_{adm} = \min(0.8 \times \sqrt{(f_{cu} / 1\ N/mm^2)}, 5) \times 1\ N/mm^2 =$ **5.000** N/mm²

PASS - Design shear stress is less than maximum shear stress

From BS8110:Part 1:1997 – Table 3.8

Design concrete shear stress; v_{c_stem} = **0.470** N/mm^2

v_{stem} < v_{c_stem} - No shear reinforcement required

Check mid height of wall in bending

Depth of reinforcement; $d_{wall} = t_{wall} – c_{wall} – (\phi_{wall} / 2)$ = **284.0** mm

Constant; $K_{wall} = M_{wall} / (b \times d_{wall}^2 \times f_{cu})$ = **0.006**

Compression reinforcement is not required

Lever arm; $z_{wall} = Min(0.5 + \sqrt{}(0.25 - (min(K_{wall}, 0.225) / 0.9)),0.95) \times d_{wall}$

z_{wall} = **270** mm

Area of tension reinforcement required; $A_{s\_wall\_des} = M_{wall} / (0.87 \times f_y \times z_{wall})$ = **175** mm^2/m

Minimum area of tension reinforcement; $A_{s\_wall\_min} = k \times b \times t_{wall}$ = **429** mm^2/m

Area of tension reinforcement required; $A_{s\_wall\_req} = Max(A_{s\_wall\_des}, A_{s\_wall\_min})$ = **429** mm^2/m

Reinforcement provided; **12 mm dia.bars @ 200 mm centres**

Area of reinforcement provided; $A_{s\_wall\_prov}$ = **565** mm^2/m

PASS - Reinforcement provided to the retaining wall at mid height is adequate

Check retaining wall deflection

Basic span/effective depth ratio; $ratio_{bas}$ = **20**

Design service stress; $f_s = 2 \times f_y \times A_{s\_stem\_req} / (3 \times A_{s\_stem\_prov})$ = **252.9** N/mm^2

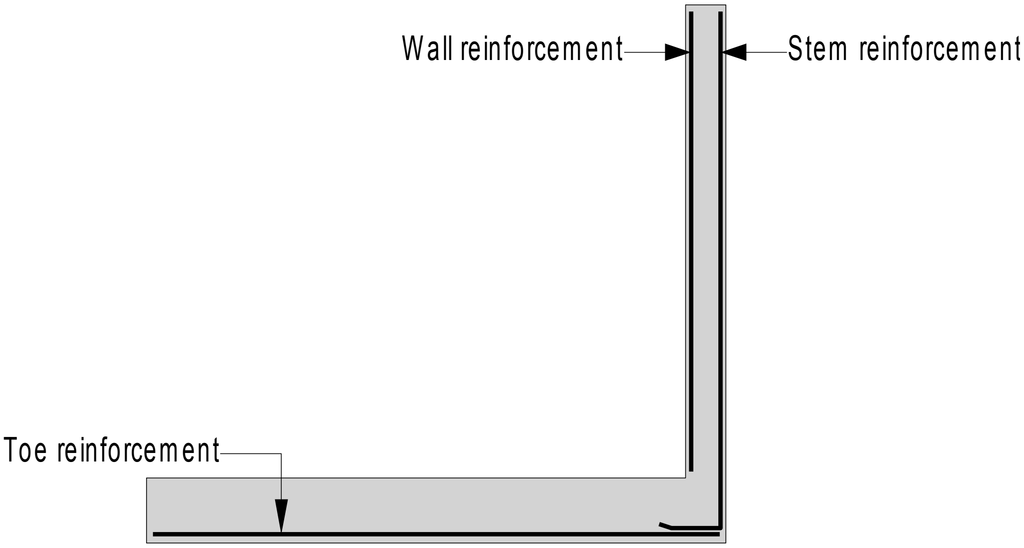
Modification factor; $factor_{tens}$ = min(0.55 + (477 N/mm^2 - f_s)/(120 × (0.9 N/mm^2 + (M_{stem}/(b × d_{stem}^2)))),2) = **1.87**

Maximum span/effective depth ratio; $ratio_{max} = ratio_{bas} \times factor_{tens}$ = **37.48**

Actual span/effective depth ratio; $ratio_{act} = h_{stem} / d_{stem}$ = **10.21**

PASS - Span to depth ratio is acceptable

Indicative retaining wall reinforcement diagram



-

Toe bars - 12 mm dia.@ 100 mm centres - (1131 mm^2/m)
Wall bars - 12 mm dia.@ 200 mm centres - (565 mm^2/m)
Stem bars - 12 mm dia.@ 200 mm centres - (565 mm^2/m)

Geotechnical Survey Report

FSI Ref:	11466
Issue Date:	May 2018
Risk Address:	8 Kentish Town Road London NW1 9NX
Engineer:	Andries Kruger
Company:	Ambigram Architects

Managing Director:	Martin Rush MSc FGS
Finance Director:	Louise Ayres BSc (Hons)
Geotechnical Compliance & Logistics Supervisor:	Perry Martin MCIHT
Laboratory Supervisor:	Jade McLellan
Assistant Geologists:	George Baron Scott Parker

FASTRACK
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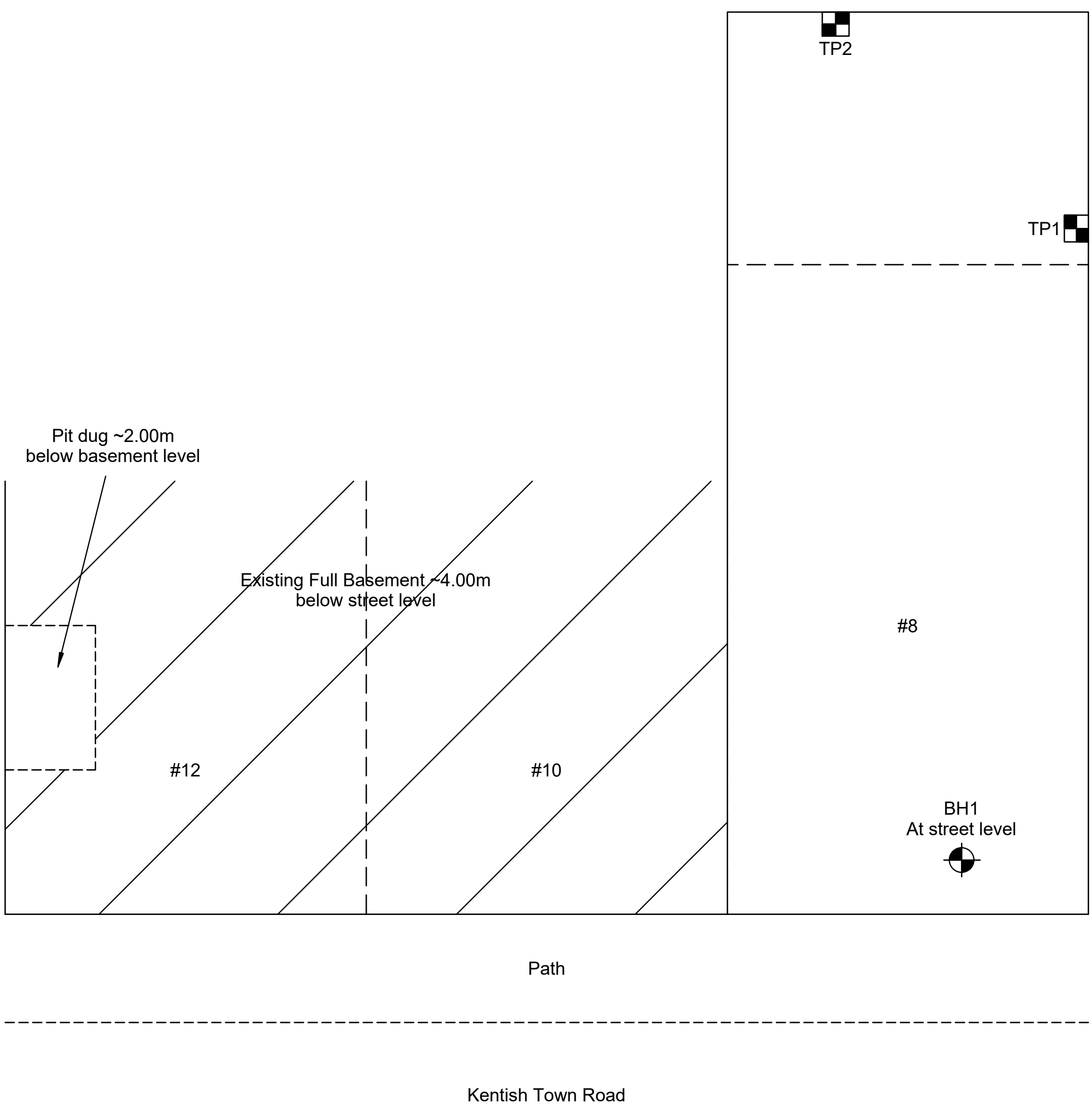
Appendix No: 1
FSI Ref: 11466

SITE PLAN

Property Address: 8 Kentish Town Road, London NW1 9NX

Survey date: 15/05/2018 **Operative:** CE1



The bottom of the pit dug in the basement of #12 is ~6.00m below street level, the same as the closing depth as BH1.
Both noted very similar geologies, therefore it can be assumed that the geology does not change drastically across the site.

Scale:	Drawn by:	Key:
NTS	GB	Trial Pit; Borehole; Manholes; Rain Water Pipe; Soil & Vent Pipe; Surface Water Gully; Foul Water Gully; Shrub; Tree (Conifer); Tree (Deciduous)

FASTRACK	Fastrack Site Investigations Ltd Unit 9, Tyndales Farm Southend Road Maldon CM9 6TQ	Borehole Log	Borehole No. **BH1** Sheet 1 of 1

Project Name: 8 Kentish Town Road	Project No. 11466	Site Date: 15/05/2018	Hole Type BH
Location: London NW1 9NX			Scale 1:35
Client: Ambigram Architects			Logged By CE1

Water Strikes	Sample and In Situ Testing: Depth (m)	Type	Results	Depth (m)	Stratum Description
				0.01	Plywood FLOOR
				0.15	VOID
				0.25	Membrane over POLYSTYRENE
				0.40	Dark Brown Gravelly Sandy Clayey MADE GROUND
					Mid Brown Silty CLAY containing Gypsum
	0.50	D	V (kPa) = 65 V (kPa) = 70		
	1.00	D	V (kPa) = 62 V (kPa) = 66		
	1.50	D	V (kPa) = 62 V (kPa) = 68		
	2.00	D	V (kPa) = 75 V (kPa) = 80		
	2.50	D	V (kPa) = 78 V (kPa) = 80		
	3.00	D	V (kPa) = 80 V (kPa) = 80		
	3.50	D	V (kPa) = 120 V (kPa) = 140		
	4.00	D	V (kPa) = 140		
	4.50	D	V (kPa) = 140		
	5.00	D	V (kPa) = 140		
	5.50	D	V (kPa) = 140		
	6.00	D	V (kPa) = 140	6.00	End of Borehole at 6.000m

Depth scale: 1, 2, 3, 4, 5, 6, 7

Key: D - Disturbed Sample V - Insitu Vane Test MP - Mackintosh Probe Test

Remarks: Borehole closed at 6.00m.
Borehole noted to be dry on completion.

AGS

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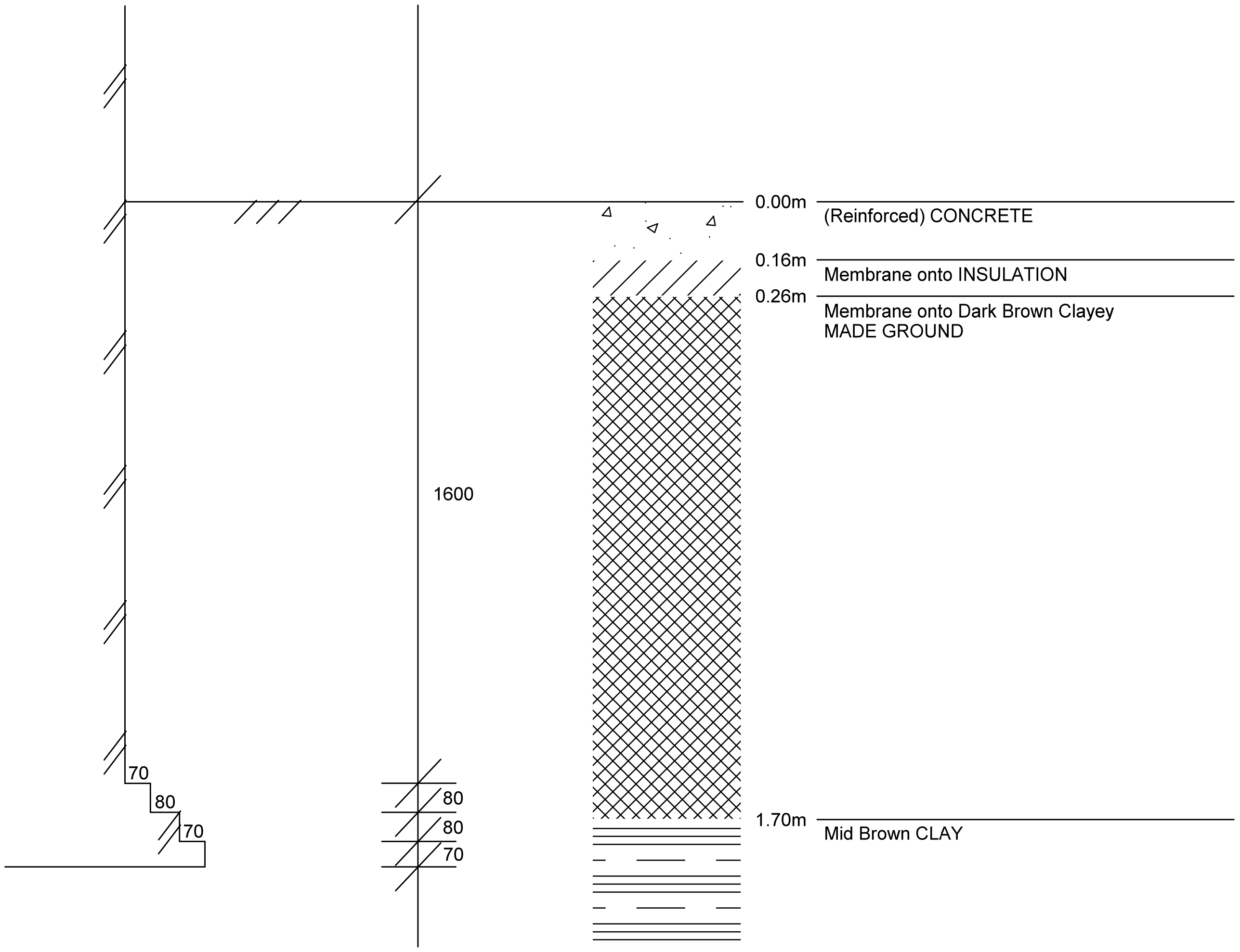
Appendix No: 2
FSI Ref: 11466

TRIAL PIT 1

Property Address: 8 Kentish Town Road, London NW1 9NX

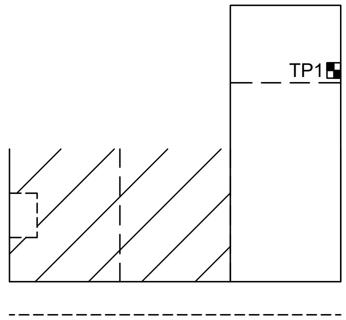
Survey date:15/05/2018 **Operative:** CE1



F.L. (1.83m)
Founding strata: Mid Brown CLAY

D= small disturbed sample, B= large bulk sample, U= undisturbed sample,
MP= mackintosh proble blow counts, V= shear vane reading (kPa)

Trial Pit Location:



Drawn by: GB

Scale: 1:20

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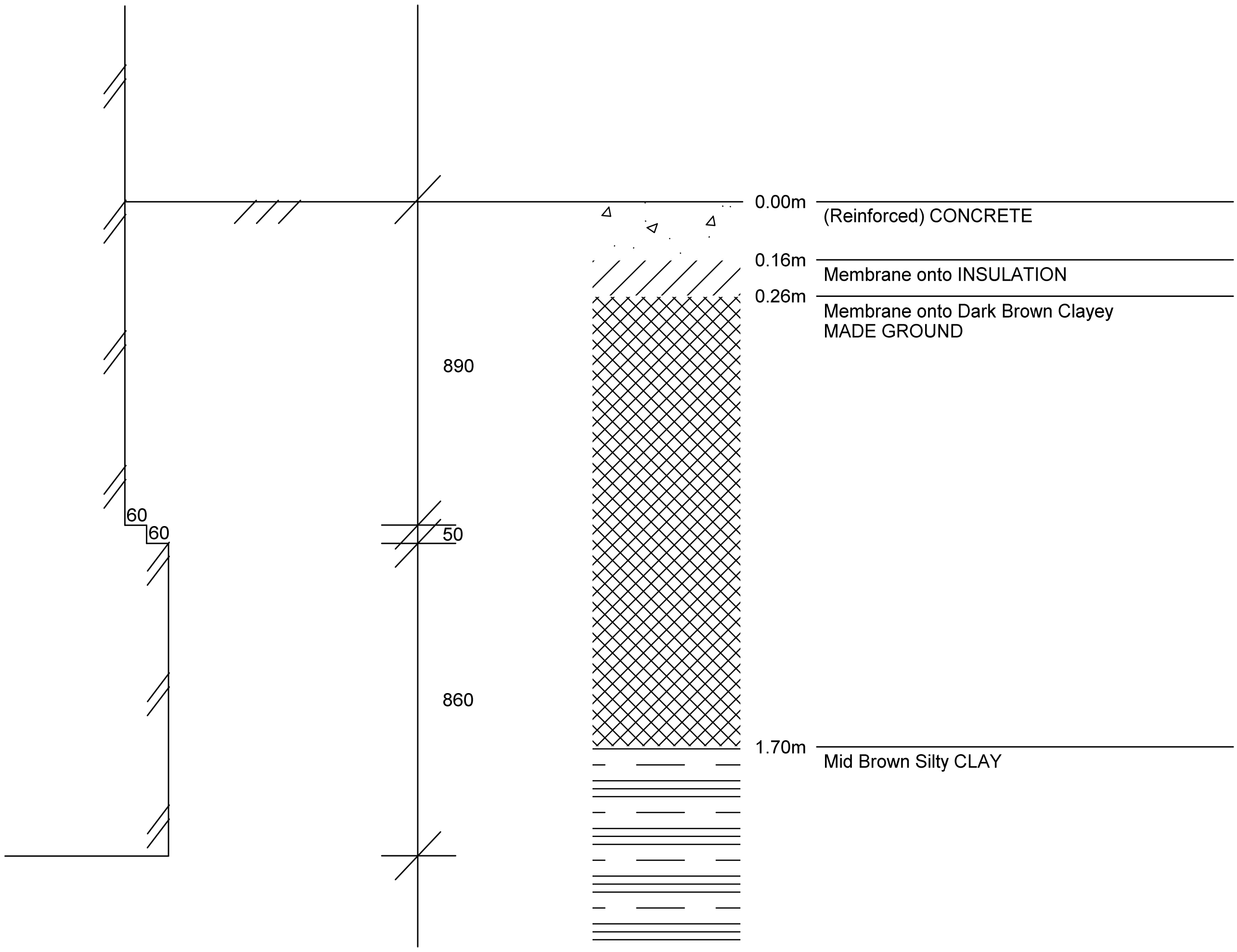
Appendix No: 2
FSI Ref: 11466

TRIAL PIT 2

Property Address: 8 Kentish Town Road, London NW1 9NX

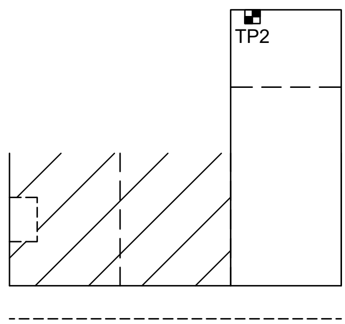
Survey date:15/05/2018 **Operative:** CE1



F.L. (1.80m)
Founding strata: Mid Brown Silty CLAY

D= small disturbed sample, B= large bulk sample, U= undisturbed sample,
MP= mackintosh proble blow counts, V= shear vane reading (kPa)

Trial Pit Location:



Drawn by: GB

Scale: 1:20

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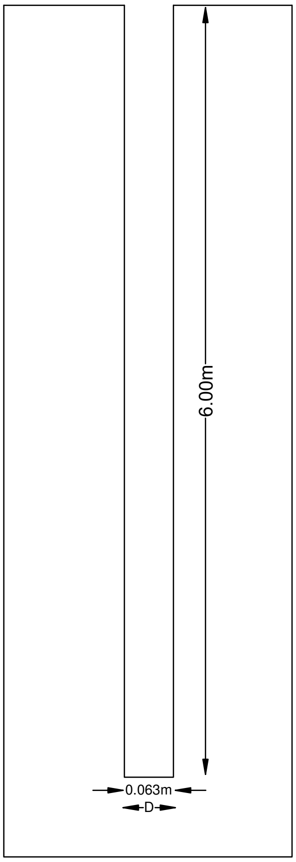
Appendix No: 1
FSI Ref: 11466

Variable Head Test

Property Address: 8 Kentish Town Road, London NW1 9NX

Survey date: 15/05/2018 **Operative:** CE1

Borehole Information



Permeability (k) = A/FT = 0.0031 / (0.17325 x N/A) = Unable to Calculate

A = Cross sectional area of BH = 0.0031 m^2

F = Intake Factor = 2.75 x D = 0.17325

T = Basic Time Factor when H/H^0 = 0.37 = N/A

H^0 = *Head at start of test*

H = *Head at any time t*

Water took 1 minute to drop 0.58m within Borehole 1 before remaining stationary for a further 15 minutes

A Falling Head Permeability Test was carried out within borehole 1 at 6.00m from existing ground level. The permeability test undertaken at this site was a falling head test undertaken in accordance with B.S. 5930:1999 Part 25.4.3 Variable Head Test.

Scale:	**Drawn by:**	**Key:**
NTS	GB	Trial Pit, Borehole, Manholes, Rain Water Pipe, Soil & Vent Pipe, Surface Water Gully, Foul Water Gully, Shrub, Tree (Conifer), Tree (Deciduous)

10 to 12 Kentish Town Road, London NW1

ID	i	Task Mode	Task Name	Duration Working Days	Start	Finish	Predecessors
			Planning				
1			**Phase 1 Construction works**				
2			Site setup and mobilization	4 wks	Mon Week 1	Fri Week 4	2
3			**Substructures**	**35.5 days**	**Mon Week 3**	**Wed Week 9**	
4			Soft strip	4 wks	Mon Week 3	Fri Week 6	2
5			Demolition	5 wks	Mon Week 5	Wed Week 9	2
6			**Basement Works**	**70.5 days**	**Wed Week 7**	**Fri Week 20**	
7			Underpinning	11.5 wks	Wed Week 7	Fri Week 18	2
8			Excavation and disposal	6 wks	Mon Week 12	Fri Week 17	5
9			Below ground drainage	1.5 wks	Mon Week 17	Wed Week 18	5
10			Basement slab	2.5 wk	Wed Week 18	Fri Week 20	1
11			**Superstructures**	**30 days**	**Mon Week 20**	**Fri Week 25**	
12			Basement Columns	3 wks	Mon Week 20	Fri Week 22	2
13			Shuttering of slab	1.5 wks	Mon Week 22	Wed Week 23	2
14			Ground Floor Slab	2.5 wks	Wed Week 23	Fri Week 25	1
15			**Cavity Drainage**	**35 days**	**Mon Week 26**	**Fri Week 32**	
16			Waterproofing	5 wks	Mon Week 26	Fri Week 30	3
17			Commissioning of pumps	2 wks	Mon Week 31	Fri Week 32	5
18			**External envelope**	**30 days**	**Mon Week 1**	**Fri Week 6**	
19			Erect Scaffolding	6 wks	Mon Week 1	Fri Week 6	2