29-31 High Street, Hampton Wick
E0811

engineeria
structural and civil engineers

Structural Impact Assessment

Reference: E0811-EEE-XX-RP-S-0001
Date: 28.07.2023
Revision: P03
Description: For Information

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1.0 Introduction

Engineeria have been commissioned by Mr. and Mrs Frost to prepare a Structural Impact Assessment Report and provide preliminary structural design input for the proposed development at 29-31 High Street, Hampton Wick, Kingston Upon Thames, KT14DA.

The site falls within the area of the London Borough of Richmond Upon Thames.

This report has been prepared in accordance with the guidance document “Planning Advice Note-Good Practice Guide on Basement Developments”, by the London Borough of Richmond Upon Thames, dated May 2015.

The contents of this report are intended to be used in support of the planning application relating to the proposed works only.

2.0 The Project

The project consists of the demolition of the existing two-storey and three-storey units at 29 and 31 High Street and the construction of a series of new units.

The existing units are as described below:
- units 29 and 31 High Street along the main road comprises a retail and commercial unit on ground floor, and residential units on upper floors.
- unit 29b is located to the rear of units 29 and 31 and comprises two light industrial workshops
- a basement is present below unit 31
- two dilapidated storage units and car parking spaces are located to the back of the site.

The new units comprise three Class E units, two workshops at the rear of the site, and eight residential units.
Furthermore, as part of the proposed development, the existing basement will be extended in footprint and height.

Figure 2.1 Existing Basement Plan

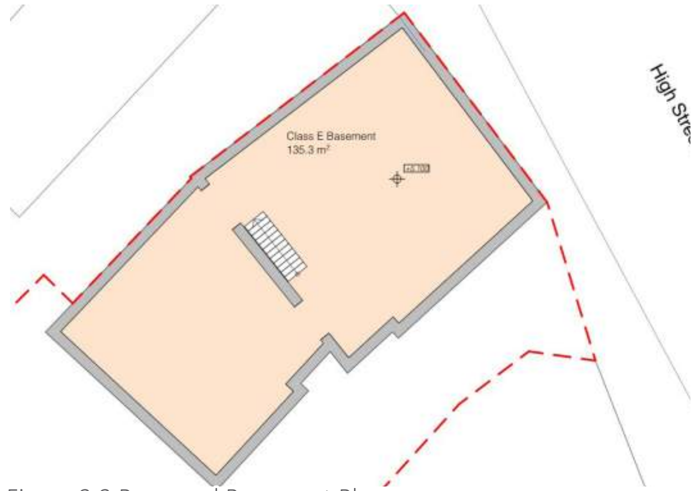


Figure 2.2 Proposed Basement Plan


Figure 2.1 Existing Block Plan

Figure 2.1 Proposed Block Plan

Figure 2.3 Existing Building Facade

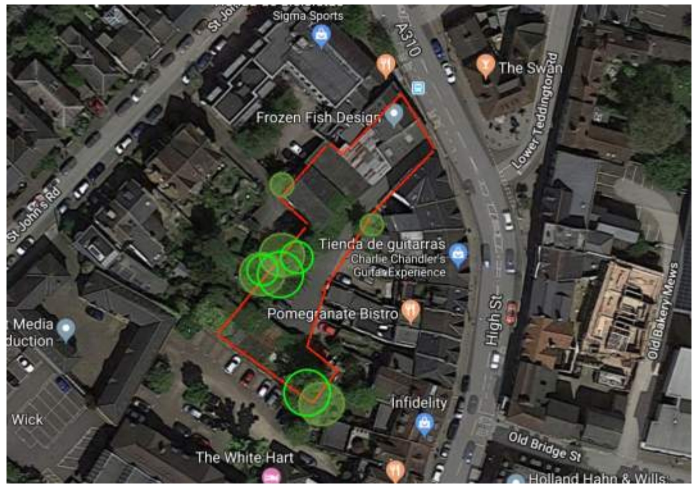


Figure 2.3 Aerial map, red line indicates site boundary, green indicates existing trees

3.0 The Site and The Existing Building

3.1 Site Location

The site is located in Hampton Wick, KT1 4DA, within the London Borough of Richmond Upon Thames. The National Grid reference for the site is 517550, 169500.

The facade of the existing building faces north-east onto High street which is the main point of access for the site.

3.2 Existing Building

The existing buildings are a series of two and three story buildings that appear to consist of traditional timber floors and load bearing masonry wall type construction. The main building (units 29 and 31) appear to be built of traditional brickwork with timber floors. The two back-of-the-house workshops appear to be built of concrete blockwork and lightweight steel structure.

There is a partial basement (cellar) to the front half of the property (below the unit 31).
A walkover survey of the existing building was undertaken in June 2023 (Appendix A).

3.3 Thames Water & Utilities

A Thames Water Asset Location search has been undertaken as part of a Flood Risk Assessment document, produced by others.

An existing Thames Water owned Foul and Surface Water sewer are present on the site. Their position is shown as outside the proposed basement footprint, which also corresponds to a number of inspection chambers noted on the topographical survey drawing provided. However, the exact position of these assets should be verified at a later project stage.

3.4 Adjacent Properties

Number 33a High Street appears to be the mirror copy of the building at 31 High Street. It appears that the boundary line between the two properties is located within the centre of the party wall (i.e. the wall is shared).

The extent of any alteration or extension work (specifically to provide a basement) is unknown, therefore at this stage it is assumed that no full basement extension is present on that side. A similar cellar basement is assumed to be located at the front half of the building, adjacent to 31 High Street.

The site at number 33a High Street appears to be of similar construction to 31 High Street.
For structural design purposes an existing basement has been assumed to be present below number 33a High Street.

3.5 Topography

A topographical survey has been undertaken as part of survey works to the existing structure. The site appears generally sloping downwards from south-west to north-east direction. However, the overall level difference across the site is less than 1m.

3.6 Trees

According to the Screening Assessment Report undertaken by RSK on April 2021, no trees are present on the site; however, a number of trees are present immediately beyond the site boundary.

This information has been verified visually during the site visit undertaken by engineeria.

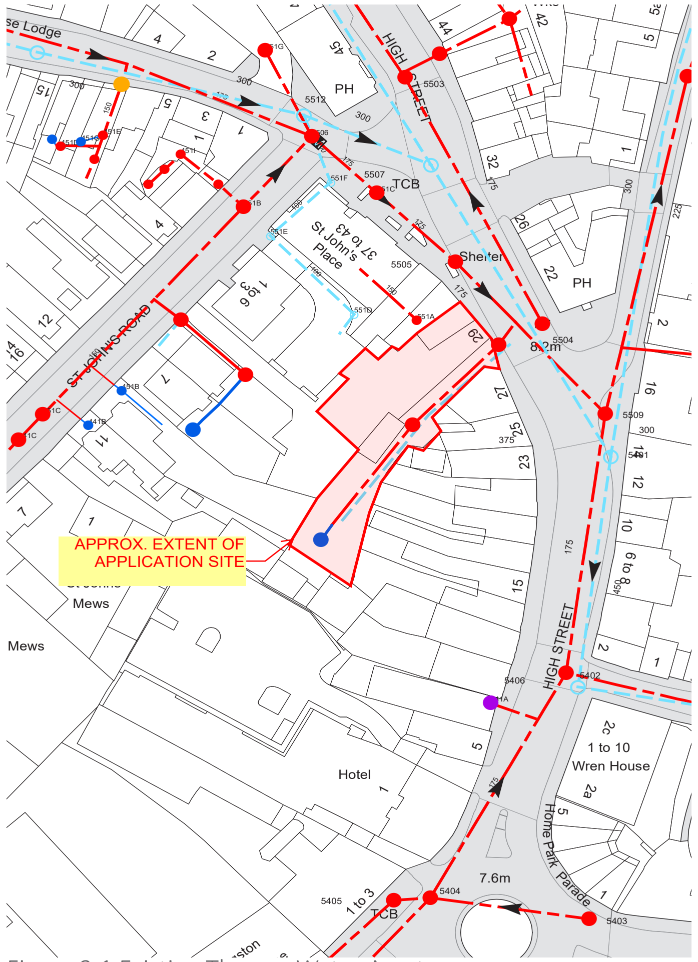


Figure 3.1 Existing Thames Water Assets

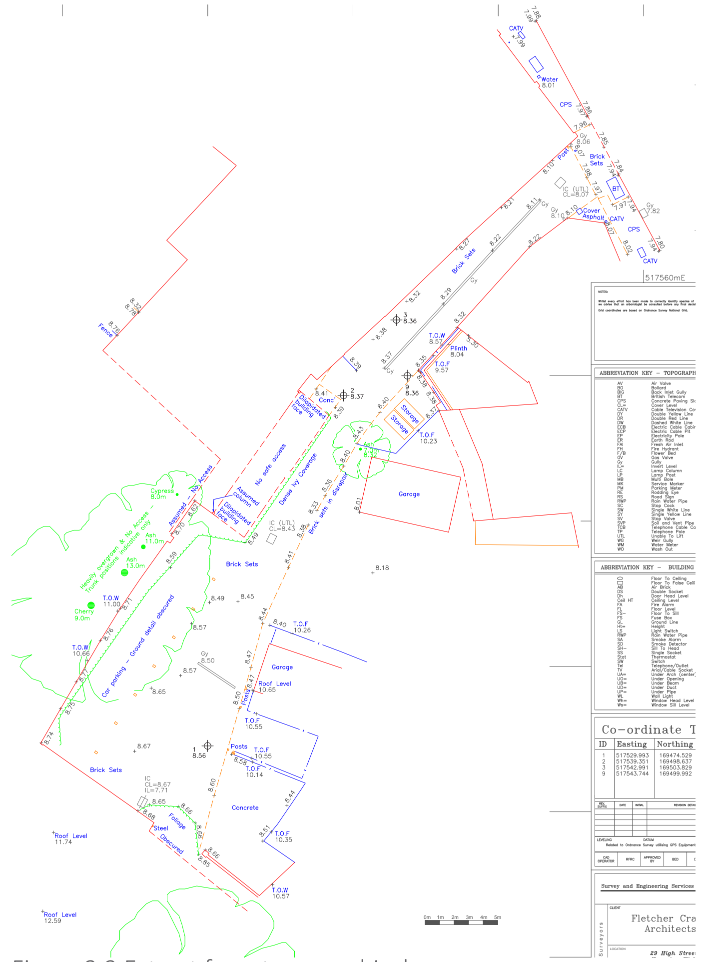


Figure 3.2 Extract from topographical survey

4.1 Ground Investigation Works

Ground Investigation works were undertaken by GEA on 23rd of June 2023. Three boreholes were advanced to a depth varying between 5.45m BGL and 7.10m BGL.

Trial pits to the neighbouring building were not possible due to access constraints.

4.2 Ground Conditions

The ground conditions are generally as follows;

- Made Ground varying in thickness between 1.3m and 1.6m.
- Kempton Park Gravel extending to approximately 7m Below Ground level (approx 6m thick)
- London Clay to depth

Ground water was encountered at depths between 3.55m BGL and 3.7m BGL. This was noted as being below the proposed basement formation level.

4.3 Contamination

The site is not located within a Ground Water source protection zone.

A previous desk study undertaken by RSK did not identify any potential former contaminative uses on the site, and no visual or olfactory signs of contamination were noted during the ground investigation works.

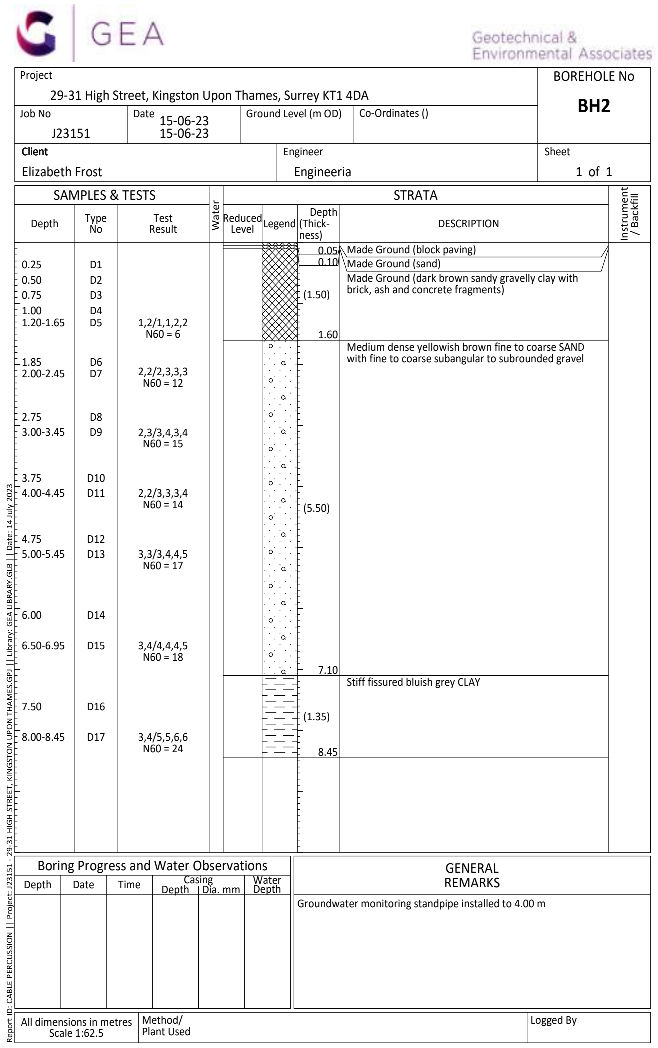
4.4 Ground Movement & Damage Assessment

When considering the loading to be applied and the proposed sequence of construction, engineeria's professional opinion is that any damage to the neighbouring building resulting from the construction would not exceed 'Category 2' as defined by the Burland and Burridge classification, reproduced in CIRIA C580. This is the maximum level of damage considered as acceptable in the borough's basement development guidance document..

The extent of ground movement will be heavily dependent on the level of workmanship and control measures on site, therefore it is assumed that the works would be carried out by a suitably competent and experienced contractor.

It is recommended that during construction a series of monitoring points are established, and regular movement surveys are undertaken. A 'traffic light' system of trigger levels should be agreed with the neighbouring owners, with a level of movement agreed at which works would cease until the cause is established.

Figure 4.1- Borehole locations- Refer to Appendix A for original



GEA — Geotechnical & Environmental Associates

Project			BOREHOLE No
29-31 High Street, Kingston Upon Thames, Surrey KT1 4DA			BH2
Job No J23151	Date 15-06-23 15-06-23	Ground Level (m OD)	Co-Ordinates ()
Client Elizabeth Frost	Engineer Engineeria		Sheet 1 of 1

Depth	Type No	Test Result	Water	Reduced Level	Legend	Depth (Thickness)	DESCRIPTION	Instrument / Backfill
						0.05	Made Ground (block paving)	
						0.10	Made Ground (sand)	
0.25	D1						Made Ground (dark brown sandy gravelly clay with brick, ash and concrete fragments)	
0.50	D2							
0.75	D3					(1.50)		
1.00	D4							
1.20-1.65	D5	1,2/1,1,2,2 N60 = 6				1.60		
1.85	D6						Medium dense yellowish brown fine to coarse SAND with fine to coarse subangular to subrounded gravel	
2.00-2.45	D7	2,2/2,3,3,3 N60 = 12						
2.75	D8							
3.00-3.45	D9	2,3/3,4,3,4 N60 = 15						
3.75	D10							
4.00-4.45	D11	2,2/3,3,3,4 N60 = 14				(5.50)		
4.75	D12							
5.00-5.45	D13	3,3/3,4,4,5 N60 = 17						
6.00	D14							
6.50-6.95	D15	3,4/4,4,4,5 N60 = 18				7.10		
							Stiff fissured bluish grey CLAY	
7.50	D16					(1.35)		
8.00-8.45	D17	3,4/5,5,6,6 N60 = 24				8.45		

Boring Progress and Water Observations

Depth	Date	Time	Casing Depth	Casing Dia. mm	Water Depth

GENERAL REMARKS

Groundwater monitoring standpipe installed to 4.00 m

All dimensions in metres Scale 1:62.5 | Method/ Plant Used | Logged By

Report ID: CABLE PERCUSSION || Project: J23151 - 29-31 HIGH STREET, KINGSTON UPON THAMES.GPJ || Library: GEA LIBRARY.GLB || Date: 14 July 2023

Figure 4.2- Borehole log- Refer to Appendix A for original

5.0 Proposed Structure

5.1 Substructure

The new basement walls are proposed to be formed in reinforced concrete. These are proposed to be constructed in a hit and miss 'underpinning' sequence, in maximum lengths of approximately 1m, to minimise damage to the adjacent party wall, Thames Water assets and public highway present adjacent to the proposed walls. The walls are designed as propped by the ground floor slab in both the temporary and permanent conditions.

Local pad and strip footings are proposed to be formed in the Kempton Park gravel stratum to transfer vertical and horizontal loads to the ground. The ground investigation has confirmed that an allowable net bearing pressure of 150kN/m2 can be considered for the design of these elements.

The proposed basement slab consists of a 250mm thick reinforced concrete slab, to be cast on proprietary collapsible void former (e.g. Cordek Cellcore), to minimise the impact of ground heave. The proposed slab is designed as suspended between foundations due to the provision of anti-heave measures.

The ground floor slab above basement level is proposed as a 250mm thick reinforced concrete suspended slab. In the permanent condition this provides lateral support to the basement walls.

Outside the footprint of the basement, due to the presence of a significant thickness of made ground, a suspended ground floor slab is also proposed.

5.2 Superstructure

The perimeter walls of the new building are assumed to be formed from traditional masonry cavity walls above ground floor level.

Where walls are not continuous through the building (e.g. where shop fronts are proposed at ground floor level, a series of steel beams are proposed to support the masonry above.

The upper floor construction is proposed as timber floor joists (typically 225mm x 50mm at 300 c/c), spanning between either load bearing masonry walls or intermediate steel beams.

Steel columns are proposed to reduce the span of the proposed steel beams. The final position of these is to be co-oridnated with the project architect at a later project stage.

5.3 Stability

To the upper floors of the building, masonry cross walls will provide lateral stability against wind and notional horizontal loads

Where walls are discontinuous, moment resisting connections between steel columns and beams are proposed, in order to compensate for the removal of masonry walls.

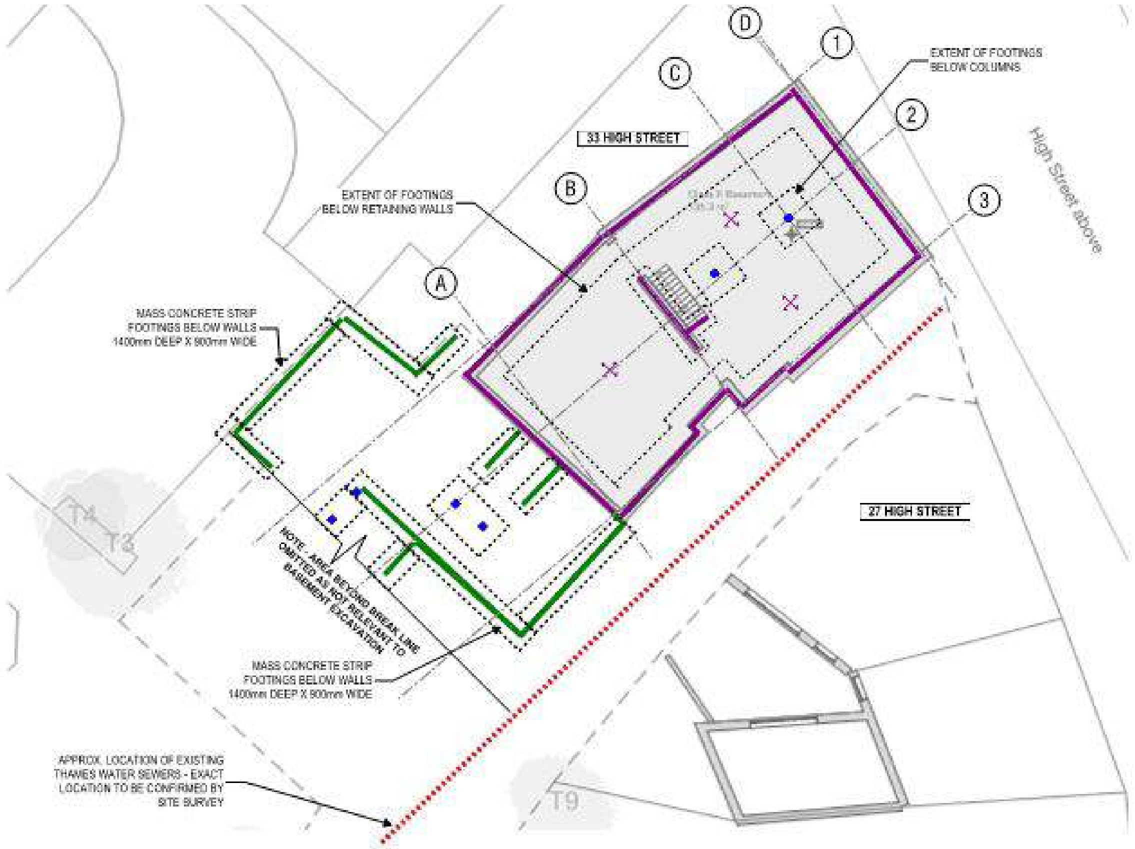


Figure 5.1 Proposed Basement Plan

Figure 5.2 Proposed First Floor Plan

6.0 Proposed Construction Sequence

6.1 Construction Sequence Stages

The proposed construction sequence is as follows:

Stage 0:
- Site set up

Stage 1:
- Using hit and miss underpinning sequence (refer to plan), dig down to underside of corbel level in pins marked “1”.
- Install mass concrete underpin to 75mm below underside of existing foundation. Provide shear key to adjacent pins.
- Install dry mortar pack with non-shrink additive to underside of existing foundation, well rammed in.
- Cast wall sections and wall toe with continuity rebars for future connection to basement slab.
- Backfill excavation using well compacted granular material or leave excavation support in place.
- Repeat for remaining pins, in sequence indicated.

Stage 2:
- Excavate down using RC underpinning sequence, installing trench sheeting and struts/waling beams to support excavation. Exact size of pins to suit contractor’s temporary works design.
- Cast retaining wall sections and wall toe with continuity rebars for future connection to basement slab.

Stage 3:
- Excavate ground level within basement to underside of upper level of horizontal props (to contractor’s temporary works design) and install horizontal props.
- Excavate to underside of lower level of horizontal props and install props before excavating to formation level.
- Pull out continuity bars from retaining wall toes and construct remaining basement slab between. This provides permanent lower level horizontal prop.
- Construct ground floor slab to provide permanent horizontal prop to top of retaining walls and remove temporary propping.

For full details, refer to Appendix E.

Detailed design of all temporary works and the final construction sequence are subject to final design at a later project stage.

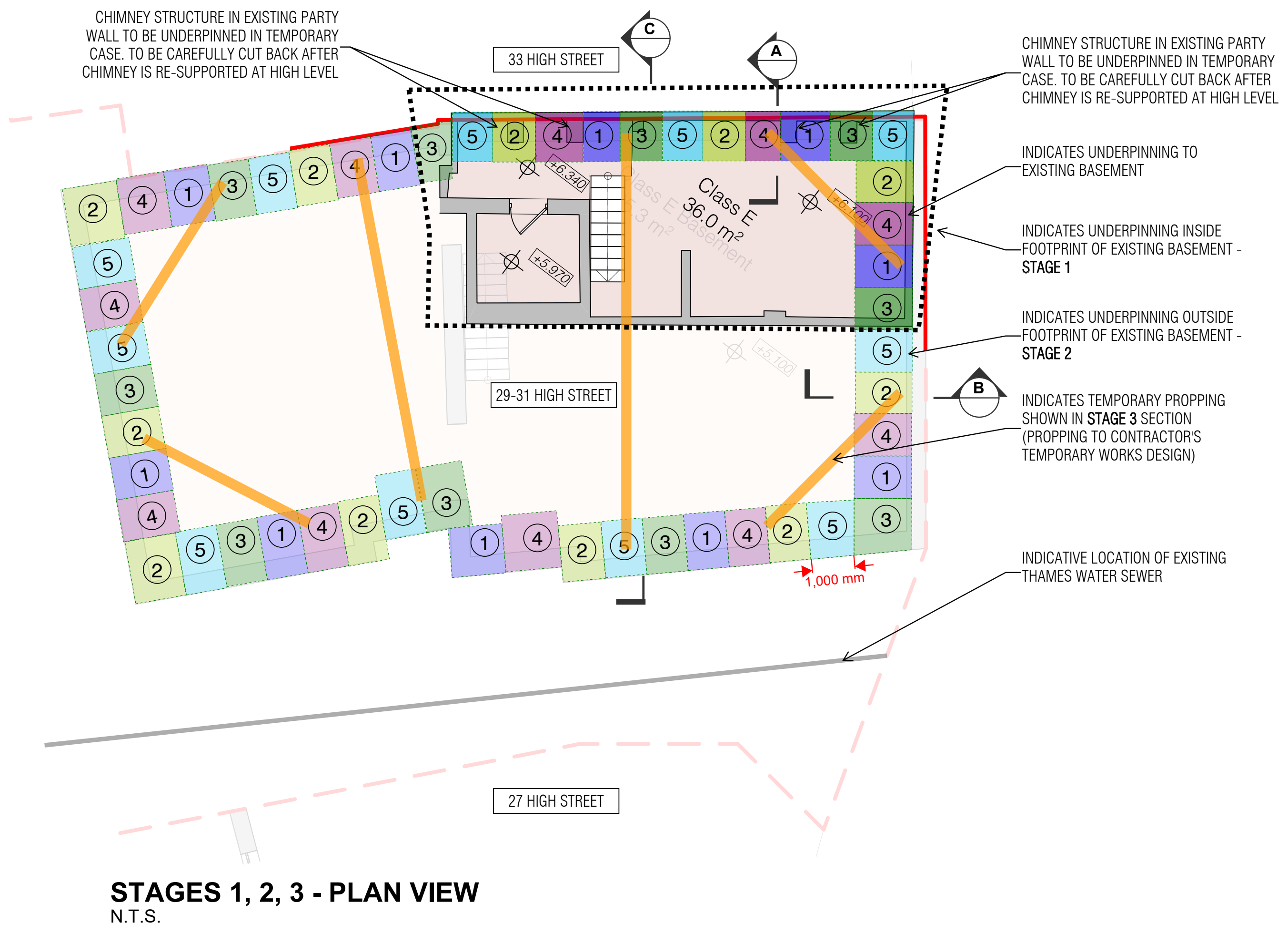


Figure 6.1- Extract from Proposed Construction Sequence

7.0 Conclusions

- A site investigation has been undertaken, confirming the existing ground conditions present on site, including ground water levels and geotechnical parameters.
- Based on this, a proposed structural design has been produced which demonstrates a feasible manner of constructing the proposed basement development.
- A sequence of works is proposed which will allow the proposed basement to be constructed in a safe manner, subject to detailed design of temporary works by the contractor at a later stage.
- Provided that the works are executed in the manner indicated and by a suitably experienced contractor, it is anticipated that ground movements in adjacent properties will be limited to Damage Class 2 or lower (within the limits deemed acceptable as part of the Borough of Richmond’s guidance document).

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Appendix A - Preliminary Site Investigation Report (by GEA)

29-31 High Street
Hampton Wick
Kingston Upon Thames
KT1 4DA

Ground Investigation Report
& Basement Impact Assessment

Mrs. Elizabeth Frost

July 2023

J23151
Rev 1

Ground investigation | Geotechnical consultancy | Contaminated land assessment

Report prepared by

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Report checked by

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Report approved by

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Managing Director

Rev No	Status	Revision Details	Date	Approved for Issue
0	Final		14 July 2023	
1	Final		21 July 2023	SH

This report has been issued by the GEA office indicated below. Any enquiries regarding the report should be directed to the report project engineer at the office indicated or to Steve Branch in our main Herts office.

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	Nottinghamshire	tel 01509 674888
	Manchester	tel 0161 209 3032

Contents

This executive summary contains an overview of the key findings and conclusions. No reliance should be placed on any part of the executive summary until the whole of the report has been read. Other sections of the report may contain information that puts into context the findings that are summarised in the executive summary.

Brief

This report describes the findings of a ground investigation and basement impact assessment (BIA), carried out by Geotechnical and Environmental Associates Limited (GEA) on the instructions of Mrs Elizabeth Frost, with respect to the redevelopment of the site through the partial demolition of the existing buildings and subsequent construction of new units as well as the deepening and extension of the existing basement.

The purpose of the investigation has been to review the site history, to determine the ground conditions and engineering properties in order to undertake a BIA in support of a planning application for the proposed development. Whilst outline advice for the proposed development is provided, this report does not form a design report and no reliance can be made as such; a subsequent report will be issued, which should be referred to for design aspects of the proposed development.

The site has previously been the subject of a desk study report by Alban SI (report reference 20/11967/KJC, dated November 2023), and a screening assessment undertaken by RSK (report reference 1921843-01 (01), dated April 2023). The findings of these reports have been reviewed and referred to where appropriate.

Site history

The earliest map studied, dated 1865, shows the site to have been occupied by a number of buildings, some falling within the curtilage of the site and others extending beyond the site boundaries.

The site remained unchanged until some time between 1913 and 1915, when a number of the buildings in the northeast of the site fronting onto the High Street are shown to have been demolished, along with a number of other buildings on the site. By 1955, the existing buildings are labelled as Nos 29 and 31 and No 29 is detailed to be part of an engineering works, which are no longer shown on the map dated 1969. The site has remained essentially unchanged from that time to the present day.

Ground conditions

The investigation has generally confirmed the expected ground conditions in that, beneath a moderate thickness of made ground, the Kempton Park Gravel was encountered over the London Clay Formation. The made ground extended to depths of between 1.30 m and 1.60 m and comprised a surface covering of block paving over sand, over dark grey and brown sandy gravelly clay with variable inclusions of ash, brick and concrete fragments. The underlying Kempton Park Gravel generally comprised yellowish brown fine to coarse sand and fine to coarse subangular to subrounded gravel, extending to the full depth of Borehole No 1, of 5.45 m, and to a depth of 7.10 m in Borehole No 2. Within Borehole No 3, the Kempton Park Gravel comprised yellowish brown fine to coarse sand and fine to coarse subangular to subrounded gravel, extending to a depth of 3.00 m, whereupon brown silty slightly clayey sand with fine to coarse subangular to subrounded gravel was encountered and extended to the full depth of the borehole, of 5.45 m. The underlying London Clay comprised stiff fissured bluish grey clay and extended to the full depth of the investigation, of 8.45 m.

Groundwater was encountered within the Kempton Park Gravel at depths of between 3.60 m and 3.70 m during drilling and has subsequently been measured within the standpipes at depths of 3.55 m, 3.77 m and 3.87 m, within Borehole Nos 1, 2 and 3, respectively.

Recommendations

The proposed basement will extend to a depth of approximately 3.20 m below street level, such that formation level is expected to be within the Kempton Park Gravel. Groundwater is not expected to be encountered within the basement excavation, and the use of either traditional mass concrete underpinning or a contiguous bored pile wall is therefore considered suitable for the formation of new retaining walls. However, ongoing monitoring should be carried out to determine the extent of any seasonal variation and confirm this.

On the basis that a dry excavation can be maintained, spread foundations excavated below basement level may be designed to apply a net allowable bearing pressure of 150 kN/m^2.

Basement Impact Assessment

It has been concluded that the potential impacts identified by the BIA can be mitigated by appropriate design and standard construction practice. Groundwater is unlikely to be encountered within the basement excavation and will still be able to flow around and beneath the basement structure. As the new basement does not close a pathway or create a cut-off, it is considered that the groundwater will follow a pathway around and beneath the proposed basement and will not cause a rise in groundwater level on the upstream side such that it is considered that the proposed basement will not have an impact on the local hydrogeological setting.

Part 1: Investigation Report

This section of the report details the objectives of the investigation, the work that has been carried out to meet these objectives and the results of the investigation. Interpretation of the findings is presented in Part 2.

1.0 Introduction

Geotechnical and Environmental Associates Limited (GEA) has been commissioned by Mrs. Elizabeth Frost, to carry out a ground investigation at Nos 29-31 High Street, Hampton Wick, Kingston Upon Thames KT1 4DA. This report also forms part of a Basement Impact Assessment (BIA), which has been carried out in accordance with guidelines from the London Borough of Richmond Upon Thames in support of a planning application. Whilst outline advice for the proposed development is provided, this report does not form a design report and no reliance can be made as such; a subsequent report will be issued, which should be referred to for design aspects of the proposed development.

The site has previously been the subject of a desk study report by Alban SI (report reference 20/11967/KJC, dated November 2023), and a screening assessment undertaken by RSK (report reference 1921843-01 (01), dated April 2023). The findings of these reports have been reviewed and referred to where appropriate.

1.1 Proposed Development

It is understood that the existing retail units fronting onto the High Street, with commercial and retail space on the ground floor and residential flats on the upper floors, will be partly demolished, along with the workshops forming part of No 29 High Street and the delipidated workshops to the rear of the site. In their place, the following will be constructed;

- two Class E units located at the ground floors of Nos 29 and 31 High Street with a finished floor level of 8.10 m AOD;
- a Class E business unit to the rear of Nos 29 and 31 High Street, with a finished floor level of 8.10 m AOD and 8.39 m AOD respectively, and two workshops at the rear of the site beyond the car parking area, with finished floor levels of 8.10 m AOD and 8.71 m AOD respectively;
- eight residential units comprising six flats located at the first and second floors of Nos 29 and 31 High Street and two further flats located to the rear of the site above the workshops at first and second floor level; and
- an extension to the existing basement, both laterally and vertically, beneath No 31 High Street, ancillary Class E units, with a finished floor level of 5.10 m AOD. The existing basement floor level will be lowered by 1.24 m, with an increase in area of 99.3 m^2, from 36.0 m^2 to 135.3 m^2.

Outside of the footprint of the basement, the development will effectively maintain existing ground levels.

The existing site access between Nos 27 and 29 High Street will be maintained and within the courtyard there are two proposed pedestrian entrances into the commercial units and upper floor residential flats. The proposed parking layout will be similar to the existing, with four spaces allocated for the residential units and one allocated for the commercial and retail units.

Surface water from the proposed development will be managed by attenuation prior to discharge into the nearby sewer. In order to prevent flooding, both on and off the site, a variety of SuDS will be utilised to control surface water inflows, including an area of permeable paving, a modular storage tank and a green roof.

This report is specific to the proposed development and the advice herein should be reviewed if the development proposals are amended.

1.2 Purpose of Work

The principal technical objectives of the work carried out were as follows:

- to review the previous desk study and screening assessment carried out for the site;
- to determine the ground conditions and their engineering properties; and,
- to assess the impact of the proposed basement development on the surrounding environment.

1.3 Scope of Work

In order to meet the above objectives, the below work was carried out:

- a review of the previous reports prepared for the site and proposed development;
- a walkover survey of the site carried out in conjunction with the fieldwork.
- three boreholes advanced through rotary continuous flight auger (CFA) methods to depths of between 5.45 m and 8.45 m;
- standard penetration tests (SPTs) carried out at regular intervals within the boreholes to provide quantitative data on the strength of the soils;
- the installation of three groundwater monitoring standpipes to depths of between 4.00 m and 4.50 m, and a single monitoring visit; and
- provision of a report presenting and interpreting the above data.

The exploratory methods adopted in this investigation have been selected on the basis of the constraints of the site including but not limited to access and space limitations, together with any budgetary or timing constraints. Where it has not been possible to reasonably use an EC7 compliant investigation technique a practical alternative has been adopted to obtain indicative soil parameters and any interpretation is based upon engineering experience, local precedent where applicable and relevant published information.

2.0 The Site

2.1 Site Description

The site is located in the London Borough of Richmond Upon Thames, approximately 250 m southeast of Hampton Wick Railway station. It fronts onto the High Street to the northeast. The site may be additionally located by National Grid Reference 517522, 169449.

The site covers an irregularly shaped area of approximately 921 m^2 and currently comprises two retail units of Nos 29 and 31 High Street, which make up the site frontage. These units are two and three storeys in height respectively and the upper floors comprise residential units. No 29b is located to the rear and comprises two light industrial workshops. A small basement is located below No 31 High Street with a finished floor level of between 2.23 m below ground level (5.97 m AOD) and 1.86 m below ground level (6.34 m AOD). Two dilapidated storage units and car parking spaces occupy the rear of the site.

The site is essentially level and is formed at a ground level of about 8.20 m AOD and is almost entirely hardcovered with only small areas of soft landscaping around part of the sites perimeter. No trees are present on the site but there are a number of mature deciduous trees present immediately beyond the site boundary to the northwest and southwest.

2.2 Summary of Previous Desk Study Findings

The earliest map studied, dated 1865, shows the site to have been occupied by a number of buildings, some falling within the curtilage of the site and others extending beyond the site boundaries. The site remained unchanged until some time between 1913 and 1915, when a number of the buildings in the northeast of the site fronting onto the High Street are shown to have been demolished, along with a number of other buildings on the site. By 1955, the existing buildings are labelled as Nos 29 and 31 and No 29 is detailed to be part of an engineering works, which are no longer shown on the map dated 1969. The site has remained essentially unchanged since that time to the present day.

There are no reported active or historical landfills or waste sites located within 500 m of the site.

Reference to records compiled by the Health Protection Agency (formerly the National Radiological Protection Board) indicates that the site falls within an area where less than

1% of homes are affected by radon emissions and therefore radon protective measures will not be necessary.

The British Geological Survey (BGS) map of the area indicates that the site is underlain by the Kempton Park Gravel, which is underlain by the London Clay Formation.

The closest BGS archive borehole record to the site is located about 50 m south of the site. The borehole record indicates that made ground extended to a depth of 1.37 m, whereupon the Kempton Park Gravel was encountered and initially comprised a horizon of soft grey/brown silty clay extending to a depth of 2.95 m below ground level. Medium to coarse flint gravel was then encountered and extended to a depth of 5.64 m, below which the London Clay initially comprised firm brown clay extending to a depth of 6.10 m, whereupon firm to stiff grey clay was encountered and extended to the full depth of the borehole, of 6.40 m below ground level.

The RSK report details that, because of the brownfield nature of the site and the presence of an existing basement, it is anticipated that made ground and/or reworked natural ground will be present on site.

The Kempton Park Gravel is classified as a Secondary 'A' Aquifer, which refers to permeable layers capable of supporting water supplies at a local rather than strategic scale, and in some cases forming an important source of base flow to rivers. The London Clay Formation is classified as an Unproductive Stratum, rather than its former classification as a non-aquifer, referring to rock layers or drift deposits with low permeability that have negligible significance for water supply or river base flow. The London Clay cannot support a water table or effectively transmit groundwater flow because of its low permeability and cohesive nature. The permeability will be predominantly secondary, through fissures in the clay. Published data indicates the horizontal permeability of the London Clay to generally range between 1×10^{-11} m/s and 1×10^{-9} m/s.

The site is not located within a groundwater Source Protection Zone.

The nearest surface water feature to the site is the River Thames, located about 130 m to the east of the site. The river flows in a generally south to north direction in the vicinity of the site. Whilst the river has a tidal influence, the tidal influence is limited to areas downstream of Teddington Lock, which is located 2.8 km downstream of the site. Therefore, the river and surrounding groundwater is not considered to be affected by tidal influence.

The site is understood to be in an area of moderate risk from flooding from rivers and sea, and groundwater flooding. However, it is at low risk from all other potential flood sources.

Groundwater is likely to be present near the boundary between the relatively high permeability Kempton Park Gravel and the low permeability London Clay and is likely to flow in a generally southerly direction, with the local topography and towards the River Thames. Groundwater was struck within the aforementioned BGS borehole at a depth of 1.37 m below ground level with a standing water level of 1.52 m observed upon completion.

The site is almost entirely covered by the existing building and hardstanding and therefore infiltration of rainwater into the ground beneath the site is limited such that the majority of surface runoff is likely to drain into combined sewers in the road.

3.0 Screening

The Richmond Upon Thames planning guidance suggests that any development proposal that includes a basement should be screened to determine whether or not a full BIA is required.

The previous screening report produced by RSK is included in the appendix, with the report identifying the following potential impacts.

Potential Impact	Potential Consequence
The recorded water table potentially extends above the base of the proposed subsurface structure	The development may impact groundwater flow and levels in the surrounding area.
Infiltration methods are proposed as part of the site drainage strategy	A change in the amount of water entering the ground and the rates of percolation could impact groundwater flow and level
The proposed basement excavation is likely to extend below the local water table or spring line	
the site is underlain by an aquifer and or permeable geology	
The development will increase the depth of foundations with respect to the foundations of the neighbouring structures.	The movements associated with the construction of the new basement may result in damage to nearby structures.
The ground at the site has potentially been previously worked	

The proposed subsurface development will potentially impact the flow profile of throughflow groundwater to downstream areas	Could result in a flood risk around the vicinity of the site or at downstream locations
The proposed development may result in an increase groundwater risk to neighboring properties.	

These potential impacts have been investigated through the ground investigation, as described below.

3.1 Exploratory Work

In order to meet the objectives described in Section 1.2, three boreholes were advanced through rotary percussive methods to depths of between 5.45 m and 8.45 m.

Three groundwater monitoring standpipes were installed to depths of between 4.00 m and 4.50 m to facilitate groundwater monitoring, which has been carried out on a single occasion to date, approximately two weeks after installation.

During boring, disturbed samples were obtained from the boreholes for subsequent laboratory examination and testing. Standard Penetration Tests (SPTs) were carried out at regular intervals to provide additional quantitative data on the strength of soils encountered.

All of the above work was carried out under the supervision of a geotechnical engineer from GEA, with the boreholes positioned to provide general coverage of the site, whilst avoiding known buried services. The borehole records are appended, together with a site plan indicating the exploratory positions.

4.0 Ground Conditions

The investigation has generally confirmed the expected ground conditions in that, beneath a moderate thickness of made ground, the Kempton Park Gravel was encountered, which was underlain by the London Clay Formation.

4.1 Made Ground

Below a surface covering of block paving over sand, the made ground comprised dark grey and brown sandy gravelly clay with variable, gravel, brick, ash and concrete fragment content. The made ground extended to depths of between 1.30 m and 1.60 m.

Apart from the presence of fragments of extraneous material noted above, no visual or olfactory evidence of contamination was observed during the fieldwork.

4.2 Kempton Park Gravel

This stratum generally comprised medium dense yellowish brown fine to coarse sand with fine to coarse subangular to subrounded gravel, extending to the full depth of Borehole No 1, of 5.45 m and to a depth of 7.10 m in Borehole No 2. Within Borehole No 3, the Kempton Park Gravel comprised yellowish brown fine to coarse sand and fine to coarse subangular to subrounded gravel, extending to a depth of 3.00 m, whereupon brown silty slightly clayey sand with fine to coarse subangular to subrounded gravel was encountered and extended to the full depth of the borehole, of 5.45 m.

4.3 London Clay

The London Clay was encountered in Borehole No 2 only and comprised stiff fissured bluish grey clay and extended to the full depth of the investigation, of 8.45 m.

4.4 Groundwater

Groundwater was encountered at depths of between 3.60 m and 3.70 m during drilling and has subsequently been measured within the standpipes at depths of 3.55 m, 3.77 m and 3.87 m, within Borehole Nos 1, 2 and 3, respectively.

Part 2: Design Basis Report

This section of the report provides an interpretation of the findings detailed in Part 1, in the form of a ground model, and then provides advice and recommendations with respect to the proposed development.

5.0 Introduction

It is understood that the existing retail units fronting onto the High Street will be partly demolished, along with the workshops forming part of No 29 High Street, 29 High Street and the dilapidated workshops to the rear of the site. In their place two Class E units will be constructed along the frontage with the High Street with a finished floor level of 8.10 m AOD, along with a class E business unit and two workshops to the rear. Eight residential units will be constructed on the floor above and an extension to the existing basement below No 31 High Street, both laterally and vertically, with a finished floor level of 5.10 m AOD. The existing basement floor level will be lowered by between an additional 1.24 m and 3.20 m, and the basement area will be increased by 99.3 m^2, from 36.0 m^2 to 135.3 m^2.

6.0 Ground Model

The desk study has revealed that the site has not had a potentially contaminative historical use, as it has been developed with the unspecified commercial and residential buildings since prior to 1878. On the basis of the fieldwork, the ground conditions at this site can be characterised as follows:

- below a moderate thickness of made ground, the Kempton Park Gravel is present, and is underlain by the London Clay which extends to the maximum depth of the investigation, of 8.45 m;
- the made ground comprises dark grey and brown sandy gravelly clay with variable inclusions of ash, brick and concrete fragments and extends to depth of between 1.30 m and 1.60 m;
- the Kempton Park Gravel generally comprises medium dense yellowish brown fine to coarse sand and fine to coarse subangular to subrounded gravel, or sandy gravel extending to a depth of 7.10 m;
- the London Clay comprises stiff fissured bluish grey clay and extends to the full depth of the investigation, of 8.45 m; and,
- groundwater is present within the Kempton Park Gravel at a depth of approximately 3.50 m.

7.0 Advice & Recommendations

It is understood that the basement will be lowered to a maximum depth of approximately 3.20 m below existing street level which equates to 1.24 m below the level of the existing basement. Formation level for the proposed basement should therefore be within the medium dense sand of the Kempton Park Gravel. On the basis of the fieldwork and subsequent monitoring, groundwater is not expected to be encountered within the basement excavation.

7.1 Basement Construction

Groundwater has been measured at depths of between 3.55 m and 3.87 m below ground level within the monitoring standpipes, and therefore inflows of groundwater are not expected to be encountered within the basement excavation, although in line with good practice, ongoing monitoring of the standpipes should be carried out to determine the extent of any seasonal variation. It is however plausible that shallow inflows of perched water may be encountered from within the made ground, although such inflows should be controllable using conventional sump pumping. It is always advisable that where possible, a number of trial excavations are be carried out, to depths as close to the full basement depth as possible, to provide an indication of stability and the extent to which the excavation may be affected by any groundwater inflows.

The design of basement support in the temporary and permanent conditions needs to take account of the necessity to maintain the stability of the surrounding structures and the possible requirement to control groundwater inflows.

There are a number of methods by which the sides of the basement excavation could be supported in the temporary and permanent conditions. The choice of wall may be governed to a large extent by whether it is to be incorporated into the permanent works and have a load bearing function. For the ground conditions at this site a bored pile wall could be utilised to support the basement excavation and could have the advantage of being incorporated into the permanent works to provide support for structural loads. A contiguous wall could be feasible at this site, with some localised grouting between piles if instability and minor groundwater inflows are encountered. Alternatively, the use of traditional concrete underpinning could be utilised to construct the retaining walls, although if instability is encountered within thin the Kempton Park Gravel, sacrificial backing boards may be required to allow the concrete to be cast.

The ground movements associated with the basement excavation will depend on the method of excavation and support and the overall stiffness of the basement structure in the temporary condition. Thus, a suitable amount of propping will be required to provide the necessary rigidity. In this respect the timing of the provision of support to the wall will have an important effect on movements.

7.1.1 Basement Retaining Walls

The following parameters are suggested for the design of the permanent basement retaining walls.

Stratum	Bulk Density (kg/m³)	Effective Cohesion (c′ – kN/m²)	Effective Friction Angle (ϕ′ – degrees)
Made ground	1700	Zero	27
Kempton Park Gravel	1800	Zero	34
London Clay	1900	Zero	23

Monitoring of the standpipe should be continued to assess the design water level, but based on the monitoring carried out to date, groundwater may be assumed to be below basement level; the advice in BS8102:2009[1] should also be followed in this respect.

7.1.2 Basement Heave

The 1.24 m to 3.20 m deep excavation of the basement will result in a differential net unloading of between around 25 kN/m² to 65 kN/m², which will result in differential heave of the underlying London Clay. This will comprise immediate elastic movement, which will account for approximately 40 % of the total movement and be expected to be complete during the construction period, and long-term movements, which will theoretically take many years to complete. These movements will, to some extent, be mitigated by the loads applied by the proposed development and the remaining thickness of Kempton Park Gravel between the basement and London Clay. Further analysis should be undertaken once the proposed loads are known.

7.2 Spread Foundations

Moderate width strip or pad foundations bearing on the medium dense sand of the Kempton Park Gravel, constructed at basement formation level, may be designed to apply a net allowable bearing pressure of 150 kN/m². This value incorporates an adequate factor of safety against bearing capacity failure and should ensure that settlement remains within normal tolerable limits.

7.3 Shallow Excavations

On the basis of the borehole findings it is considered that it will be generally feasible to form relatively shallow excavations terminating within the Kempton Park Gravel without the requirement for lateral support, although localised instabilities may occur where more granular material or groundwater is encountered.

Significant inflows of groundwater into shallow excavations are not generally anticipated, although minor seepages may be encountered from perched water tables within the made ground, although such inflows do not pose a risk to the surrounding neighbouring structures or significant instability and should be suitably controlled by sump pumping.

If deeper excavations are considered or if excavations are to remain open for prolonged periods it is recommended that provision be made for battered side slopes or lateral support. Where personnel are required to enter excavations, a risk assessment should be carried out and temporary lateral support or battering of the excavation sides considered in order to comply with normal safety requirements.

7.4 Basement Floor Slab

Following the excavation of the basement, formation level will be within the granular soils of the Kempton Park Gravel and it should be possible to adopt a moderately loaded ground bearing floor slab for both the reduced lower ground floor and basement floor slabs. As recommended previously, further analysis will need to be undertaken to determine the magnitude of heave arising due to the basement excavation, in order to inform the final design of the slab.

1 BS8102 (2009) *Code of practice for protection of below ground structures against water from the ground*

Part 3: Basement Impact Assessment

This section of the report evaluates the direct and indirect implications of the proposed project, based on the findings of the previous screening and scoping, site investigation and ground movement assessment.

8.0 Introduction

The screening identified a number of potential impacts. The desk study and ground investigation information has been used below to review the potential impacts, to assess the likelihood of them occurring and the scope for reasonable engineering mitigation.

8.1 Potential Impacts

The table below summarises the previously identified potential impacts and the additional information that is now available from the ground investigation in consideration of each impact.

Potential Impact	Potential Consequence
The recorded groundwater table potentially extends above the base of the proposed subsurface structure	The development may impact groundwater flow and levels in the surrounding area.
Infiltration methods are proposed as part of the site drainage strategy	A change in the amount of water entering the ground and the rates of percolation could impact groundwater flow and level
The proposed basement excavation is likely to extend below the local water table or spring line	
the site is underlain by an aquifer and or permeable geology	
The development will increase the depth of foundations with respect to the foundations of the neighbouring structures.	The movements associated with the construction of the new basement may result in damage to nearby structures.
The ground at the site has potentially been previously worked	
The proposed subsurface development will potentially impact the flow profile of throughflow groundwater to downstream areas	Could result in a flood risk around the vicinity of the site or at downstream locations
The proposed development may result in an increase groundwater risk to neighboring properties.	

The results of the site investigation have therefore been used below to review the remaining potential impacts, to assess the likelihood of them occurring and the scope for reasonable engineering mitigation.

Recorded groundwater table potentially extends above the base of proposed subsurface structure / proposed basement excavation is likely to extend below the local water table

The results of the investigation have indicated groundwater to be present within the Kempton Park Gravel at a level of about 3.50 m below ground level, which corresponds to a level of 4.70 m AOD. The current ground level at the site is 8.20 m AOD and the new basement extension is to increase the depth of the basement to a level of 5.10 m AOD. Therefore, the basement will not extend below the groundwater table and therefore will not interrupt or obstruct groundwater flow within the Kempton Park Gravel. It will therefore not have an impact on the local hydrogeology.

The development will increase the depth of the foundations with respect to the foundations of the neighbouring structures / the ground at the site has been previously worked

In view of the relatively small scale of the basement deepening and extension, it should be possible to restrict movements caused by the basement works, such that damage to neighbouring properties is limited to a maximum of Category 2 – Slight on the Burland Damage Classification, which is in accordance with London Borough of Richmond requirements.

The ground investigation has indicated the made ground at the site to extend to depths of between 1.30 m and 1.60 m. No evidence of worked ground has been identified below this depth and therefore there is not considered to be an impact.

The site is underlain by an aquifer and or permeable geology / infiltration methods are proposed as part of the site drainage strategy / the proposed development will impact the flow profile of groundwater to downstream areas / the proposed development may result in an increased groundwater risk to neighbouring properties

The proposed development for the site will include the use of permeable paving and other SuDS measures, which may result in a larger proportion of surface water entering the ground than currently takes place. It is understood that attenuation systems will be adopted to mitigate any potential impact on surface water inflows and run-off. As a result, there is not considered to be an increase to the risk of groundwater flooding to neighbouring sites, as the groundwater table is significantly below ground level and the SuDS will be designed to discharge water into the ground at a reasonable rate as to minimise the impact of the additional surface water. Therefore, the additional surface water discharge should also not result in a significant change to the flow profile of groundwater to downstream areas. The proposed basement is not considered to have the potential to have impact on the local hydrology.

8.2 BIA Conclusions

A Basement Impact Assessment has been carried out following the information and guidance published by the Borough of Richmond Upon Thames. It is concluded that the proposed development is unlikely to result in any specific hydrogeological, hydrological, land or slope stability issues. There is nothing about the proposed development that would fall outside of standard engineering practice and design, such that it is not considered to pose a risk to the immediate surrounding area. Therefore, the potential impacts identified by the BIA can be mitigated by appropriate design and standard construction practice.

Appendix

a. Field Work

Site Plan
Borehole Records

appendix a

Field Work

Site Plan
Borehole Records

appendix a

Geotechnical &
Environmental
Associates

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GEA

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Site Plan

Site 29-31 High Street, Kingston-upon-Thames KT1 4DA

Job Number J23151

Sheet 1 / 1

GEA

Geotechnical & Environmental Associates

Project				BOREHOLE No
29-31 High Street, Kingston Upon Thames, Surrey KT1 4DA				**BH1**
Job No: J23151	Date: 15-06-23 15-06-23	Ground Level (m OD)	Co-Ordinates ()	
Client: Elizabeth Frost		Engineer: Engineeria		Sheet: 1 of 1

SAMPLES & TESTS				STRATA				
Depth	Type No	Test Result	Water	Reduced Level	Legend	Depth (Thickness)	DESCRIPTION	Instrument / Backfill
						0.05	Made Ground (block paving)	
						0.10	Made Ground (sand over membrane)	
0.25	D1					(1.40) 1.50	Made Ground (dark grey and brown sandy gravelly clay with brick, ash and concrete fragments)	
0.50	D2							
0.75	D3							
1.00	D4							
1.20-1.65	D5	1,2/3,2,2,2 N60 = 10						
1.85	D6					(3.95) 5.45	Medium dense yellowish brown fine to coarse SAND with fine to coarse subangular to subrounded gravel	
2.00-2.45	D7	2,2/3,3,2,3 N60 = 12						
2.75	D8							
3.00-3.45	D9	2,3/3,2,3,2 N60 = 11						
3.75	D10							
4.00-4.45	D11	3,3/4,5,4,5 N60 = 19						
4.75	D12							
5.00-5.45	D13	3,4/4,5,5,6 N60 = 22						

Boring Progress and Water Observations

Depth	Date	Time	Casing Depth	Casing Dia. mm	Water Depth

GENERAL REMARKS

Groundwater monitoring standpipe installed to 4.50 m

All dimensions in metres Scale 1:62.5	Method/ Plant Used	Logged By

GEA

Geotechnical & Environmental Associates

Project				BOREHOLE No
29-31 High Street, Kingston Upon Thames, Surrey KT1 4DA				**BH2**
Job No: J23151	Date: 15-06-23 15-06-23	Ground Level (m OD)	Co-Ordinates ()	
Client: Elizabeth Frost		Engineer: Engineeria		Sheet: 1 of 1

SAMPLES & TESTS				STRATA				
Depth	Type No	Test Result	Water	Reduced Level	Legend	Depth (Thickness)	DESCRIPTION	Instrument / Backfill
						0.05	Made Ground (block paving)	
						0.10	Made Ground (sand)	
0.25	D1						Made Ground (dark brown sandy gravelly clay with brick, ash and concrete fragments)	
0.50	D2							
0.75	D3					(1.50)		
1.00	D4							
1.20-1.65	D5	1,2/1,1,2,2 N60 = 6				1.60		
1.85	D6						Medium dense yellowish brown fine to coarse SAND with fine to coarse subangular to subrounded gravel	
2.00-2.45	D7	2,2/2,3,3,3 N60 = 12						
2.75	D8							
3.00-3.45	D9	2,3/3,4,4,3,4 N60 = 15						
3.75	D10							
4.00-4.45	D11	2,2/3,3,3,4 N60 = 14				(5.50)		
4.75	D12							
5.00-5.45	D13	3,3/3,4,4,5 N60 = 17						
6.00	D14							
6.50-6.95	D15	3,4/4,4,4,5 N60 = 18				7.10		
7.50	D16					(1.35)	Stiff fissured bluish grey CLAY	
8.00-8.45	D17	3,4/5,5,6,6 N60 = 24				8.45		

Boring Progress and Water Observations					
Depth	Date	Time	Casing Depth	Casing Dia. mm	Water Depth

GENERAL REMARKS

Groundwater monitoring standpipe installed to 4.00 m

All dimensions in metres Scale 1:62.5	Method/ Plant Used	Logged By

GEA

Geotechnical & Environmental Associates

Project				BOREHOLE No
29-31 High Street, Kingston Upon Thames, Surrey KT1 4DA				**BH3**
Job No: J23151	Date: 15-06-23 15-06-23	Ground Level (m OD)	Co-Ordinates ()	
Client: Elizabeth Frost		Engineer: Engineeria		Sheet: 1 of 1

SAMPLES & TESTS

Depth	Type No	Test Result
0.25	D1	
0.50	D2	
0.75	D3	
1.00	D4	
1.20-1.65	D5	
1.85	D6	
2.00-2.45	D7	
2.75	D8	
3.00-3.45	D9	
3.75	D10	
4.00-4.45	D11	
4.75	D12	
5.00-5.45	D13	

STRATA

Water	Reduced Level	Depth (Thickness)	DESCRIPTION	Instrument / Backfill
		0.05	Made Ground (block paving)	
		0.10	Made Ground (sand)	
		0.50	Made Ground (dark grey to black gravelly clayey sand with brick and concrete fragments)	
		(0.80) 1.30	Made Ground (dark brown and dark grey sandy gravelly clay with fine brick and ash fragments)	
		(1.70) 3.00	Medium dense yellowish brown fine to coarse SAND and fine to coarse subangular to subrounded GRAVEL	
		(2.45) 5.45	Medium dense brown silty slightly clayey SAND with fine to coarse subangular to subrounded gravel	

Boring Progress and Water Observations

Depth	Date	Time	Casing Depth	Casing Dia. mm	Water Depth

GENERAL REMARKS

Groundwater monitoring standpipe installed to 4.00 m

All dimensions in metres Scale 1:62.5	Method/ Plant Used	Logged By

Geotechnical &
Environmental
Associates

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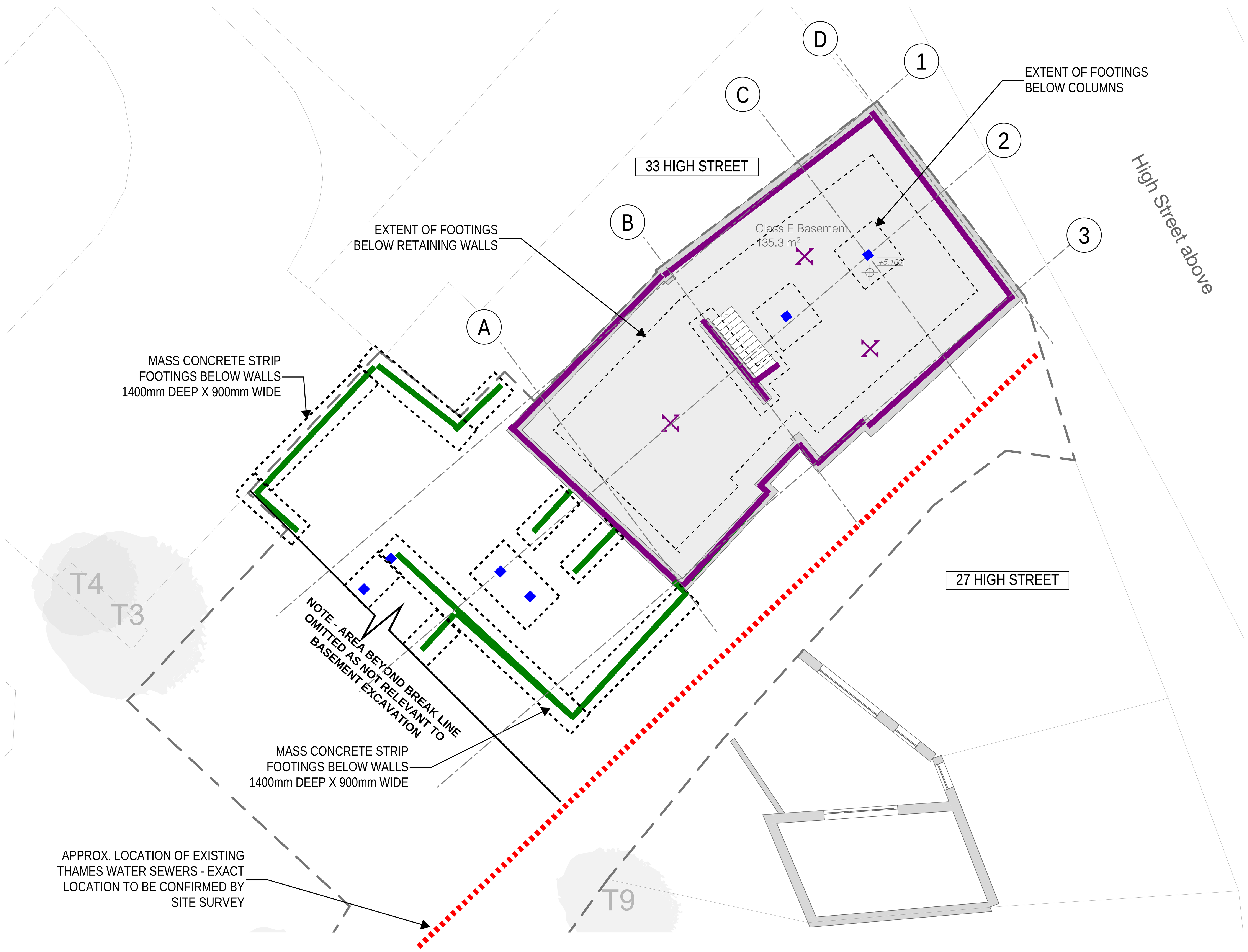
mail @ gea-ltd.co.uk

www.gea-ltd.co.uk

Appendix B - Preliminary Structural Scheme

1. THIS DRAWING IS TO BE READ IN CONJUNCTION WITH ALL ENGINEER'S AND ARCHITECT'S DRAWINGS, SPECIFICATIONS AND RISK REGISTERS.
2. DO NOT SCALE FROM THIS DRAWING. USE ONLY DIMENSIONS AS INDICATED. CHECK ALL SITE DIMENSIONS PRIOR TO PLACING ANY ORDER OR FABRICATION. WHERE A CONFLICT OF INFORMATION EXISTS SEEK CONFIRMATION FROM CONSULTANTS PRIOR TO PROCEEDING FURTHER WITH THE WORKS
3. TEMPORARY STABILITY OF THE EXISTING STRUCTURE AND ANY NEWLY CONSTRUCTED ELEMENTS OF PERMANENT WORKS DURING CONSTRUCTION IS SOLELY CONTRACTOR'S RESPONSIBILITY
4. ONLY DRAWINGS AND SPECIFICATIONS ISSUED FOR **CONSTRUCTION** CAN BE USED FOR THE WORKS. IT IS THE CONTRACTOR'S RESPONSIBILITY TO SEEK THE INFORMATION FROM CONSULTANTS.
5. ALL PROPRIETARY ITEMS TO BE INSTALLED STRICTLY IN ACCORDANCE WITH MANUFACTURER'S REQUIREMENTS AND SPECIFICATIONS
6. ALL WATERPROOFING SUCH AS TANKING DETAILS, DAMP PROOF MEMBRANES, DAMP PROOF COURSES, CAVITY TRAYS ETC. ARE TO BE INSTALLED AS PER ARCHITECT'S DETAILS



PROPOSED BASEMENT PLAN

EXTRACT FROM ARCHITECTURAL DRAWING TP(10)20

MINOR OPENINGS FOR WINDOWS, DOORS ETC. OMITTED FOR CLARITY, MAKE SUITABLE ALLOWANCE FOR LINTELS

N.T.S.

PROPOSED MEMBER SCHEDULE

- STEEL COLUMN - ALLOW 100x100x8 SHS S355
- STEEL COLUMN FORMING PART OF MOMENT RESISTING FRAME - ALLOW 203x203x71 UC
- STEEL BEAM - ALLOW 203x203x 60 UC
- C32/40 250mm THK SUSPENDED SLAB WITH COLLAPSIBLE VOID FORMER BENEATH.
- C32/40 250mm THK SUSPENDED SLAB
- DENOTES 300mm THK REINFORCED CONCRETE BASEMENT WALL
- DENOTES MASONRY CAVITY WALL FORMED FROM 140mm THICK BLOCKWORK INTERNAL LEAF, CAVITY (SIZE T.B.C. BY ARCHITECT), AND 100mm THK EXTERNAL LEAF BRICKWORK
- DENOTES LOAD BEARING MASONRY WALL (ASSUMED 140mm THK BLOCKWORK, BUILD UP T.B.C.)
- TIMBER FLOOR JOISTS - ALLOW 225x75 C24 JOISTS WITH 18mm THK PLYWOOD GLUED AND SCREWED TO JOISTS

NOTES

MOVEMENT JOINTS TO BE PROVIDED FOR THE MASONRY WALL. MAXIMUM DISTANCE BEETWEN MOVEMENT JOINTS IN BLOCKWORK TO BE 7m AND BRICKWORK TO BE 12m. THIS IS TO BE CONFIRMED BY THE BRICK AND BLOCK MANUFACTURER

BLOCKWORK TO HAVE MINIMUM COMPRESSIVE STRENGTH: 7.3 N/mm^2

MORTAR COMPRESSIVE STRENGTH CLASS MIN. M6 FOR EXTERNAL WALLS

ALL STEEL MEMBERS TO BE GRADE S355

ALL REINFORCED CONCRETE TO BE GRADE C32/40

ALL MASS CONCRETE TO BE GRADE C25/30

NOTE - ALL SIZES ARE FOR INITIAL DESIGN PURPOSES ONLY BASED ON THE INFORMATION PROVIDED, AND HAVE NOT BEEN CO-ORDINATED WITH THE ARCHITECTURAL LAYOUTS. MAKE A SUITABLE ALLOWANCE FOR FURTHER SCHEME DEVELOPMENT AT A LATER PROJECT STAGE

rev	date	description	by	checked
P01	2023.07.19	ISSUED FOR INFORMATION	MT	MW

PROJECT TITLE: 29-31 HIGH STREET, HAMPTON WICK

CLIENT: MR. & MRS. FROST

PROJECT No: E0811 DRAWN: MT CHECKED: MW

DRAWING TITLE: PROPOSED BASEMENT PLAN

DRAWING No: E0811-SK-9000

STATUS DESCRIPTION: SUITABLE FOR INFORMATION

REV: P01 SCALE: As Indicated @A1

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1 THIS DRAWING IS TO BE READ IN CONJUNCTION WITH ALL ENGINEER'S AND ARCHITECT'S DRAWINGS, SPECIFICATIONS AND RISK REGISTERS.
2. DO NOT SCALE FROM THIS DRAWING. USE ONLY DIMENSIONS AS INDICATED. CHECK ALL SITE DIMENSIONS PRIOR TO PLACING ANY ORDER OR FABRICATION. WHERE A CONFLICT OF INFORMATION EXISTS SEEK CONFIRMATION FROM CONSULTANTS PRIOR TO PROCEEDING FURTHER WITH THE WORKS
3. TEMPORARY STABILITY OF THE EXISTING STRUCTURE AND ANY NEWLY CONSTRUCTED ELEMENTS OF PERMANENT WORKS DURING CONSTRUCTION IS SOLELY CONTRACTOR'S RESPONSIBILITY
4. ONLY DRAWINGS AND SPECIFICATIONS ISSUED FOR **CONSTRUCTION** CAN BE USED FOR THE WORKS. IT IS THE CONTRACTOR'S RESPONSIBILITY TO SEEK THE INFORMATION FROM CONSULTANTS.
5. ALL PROPRIETARY ITEMS TO BE INSTALLED STRICTLY IN ACCORDANCE WITH MANUFACTURER'S REQUIREMENTS AND SPECIFICATIONS
6. ALL WATERPROOFING SUCH AS TANKING DETAILS, DAMP PROOF MEMBRANES, DAMP PROOF COURSES, CAVITY TRAYS ETC. ARE TO BE INSTALLED AS PER ARCHITECT'S DETAILS

PROPOSED GROUND FLOOR PLAN

EXTRACT FROM ARCHITECTURAL DRAWING TP(10)21
MINOR OPENINGS FOR WINDOWS, DOORS ETC. OMITTED FOR CLARITY, MAKE SUITABLE ALLOWANCE FOR LINTELS
N.T.S.

PROPOSED MEMBER SCHEDULE

- STEEL COLUMN - ALLOW 100x100x8 SHS S355
- STEEL COLUMN FORMING PART OF MOMENT RESISTING FRAME - ALLOW 203x203x71 UC
- STEEL BEAM - ALLOW 203x203x60 UC
- C32/40 250mm THK SUSPENDED SLAB WITH COLLAPSIBLE VOID FORMER BENEATH.
- C32/40 250mm THK SUSPENDED SLAB
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- DENOTES LOAD BEARING MASONRY WALL (ASSUMED 140mm THK BLOCKWORK, BUILD UP T.B.C.)
- TIMBER FLOOR JOISTS - ALLOW 225x75 C24 JOISTS WITH 18mm THK PLYWOOD GLUED AND SCREWED TO JOISTS

NOTES

MOVEMENT JOINTS TO BE PROVIDED FOR THE MASONRY WALL. MAXIMUM DISTANCE BEETWEN MOVEMENT JOINTS IN BLOCKWORK TO BE 7m AND BRICKWORK TO BE 12m. THIS IS TO BE CONFIRMED BY THE BRICK AND BLOCK MANUFACTURER

BLOCKWORK TO HAVE MINIMUM COMPRESSIVE STRENGTH: 7.3 N/mm²

MORTAR COMPRESSIVE STRENGTH CLASS MIN. M6 FOR EXTERNAL WALLS

ALL STEEL MEMBERS TO BE GRADE S355

ALL REINFORCED CONCRETE TO BE GRADE C32/40

ALL MASS CONCRETE TO BE GRADE C25/30

NOTE - ALL SIZES ARE FOR INITIAL DESIGN PURPOSES ONLY BASED ON THE INFORMATION PROVIDED, AND HAVE NOT BEEN CO-ORDINATED WITH THE ARCHITECTURAL LAYOUTS. MAKE A SUITABLE ALLOWANCE FOR FURTHER SCHEME DEVELOPMENT AT A LATER PROJECT STAGE

rev	date	description	by	checked
P01	2023.07.19	ISSUED FOR INFORMATION	MT	MW

PROJECT TITLE: 29-31 HIGH STREET, HAMPTON WICK
CLIENT: MR. & MRS. FROST
PROJECT No: **E0811** DRAWN: **MT** CHECKED: **MW**
DRAWING TITLE: PROPOSED GROUND FLOOR PLAN
DRAWING No: E0811-SK-9001
STATUS DESCRIPTION: SUITABLE FOR INFORMATION
REV: **P01** SCALE: **As Indicated @A1**

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PROPOSED MEMBER SCHEDULE

- STEEL COLUMN - ALLOW 100x100x8 SHS S355
- STEEL COLUMN FORMING PART OF MOMENT RESISTING FRAME - ALLOW 203x203x71 UC
- STEEL BEAM - ALLOW 203x203x60 UC
- C32/40 200mm THK SUSPENDED SLAB WITH COLLAPSIBLE VOID FORMER BENEATH.
- C32/40 200mm THK SUSPENDED SLAB
- DENOTES 300mm THK REINFORCED CONCRETE BASEMENT WALL
- DENOTES MASONRY CAVITY WALL FORMED FROM 140mm THICK BLOCKWORK INTERNAL LEAF, CAVITY (SIZE T.B.C. BY ARCHITECT), AND 100mm THK EXTERNAL LEAF BRICKWORK
- DENOTES LOAD BEARING MASONRY WALL (ASSUMED 140mm THK BLOCKWORK, BUILD UP T.B.C.)
- TIMBER FLOOR JOISTS - ALLOW 225x75 C24 JOISTS WITH 18mm THK PLYWOOD GLUED AND SCREWED TO JOISTS

NOTES

MOVEMENT JOINTS TO BE PROVIDED FOR THE MASONRY WALL. MAXIMUM DISTANCE BEETWEN MOVEMENT JOINTS IN BLOCKWORK TO BE 7m AND BRICKWORK TO BE 12m. THIS IS TO BE CONFIRMED BY THE BRICK AND BLOCK MANUFACTURER

BLOCKWORK TO HAVE MINIMUM COMPRESSIVE STRENGTH: 7.3 N/mm^2

MORTAR COMPRESSIVE STRENGTH CLASS MIN. M6 FOR EXTERNAL WALLS

ALL STEEL MEMBERS TO BE GRADE S355

ALL REINFORCED CONCRETE TO BE GRADE C32/40

ALL MASS CONCRETE TO BE GRADE C25/30

NOTE - ALL SIZES ARE FOR INITIAL DESIGN PURPOSES ONLY BASED ON THE INFORMATION PROVIDED, AND HAVE NOT BEEN CO-ORDINATED WITH THE ARCHITECTURAL LAYOUTS. MAKE A SUITABLE ALLOWANCE FOR FURTHER SCHEME DEVELOPMENT AT A LATER PROJECT STAGE

PROPOSED FIRST FLOOR PLAN

EXTRACT FROM ARCHITECTURAL DRAWING TP(10)22

MINOR OPENINGS FOR WINDOWS, DOORS ETC. OMITTED FOR CLARITY, MAKE SUITABLE ALLOWANCE FOR LINTELS

N.T.S.

rev	date	description	by	checked
P01	2023.07.19	ISSUED FOR INFORMATION	MT	MW

PROJECT TITLE: 29-31 HIGH STREET, HAMPTON WICK

CLIENT: MR. & MRS. FROST

PROJECT No: **E0811**

DRAWN: **MT**

CHECKED: **MW**

DRAWING TITLE: PROPOSED FIRST FLOOR PLAN

DRAWING No: E0811-SK-9002

STATUS DESCRIPTION: SUITABLE FOR INFORMATION

REV: **P01**

SCALE: **As Indicated @A1**

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PROPOSED MEMBER SCHEDULE

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- STEEL COLUMN FORMING PART OF MOMENT RESISTING FRAME - ALLOW 203x203x71 UC
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- C32/40 200mm THK SUSPENDED SLAB
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- TIMBER FLOOR JOISTS - ALLOW 225x75 C24 JOISTS WITH 18mm THK PLYWOOD GLUED AND SCREWED TO JOISTS

NOTES

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PROPOSED SECOND FLOOR PLAN

EXTRACT FROM ARCHITECTURAL DRAWING TP(10)23

MINOR OPENINGS FOR WINDOWS, DOORS ETC. OMITTED FOR CLARITY, MAKE SUITABLE ALLOWANCE FOR LINTELS

N.T.S.

rev	date	description	by	checked
P01	2023.07.19	ISSUED FOR INFORMATION	MT	MW

PROJECT TITLE: 29-31 HIGH STREET, HAMPTON WICK

DRAWING TITLE: PROPOSED SECOND FLOOR PLAN

CLIENT: MR. & MRS. FROST

DRAWING No: E0811-SK-9003

STATUS DESCRIPTION: SUITABLE FOR INFORMATION

PROJECT No: **E0811** DRAWN: **MT** CHECKED: **MW** REV: **P01** SCALE: **As Indicated @A1**

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PROPOSED ROOF STRUCTURE

EXTRACT FROM ARCHITECTURAL DRAWING TP(10)24

N.T.S.

PROPOSED MEMBER SCHEDULE

- STEEL COLUMN - ALLOW 100x100x8 SHS S355
- STEEL COLUMN FORMING PART OF MOMENT RESISTING FRAME - ALLOW 203x203x71 UC
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- DENOTES LOAD BEARING MASONRY WALL (ASSUMED 140mm THK BLOCKWORK, BUILD UP T.B.C.)
- TIMBER RAFTERS - ALLOW 225x50 C24 RAFTERS
- TIMBER ROOF JOISTS - ALLOW 225x75 C24 JOISTS WITH 18mm THK PLYWOOD GLUED AND SCREWED TO JOISTS

NOTES

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BLOCKWORK TO HAVE MINIMUM COMPRESSIVE STRENGTH: 7.3 N/mm^2

MORTAR COMPRESSIVE STRENGTH CLASS MIN. M6 FOR EXTERNAL WALLS

ALL STEEL MEMBERS TO BE GRADE S355

ALL REINFORCED CONCRETE TO BE GRADE C32/40

ALL MASS CONCRETE TO BE GRADE C25/30

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rev	date	description	by	checked
P01	2023.07.19	ISSUED FOR INFORMATION	MT	MW

PROJECT TITLE: 29-31 HIGH STREET, HAMPTON WICK

CLIENT: MR. & MRS. FROST

PROJECT No: **E0811** DRAWN: **MT** CHECKED: **MW**

DRAWING TITLE: PROPOSED ROOF STRUCTURE

DRAWING No: E0811-SK-9004

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Appendix C - Preliminary Structural Calculations

Calculation Sheet

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PROJECT TITLE:	29-31 HIGH ST. HAMPTON WICK
PROJECT NUMBER:	E0871
SHEET NUMBER:	
DATE:	19/07/2023
BY:	MW
CHECKED BY:	
REV:	
TITLE:	LOAD TAKE DOWN FOR RETAINING WALL (GL 3)

LOADING (1F – ROOF)

AREA LOADS: DEAD = 1.5kN/m² (TYPICAL)
LIVE = 2.5kN/m²

OVERALL WIDTH OF BUILDING ≈ 9m.

∴ DEAD LOADING = $\frac{9m}{2 \times 2 \text{ SPANS}} \times$ 3 FLOORS × 1.5kN/m² = 10kN/m

LIVE LOADING = " × " × 2.5kN/m² = 17kN/m

LOADING FROM GF SLAB.

DEAD LOAD = 0.25m × 25kN/m³ = 6.3kN/m²
+ 1.5kN/m²
= 7.8kN/m²

∴ GF DEAD = 7.8kN/m² × $\frac{9m}{2 \times \text{SPAN}}$ = 18 kN/m
GF LIVE = 4.0kN/m² × " = 9 kN/m.
(SHOP LOADING)

∴ TOTAL LOADING ; DEAD = 10kN/m + 18kN/m = 28kN/m
LIVE = 17kN/m + 9kN/m = 26kN/m

RETAINING WALL ANALYSIS

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

Tedds calculation version 2.9.17

Retaining wall details

Stem type	Propped cantilever pinned at the base
Stem height	h_{stem} = **3250** mm
Prop height	h_{prop} = **3100** mm
Stem thickness	t_{stem} = **300** mm
Angle to rear face of stem	α = **90** deg
Stem density	γ_{stem} = **25** kN/m^3
Toe length	l_{toe} = **1200** mm
Base thickness	t_{base} = **350** mm
Base density	γ_{base} = **25** kN/m^3
Height of retained soil	h_{ret} = **3250** mm
Angle of soil surface	β = **0** deg
Depth of cover	d_{cover} = **0** mm
Height of water	h_{water} = **2250** mm
Water density	γ_w = **9.8** kN/m^3

Retained soil properties

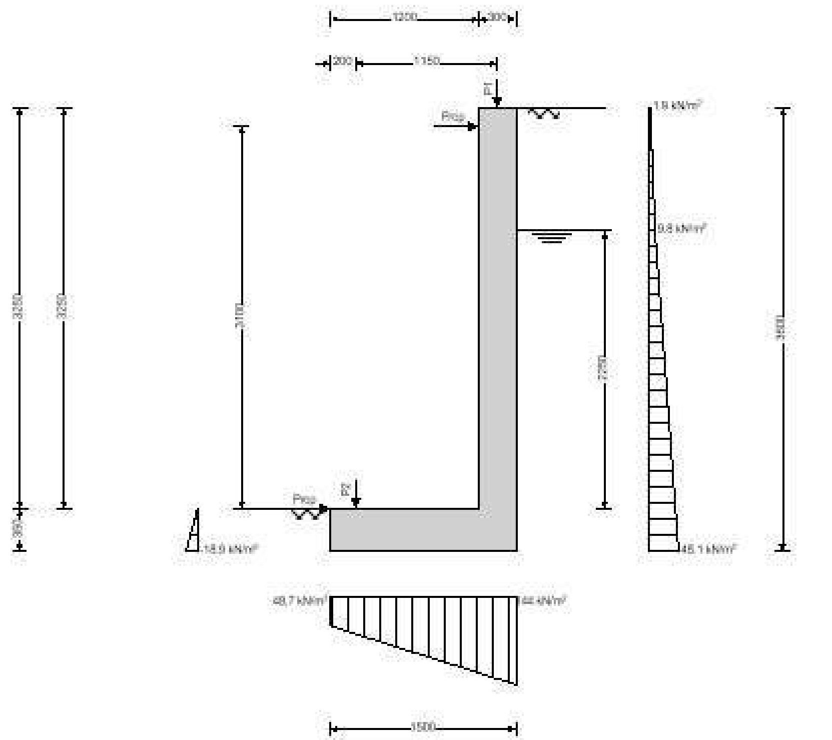
Soil type	Medium dense well graded sand
Moist density	γ_{mr} = **21** kN/m^3
Saturated density	γ_{sr} = **23** kN/m^3
Characteristic effective shear resistance angle	$\phi'_{r.k}$ = **27** deg
Characteristic wall friction angle	$\delta_{r.k}$ = **13.5** deg

Base soil properties

Soil type	Medium dense well graded sand
Soil density	γ_b = **18** kN/m^3
Characteristic effective shear resistance angle	$\phi'_{b.k}$ = **30** deg
Characteristic wall friction angle	$\delta_{b.k}$ = **15** deg
Characteristic base friction angle	$\delta_{bb.k}$ = **20** deg
Presumed bearing capacity	$P_{bearing}$ = **150** kN/m^2

Loading details

Variable surcharge load	$Surcharge_Q$ = **5** kN/m^2
Vertical line load at 1350 mm	P_{G1} = **28** kN/m
	P_{Q1} = **26** kN/m
Vertical line load at 200 mm	P_{G2} = **35** kN/m
	P_{Q2} = **18** kN/m



General arrangement - sketch pressures relate to bearing check

Calculate retaining wall geometry

Base length $l_{base} = l_{toe} + t_{stem} = \mathbf{1500}$ mm

Saturated soil height $h_{sat} = h_{water} + d_{cover} = \mathbf{2250}$ mm

Moist soil height $h_{moist} = h_{ret} - h_{water} = \mathbf{1000}$ mm

Length of surcharge load $l_{sur} = l_{heel} = \mathbf{0}$ mm

- Distance to vertical component $x_{sur\_v} = l_{base} - l_{heel} / 2 = \mathbf{1500}$ mm

Effective height of wall $h_{eff} = h_{base} + d_{cover} + h_{ret} = \mathbf{3600}$ mm

- Distance to horizontal component $x_{sur\_h} = h_{eff} / 2 = \mathbf{1800}$ mm

Area of wall stem $A_{stem} = h_{stem} \times t_{stem} = \mathbf{0.975}$ m^2

- Distance to vertical component $x_{stem} = l_{toe} + t_{stem} / 2 = \mathbf{1350}$ mm

Area of wall base $A_{base} = l_{base} \times t_{base} = \mathbf{0.525}$ m^2

- Distance to vertical component $x_{base} = l_{base} / 2 = \mathbf{750}$ mm

Using Rankine theory

Active pressure coefficient $K_A = (1 - \sin(\phi'_{r.k})) / (1 + \sin(\phi'_{r.k})) = \mathbf{0.376}$

Passive pressure coefficient $K_P = (1 + \sin(\phi'_{b.k})) / (1 - \sin(\phi'_{b.k})) = \mathbf{3.000}$

Bearing pressure check

Vertical forces on wall

Wall stem $F_{stem} = A_{stem} \times \gamma_{stem} = \mathbf{24.4}$ kN/m

Wall base $F_{base} = A_{base} \times \gamma_{base} = \mathbf{13.1}$ kN/m

Line loads $F_{P\_v} = P_{G1} + P_{Q1} + P_{G2} + P_{Q2} = \mathbf{107}$ kN/m

Total $F_{total\_v} = F_{stem} + F_{base} + F_{P\_v} + F_{water\_v} =$ **144.5** kN/m

Horizontal forces on wall

Surcharge load $F_{sur\_h} = K_A \times Surcharge_Q \times h_{eff} =$ **6.8** kN/m

Saturated retained soil $F_{sat\_h} = K_A \times (\gamma_{sr} - \gamma_w) \times (h_{sat} + h_{base})^2 / 2 =$ **16.7** kN/m

Water $F_{water\_h} = \gamma_w \times (h_{water} + d_{cover} + h_{base})^2 / 2 =$ **33.2** kN/m

Moist retained soil $F_{moist\_h} = K_A \times \gamma_{mr} \times ((h_{eff} - h_{sat} - h_{base})^2 / 2 + (h_{eff} - h_{sat} - h_{base}) \times (h_{sat} + h_{base}))$ = **24.4** kN/m

Base soil $F_{pass\_h} = -K_P \times \gamma_b \times (d_{cover} + h_{base})^2 / 2 =$ **-3.3** kN/m

Total $F_{total\_h} = F_{sur\_h} + F_{sat\_h} + F_{water\_h} + F_{moist\_h} + F_{pass\_h} =$ **77.8** kN/m

Moments on wall

Wall stem $M_{stem} = F_{stem} \times x_{stem} =$ **32.9** kNm/m

Wall base $M_{base} = F_{base} \times x_{base} =$ **9.8** kNm/m

Line loads $M_P = (P_{G1} + P_{Q1}) \times p_1 + (P_{G2} + P_{Q2}) \times p_2 =$ **83.5** kNm/m

Total $M_{total} = M_{stem} + M_{base} + M_{sur} + M_P =$ **126.3** kNm/m

Check bearing pressure

Distance to reaction $\bar{x} = M_{total} / F_{total\_v} =$ **874** mm

Eccentricity of reaction $e = \bar{x} - l_{base} / 2 =$ **124** mm

Loaded length of base $l_{load} = l_{base} =$ **1500** mm

Bearing pressure at toe $q_{toe} = F_{total\_v} / l_{base} \times (1 - 6 \times e / l_{base}) =$ **48.7** kN/m²

Bearing pressure at heel $q_{heel} = F_{total\_v} / l_{base} \times (1 + 6 \times e / l_{base}) =$ **144** kN/m²

Factor of safety $FoS_{bp} = P_{bearing} / \max(q_{toe}, q_{heel}) =$ **1.042**

PASS - Allowable bearing pressure exceeds maximum applied bearing pressure

<u>RETAINING WALL DESIGN</u>

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

Tedds calculation version 2.9.17

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

Concrete strength class C32/40

Characteristic compressive cylinder strength $f_{ck} =$ **32** N/mm²

Characteristic compressive cube strength $f_{ck,cube} =$ **40** N/mm²

Mean value of compressive cylinder strength $f_{cm} = f_{ck} + 8$ N/mm² = **40** N/mm²

Mean value of axial tensile strength $f_{ctm} = 0.3\ \text{N/mm}^2 \times (f_{ck} / 1\ \text{N/mm}^2)^{2/3} =$ **3.0** N/mm²

5% fractile of axial tensile strength $f_{ctk,0.05} = 0.7 \times f_{ctm} =$ **2.1** N/mm²

Secant modulus of elasticity of concrete $E_{cm} = 22\ \text{kN/mm}^2 \times (f_{cm} / 10\ \text{N/mm}^2)^{0.3} =$ **33346** N/mm²

Partial factor for concrete - Table 2.1N $\gamma_C =$ **1.50**

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

Design compressive concrete strength - exp.3.15 $f_{cd} = \alpha_{cc} \times f_{ck} / \gamma_C =$ **18.1** N/mm²

Maximum aggregate size $h_{agg} =$ **20** mm

Ultimate strain - Table 3.1 $\varepsilon_{cu2} =$ **0.0035**

Shortening strain - Table 3.1 $\varepsilon_{cu3} =$ **0.0035**

Effective compression zone height factor $\lambda =$ **0.80**

Effective strength factor $\eta =$ **1.00**

Bending coefficient k_1 — K_1 = **0.40**

Bending coefficient k_2 — $K_2 = 1.00 \times (0.6 + 0.0014/\varepsilon_{cu2})$ = **1.00**

Bending coefficient k_3 — K_3 =**0.40**

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

Reinforcement details

Characteristic yield strength of reinforcement — f_{yk} = **500** N/mm^2

Modulus of elasticity of reinforcement — E_s = **200000** N/mm^2

Partial factor for reinforcing steel - Table 2.1N — γ_S = **1.15**

Design yield strength of reinforcement — $f_{yd} = f_{yk} / \gamma_S$ = **435** N/mm^2

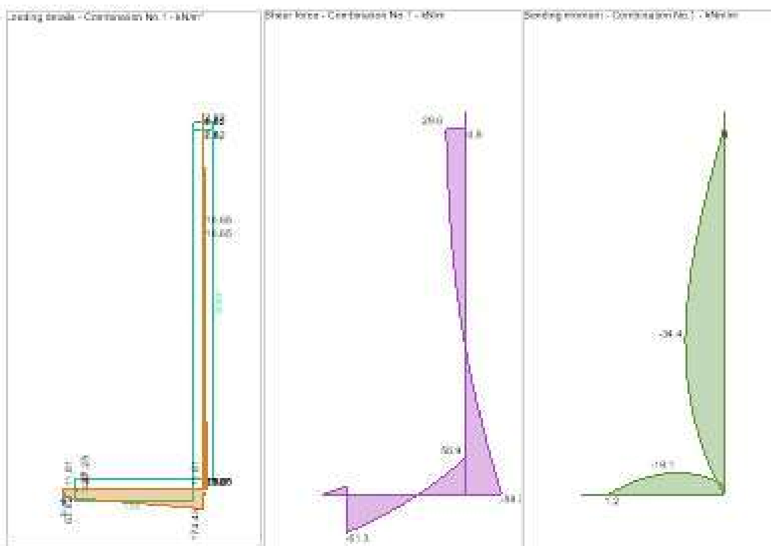
Cover to reinforcement

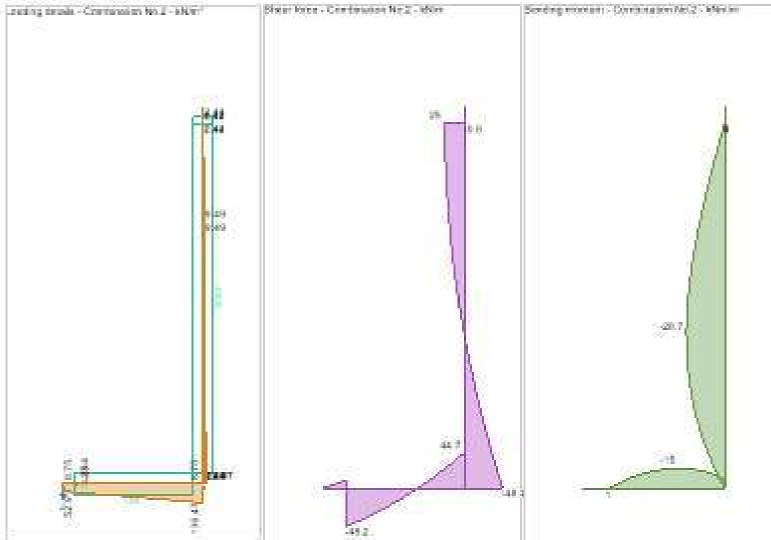
Front face of stem — c_{sf} = **30** mm

Rear face of stem — c_{sr} = **55** mm

Top face of base — c_{bt} = **30** mm

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





Check stem design at 1119 mm

Depth of section | h = **300** mm

Rectangular section in flexure - Section 6.1

Design bending moment combination 1	M = **34.4** kNm/m
Depth to tension reinforcement	$d = h - c_{sf} - \phi_{sx} - \phi_{sfM} / 2$ = **254** mm
	$K = M / (d^2 \times f_{ck})$ = **0.017**
	$K' = (2 \times \eta \times \alpha_{cc}/\gamma_C)\times(1 - \lambda \times (\delta - K_1)/(2 \times K_2))\times(\lambda \times (\delta - K_1)/(2 \times K_2))$
	K' = **0.207**
	K' > K - No compression reinforcement is required
Lever arm	$z = \min(0.5 + 0.5 \times (1 - 2 \times K / (\eta \times \alpha_{cc} / \gamma_C))^{0.5}, 0.95) \times d$ = **241** mm
Depth of neutral axis	$x = 2.5 \times (d - z)$ = **32** mm
Area of tension reinforcement required	$A_{sfM.req} = M / (f_{yd} \times z)$ = **328** mm²/m
Tension reinforcement provided	12 dia.bars @ 200 c/c
Area of tension reinforcement provided	$A_{sfM.prov} = \pi \times \phi_{sfM}^2 / (4 \times s_{sfM})$ = **565** mm²/m
Minimum area of reinforcement - exp.9.1N	$A_{sfM.min} = \max(0.26 \times f_{ctm} / f_{yk}, 0.0013) \times d$ = **399** mm²/m
Maximum area of reinforcement - cl.9.2.1.1(3)	$A_{sfM.max} = 0.04 \times h$ = **12000** mm²/m
	$\max(A_{sfM.req}, A_{sfM.min}) / A_{sfM.prov}$ = **0.706**

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

Library item: Rectangular single output

Deflection control - Section 7.4

Reference reinforcement ratio	$\rho_0 = \sqrt{(f_{ck} / 1\ \text{N/mm}^2)} / 1000$ = **0.006**
Required tension reinforcement ratio	$\rho = A_{sfM.req} / d$ = **0.001**
Required compression reinforcement ratio	$\rho' = A_{sfM.2.req} / d_2$ = **0.000**
Structural system factor - Table 7.4N	K_b = **1**
Reinforcement factor - exp.7.17	$K_s = \min(500\ \text{N/mm}^2 / (f_{yk} \times A_{sfM.req} / A_{sfM.prov}), 1.5)$ = **1.5**
Limiting span to depth ratio - exp.7.16.a	$\min(K_s \times K_b \times [11 + 1.5 \times \sqrt{(f_{ck} / 1\ \text{N/mm}^2)} \times \rho_0 / \rho + 3.2 \times \sqrt{(f_{ck} / 1\ \text{N/mm}^2)} \times (\rho_0 / \rho - 1)^{3/2}], 40 \times K_b)$ = **40**
Actual span to depth ratio	h_{prop} / d = **12.2**

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

Crack control - Section 7.3

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

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

Serviceability bending moment — M_{sls} = **24.4** kNm/m

Tensile stress in reinforcement — $\sigma_s = M_{sls} / (A_{sfM.prov} \times z)$ = **178.6** N/mm^2

Load duration — Long term

Load duration factor — k_t = **0.4**

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

$A_{c.eff}$ = **89417** mm^2/m

Mean value of concrete tensile strength — $f_{ct.eff} = f_{ctm}$ = **3.0** N/mm^2

Reinforcement ratio — $\rho_{p.eff} = A_{sfM.prov} / A_{c.eff}$ = **0.006**

Modular ratio — $\alpha_e = E_s / E_{cm}$ = **5.998**

Bond property coefficient — k_1 = **0.8**

Strain distribution coefficient — k_2 = **0.5**

k_3 = **3.4**

k_4 = **0.425**

Maximum crack spacing - exp.7.11 — $s_{r.max} = k_3 \times c_{sf} + k_1 \times k_2 \times k_4 \times \phi_{sfM} / \rho_{p.eff}$ = **425** mm

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

w_k = **0.227** mm

w_k / w_{max} = **0.758**

PASS - Maximum crack width is less than limiting crack width

Check stem design at base of stem

Depth of section — h = **300** mm

Rectangular section in shear - Section 6.2

Design shear force — V = **58.7** kN/m

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

$k = \min(1 + \sqrt{(200 \text{ mm} / d)}, 2)$ = **1.887**

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

$v_{min} = 0.035 \text{ N}^{1/2}/\text{mm} \times k^{3/2} \times f_{ck}^{0.5}$ = **0.513** N/mm^2

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

$V_{Rd.c}$ = **130.4** kN/m

$V / V_{Rd.c}$ = **0.451**

PASS - Design shear resistance exceeds design shear force

Check stem design at prop

Depth of section — h = **300** mm

Rectangular section in flexure - Section 6.1

Design bending moment combination 1 — M = **0** kNm/m

Depth to tension reinforcement — $d = h - c_{sr} - \phi_{sr1} / 2$ = **239** mm

$K = M / (d^2 \times f_{ck})$ = **0.000**

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

K' = **0.207**

K' > K - No compression reinforcement is required

Lever arm	$z = \min(0.5 + 0.5 \times (1 - 2 \times K / (\eta \times \alpha_{cc} / \gamma_C))^{0.5}, 0.95) \times d = \mathbf{227}$ mm
Depth of neutral axis	$x = 2.5 \times (d - z) = \mathbf{30}$ mm
Area of tension reinforcement required	$A_{sr1.req} = M / (f_{yd} \times z) = \mathbf{0}$ mm^2/m
Tension reinforcement provided	12 dia.bars @ 200 c/c
Area of tension reinforcement provided	$A_{sr1.prov} = \pi \times \phi_{sr1}^2 / (4 \times s_{sr1}) = \mathbf{565}$ mm^2/m
Minimum area of reinforcement - exp.9.1N	$A_{sr1.min} = \max(0.26 \times f_{ctm} / f_{yk}, 0.0013) \times d = \mathbf{376}$ mm^2/m
Maximum area of reinforcement - cl.9.2.1.1(3)	$A_{sr1.max} = 0.04 \times h = \mathbf{12000}$ mm^2/m
	$\max(A_{sr1.req}, A_{sr1.min}) / A_{sr1.prov} = \mathbf{0.665}$

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

Library item: Rectangular single output

Deflection control - Section 7.4

Reference reinforcement ratio	$\rho_0 = \sqrt{(f_{ck} / 1\ \text{N/mm}^2)} / 1000 = \mathbf{0.006}$
Required tension reinforcement ratio	$\rho = A_{sr1.req} / d = \mathbf{0.000}$
Required compression reinforcement ratio	$\rho' = A_{sr1.2.req} / d_2 = \mathbf{0.000}$
Structural system factor - Table 7.4N	$K_b = \mathbf{0.4}$
Reinforcement factor - exp.7.17	$K_s = \min(500\ \text{N/mm}^2 / (f_{yk} \times A_{sr1.req} / A_{sr1.prov}), 1.5) = \mathbf{1.5}$
Limiting span to depth ratio - exp.7.16.a	$\min(K_s \times K_b \times [11 + 1.5 \times \sqrt{(f_{ck} / 1\ \text{N/mm}^2)} \times \rho_0 / \rho + 3.2 \times \sqrt{(f_{ck} / 1\ \text{N/mm}^2)} \times (\rho_0 / \rho - 1)^{3/2}], 40 \times K_b) = \mathbf{16}$
Actual span to depth ratio	$(h_{stem} - h_{prop}) / d = \mathbf{0.6}$

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

Crack control - Section 7.3

Limiting crack width	$w_{max} = \mathbf{0.3}$ mm
Variable load factor - EN1990 – Table A1.1	$\psi_2 = \mathbf{0.6}$
Serviceability bending moment	$M_{sls} = \mathbf{0}$ kNm/m
Tensile stress in reinforcement	$\sigma_s = M_{sls} / (A_{sr1.prov} \times z) = \mathbf{0.1}$ N/mm^2
Load duration	Long term
Load duration factor	$k_t = \mathbf{0.4}$
Effective area of concrete in tension	$A_{c.eff} = \min(2.5 \times (h - d), (h - x) / 3, h / 2)$
	$A_{c.eff} = \mathbf{90042}$ mm^2/m
Mean value of concrete tensile strength	$f_{ct.eff} = f_{ctm} = \mathbf{3.0}$ N/mm^2
Reinforcement ratio	$\rho_{p.eff} = A_{sr1.prov} / A_{c.eff} = \mathbf{0.006}$
Modular ratio	$\alpha_e = E_s / E_{cm} = \mathbf{5.998}$
Bond property coefficient	$k_1 = \mathbf{0.8}$
Strain distribution coefficient	$k_2 = \mathbf{0.5}$
	$k_3 = \mathbf{3.4}$
	$k_4 = \mathbf{0.425}$
Maximum crack spacing - exp.7.11	$s_{r.max} = k_3 \times c_{sr} + k_1 \times k_2 \times k_4 \times \phi_{sr1} / \rho_{p.eff} = \mathbf{512}$ mm
Maximum crack width - exp.7.8	$w_k = s_{r.max} \times \max(\sigma_s - k_t \times (f_{ct.eff} / \rho_{p.eff}) \times (1 + \alpha_e \times \rho_{p.eff}), 0.6 \times \sigma_s) / E_s$
	$w_k = \mathbf{0}$ mm
	$w_k / w_{max} = \mathbf{0.001}$

PASS - Maximum crack width is less than limiting crack width

Rectangular section in shear - Section 6.2

Design shear force	$V = \mathbf{29.6}$ kN/m

	$C_{Rd,c} = 0.18 / \gamma_C$ = **0.120**
	$k = \min(1 + \sqrt{(200 \text{ mm} / d)}, 2)$ = **1.915**
Longitudinal reinforcement ratio	$\rho_l = \min(A_{sr1.prov} / d, 0.02)$ = **0.002**
	$v_{min} = 0.035 \text{ N}^{1/2}/\text{mm} \times k^{3/2} \times f_{ck}^{0.5}$ = **0.525** N/mm^2
Design shear resistance - exp.6.2a & 6.2b	$V_{Rd.c} = \max(C_{Rd.c} \times k \times (100 \text{ N}^2/\text{mm}^4 \times \rho_l \times f_{ck})^{1/3}, v_{min}) \times d$
	$V_{Rd.c}$ = **125.4** kN/m
	$V / V_{Rd.c}$ = **0.236**

PASS - Design shear resistance exceeds design shear force

Horizontal reinforcement parallel to face of stem - Section 9.6

Minimum area of reinforcement – cl.9.6.3(1)	$A_{sx.req} = \max(0.25 \times A_{sr.prov}, 0.001 \times t_{stem})$ = **300** mm^2/m
Maximum spacing of reinforcement – cl.9.6.3(2)	s_{sx_max} = **400** mm
Transverse reinforcement provided	10 dia.bars @ 200 c/c
Area of transverse reinforcement provided	$A_{sx.prov} = \pi \times \phi_{sx}^2 / (4 \times s_{sx})$ = **393** mm^2/m

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

Check base design at toe

Depth of section	h = **350** mm

Rectangular section in flexure - Section 6.1

Design bending moment combination 1	M = **1.2** kNm/m
Depth to tension reinforcement	$d = h - c_{bb} - \phi_{bb} / 2$ = **269** mm
	$K = M / (d^2 \times f_{ck})$ = **0.001**
	$K' = (2 \times \eta \times \alpha_{cc}/\gamma_C)\times(1 - \lambda \times (\delta - K_1)/(2 \times K_2))\times(\lambda \times (\delta - K_1)/(2 \times K_2))$
	K' = **0.207**

K' > K - No compression reinforcement is required

Lever arm	$z = \min(0.5 + 0.5 \times (1 - 2 \times K / (\eta \times \alpha_{cc} / \gamma_C))^{0.5}, 0.95) \times d$ = **256** mm
Depth of neutral axis	$x = 2.5 \times (d – z)$ = **34** mm
Area of tension reinforcement required	$A_{bb.req} = M / (f_{yd} \times z)$ = **11** mm^2/m
Tension reinforcement provided	12 dia.bars @ 200 c/c
Area of tension reinforcement provided	$A_{bb.prov} = \pi \times \phi_{bb}^2 / (4 \times s_{bb})$ = **565** mm^2/m
Minimum area of reinforcement - exp.9.1N	$A_{bb.min} = \max(0.26 \times f_{ctm} / f_{yk}, 0.0013) \times d$ = **423** mm^2/m
Maximum area of reinforcement - cl.9.2.1.1(3)	$A_{bb.max} = 0.04 \times h$ = **14000** mm^2/m
	$\max(A_{bb.req}, A_{bb.min}) / A_{bb.prov}$ = **0.748**

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

Library item: Rectangular single output

Crack control - Section 7.3

Limiting crack width	w_{max} = **0.3** mm
Variable load factor - EN1990 – Table A1.1	ψ_2 = **0.6**
Serviceability bending moment	M_{sls} = **9.5** kNm/m
Tensile stress in reinforcement	$\sigma_s = M_{sls} / (A_{bb.prov} \times z)$ = **65.9** N/mm^2
Load duration	Long term
Load duration factor	k_t = **0.4**
Effective area of concrete in tension	$A_{c.eff} = \min(2.5 \times (h - d), (h - x) / 3, h / 2)$
	$A_{c.eff}$ = **105458** mm^2/m
Mean value of concrete tensile strength	$f_{ct.eff} = f_{ctm}$ = **3.0** N/mm^2

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

Modular ratio $\alpha_e = E_s / E_{cm} = \mathbf{5.998}$

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

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

$k_3 = \mathbf{3.4}$

$k_4 = \mathbf{0.425}$

Maximum crack spacing - exp.7.11 $s_{r.max} = k_3 \times c_{bb} + k_1 \times k_2 \times k_4 \times \phi_{bb} / \rho_{p.eff} = \mathbf{635}$ mm

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

$w_k = \mathbf{0.126}$ mm

$w_k / w_{max} = \mathbf{0.419}$

PASS - Maximum crack width is less than limiting crack width

Rectangular section in shear - Section 6.2

Design shear force $V = \mathbf{61.3}$ kN/m

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

$k = \min(1 + \sqrt{(200 \text{ mm} / d)}, 2) = \mathbf{1.862}$

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

$v_{min} = 0.035 \text{ N}^{1/2}/\text{mm} \times k^{3/2} \times f_{ck}^{0.5} = \mathbf{0.503}$ N/mm^2

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

$V_{Rd.c} = \mathbf{135.3}$ kN/m

$V / V_{Rd.c} = \mathbf{0.453}$

PASS - Design shear resistance exceeds design shear force

Check base design at toe

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

Rectangular section in flexure - Section 6.1

Design bending moment combination 1 $M = \mathbf{19.1}$ kNm/m

Depth to tension reinforcement $d = h - c_{bt} - \phi_{bt} / 2 = \mathbf{314}$ mm

$K = M / (d^2 \times f_{ck}) = \mathbf{0.006}$

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

$K' = \mathbf{0.207}$

K' > K - No compression reinforcement is required

Lever arm $z = \min(0.5 + 0.5 \times (1 - 2 \times K / (\eta \times \alpha_{cc} / \gamma_C))^{0.5}, 0.95) \times d = \mathbf{298}$ mm

Depth of neutral axis $x = 2.5 \times (d - z) = \mathbf{39}$ mm

Area of tension reinforcement required $A_{bt.req} = M / (f_{yd} \times z) = \mathbf{147}$ mm^2/m

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

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

Minimum area of reinforcement - exp.9.1N $A_{bt.min} = \max(0.26 \times f_{ctm} / f_{yk}, 0.0013) \times d = \mathbf{494}$ mm^2/m

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

$\max(A_{bt.req}, A_{bt.min}) / A_{bt.prov} = \mathbf{0.873}$

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

Library item: Rectangular single output

Crack control - Section 7.3

Limiting crack width $w_{max} = \mathbf{0.3}$ mm

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

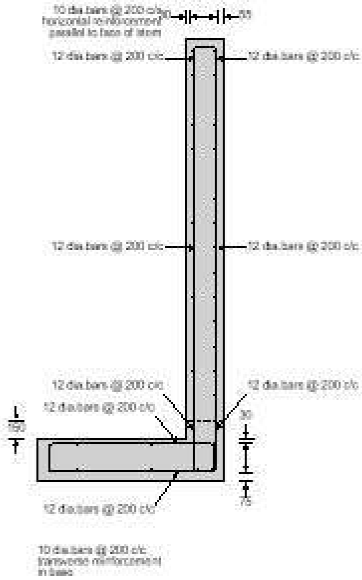
Serviceability bending moment	M_{sls} = **0** kNm/m
Tensile stress in reinforcement	$\sigma_s = M_{sls} / (A_{bt.prov} \times z)$ = **0** N/mm^2
Load duration	Long term
Load duration factor	k_t = **0.4**
Effective area of concrete in tension	$A_{c.eff} = \min(2.5 \times (h - d), (h - x) / 3, h / 2)$
	$A_{c.eff}$ = **90000** mm^2/m
Mean value of concrete tensile strength	$f_{ct.eff} = f_{ctm}$ = **3.0** N/mm^2
Reinforcement ratio	$\rho_{p.eff} = A_{bt.prov} / A_{c.eff}$ = **0.006**
Modular ratio	$\alpha_e = E_s / E_{cm}$ = **5.998**
Bond property coefficient	k_1 = **0.8**
Strain distribution coefficient	k_2 = **0.5**
	k_3 = **3.4**
	k_4 = **0.425**
Maximum crack spacing - exp.7.11	$s_{r.max} = k_3 \times c_{bt} + k_1 \times k_2 \times k_4 \times \phi_{bt} / \rho_{p.eff}$ = **427** mm
Maximum crack width - exp.7.8	$w_k = s_{r.max} \times \max(\sigma_s - k_t \times (f_{ct.eff} / \rho_{p.eff}) \times (1 + \alpha_e \times \rho_{p.eff}), 0.6 \times \sigma_s) / E_s$
	w_k = **0** mm
	w_k / w_{max} = **0**

PASS - Maximum crack width is less than limiting crack width

Secondary transverse reinforcement to base - Section 9.3

Minimum area of reinforcement – cl.9.3.1.1(2)	$A_{bx.req} = 0.2 \times A_{bb.prov}$ = **113** mm^2/m
Maximum spacing of reinforcement – cl.9.3.1.1(3)	s_{bx_max} = **450** mm
Transverse reinforcement provided	10 dia.bars @ 200 c/c
Area of transverse reinforcement provided	$A_{bx.prov} = \pi \times \phi_{bx}^2 / (4 \times s_{bx})$ = **393** mm^2/m

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



Reinforcement details

Calculation Sheet

engineeria
structural and civil engineers

PROJECT TITLE: HAMPTON WICK

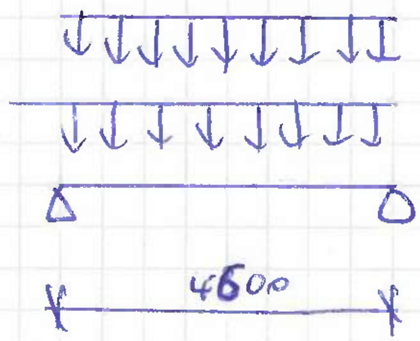
PROJECT NUMBER: E0811 | SHEET NUMBER: CAL-2200 | DATE: 19/07/23

BY: PS | CHECKED BY: MW | REV: P01

TITLE: DESIGN TIMBER FLOOR JOIST TFJ-01

DESIGN WITH THE WORST GEOMETRY POSSIBLY

STRUCTURAL MODEL



DEAD 1,5 kN/m^2

LIVE 2,5 kN/m^2

REFER TO TEDDS CALCULATION CAL-2201

USE 75x225 C/C 300 C24 TIMBER

TIMBER JOIST ANALYSIS & DESIGN (EN1995-1-1:2004)

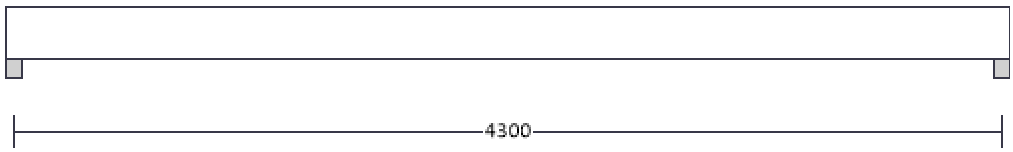
In accordance with EN1995-1-1:2004 + A2:2014 incorporating corrigendum June 2006 and the recommended values

Tedds calculation version 1.0.05

Joist details

Description 75 x 225 C24 timber joists

Joist spacing s_{Joist} = **300** mm



Forces input on Joist

Vertical permanent load on joist F_{G_Joist}= **1.50** kN/m^2

Vertical imposed load on joist F_{Q_Joist}= **2.50** kN/m^2

Joist loading details

Distributed loads

Vertical permanent load on joist $p_G = F_{G\_Joist} \times s_{Joist}$= **0.45** kN/m

Vertical imposed load on joist $p_Q = F_{Q\_Joist} \times s_{Joist}$= **0.75** kN/m

ANALYSIS

Tedds calculation version 1.0.36

Loading

Self weight included (Permanent x 1)

Load combination factors

Load combination	Permanent	Imposed	Snow	Wind
1.35G + 1.50Q (Strength)	1.35	1.50	0.00	0.00
1.00G + 1.00Q (Service)	1.00	1.00	0.00	0.00

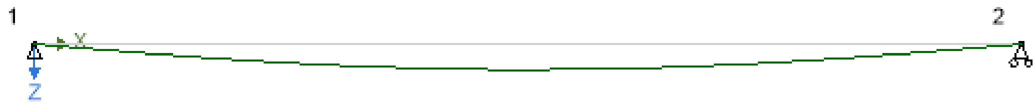
Member Loads

Member	Load case	Load Type	Orientation	Description
Member	Permanent	UDL	GlobalZ	0.45 kN/m at 0 m to 4.3 m
Member	Imposed	UDL	GlobalZ	0.75 kN/m at 0 m to 4.3 m

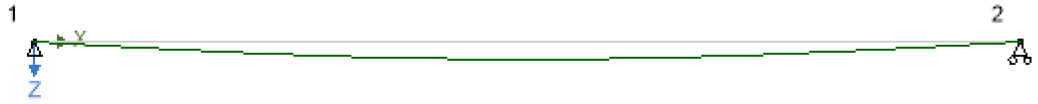
Results

Total deflection

1.35G + 1.50Q (Strength) - Total deflection



1.00G + 1.00Q (Service) - Total deflection



Node deflections

Load combination: 1.35G + 1.50Q (Strength)

Node	Deflection		Rotation	Co-ordinate system
	X (mm)	Z (mm)	(°)	
1	0	0	0.43888	
2	0	0	-0.43888	

Load combination: 1.00G + 1.00Q (Service)

Node	Deflection		Rotation	Co-ordinate system
	X (mm)	Z (mm)	(°)	
1	0	0	0.30489	
2	0	0	-0.30489	

Total base reactions

Load case/combination	Force	
	FX (kN)	FZ (kN)
1.35G + 1.50Q (Strength)	0	7.8
1.00G + 1.00Q (Service)	0	5.4

Element end forces

Load combination: 1.35G + 1.50Q (Strength)

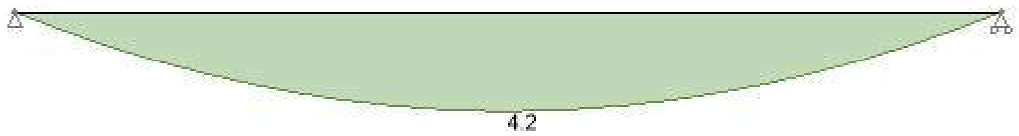
Element	Length (m)	Nodes Start/End	Axial force (kN)	Shear force (kN)	Moment (kNm)
1	4.3	1	0	-3.9	0
		2	0	-3.9	0

Load combination: 1.00G + 1.00Q (Service)

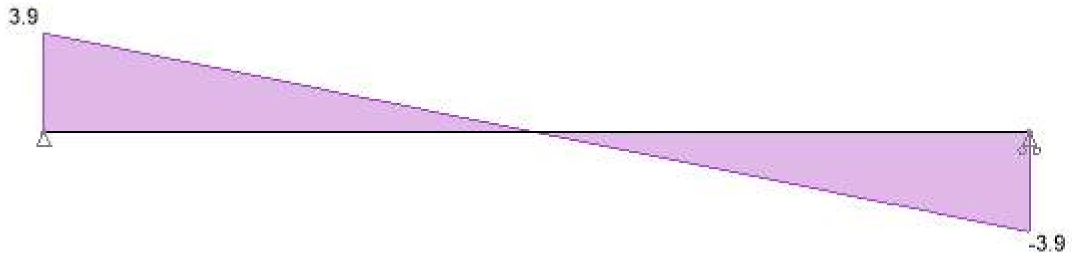
Element	Length (m)	Nodes Start/End	Axial force (kN)	Shear force (kN)	Moment (kNm)
1	4.3	1	0	-2.7	0
		2	0	-2.7	0

Forces

Strength combinations - Moment envelope (kNm)



Strength combinations - Shear envelope (kN)



Member results

Envelope - Strength combinations

Member	Position (m)	Shear force (kN)		Moment (kNm)	
Member	0	3.9 (max abs)		0 (min)	
	2.15	0		4.2 (max)	
	4.3	-3.9		0 (min)	

Tedds calculation version 2.2.10

Member - Span 1

Partial factor for material properties and resistances

Partial factor for material properties - Table 2.3 γ_M = **1.300**

Member details

Load duration - cl.2.3.1.2 Medium-term

Service class - cl.2.3.1.3 1

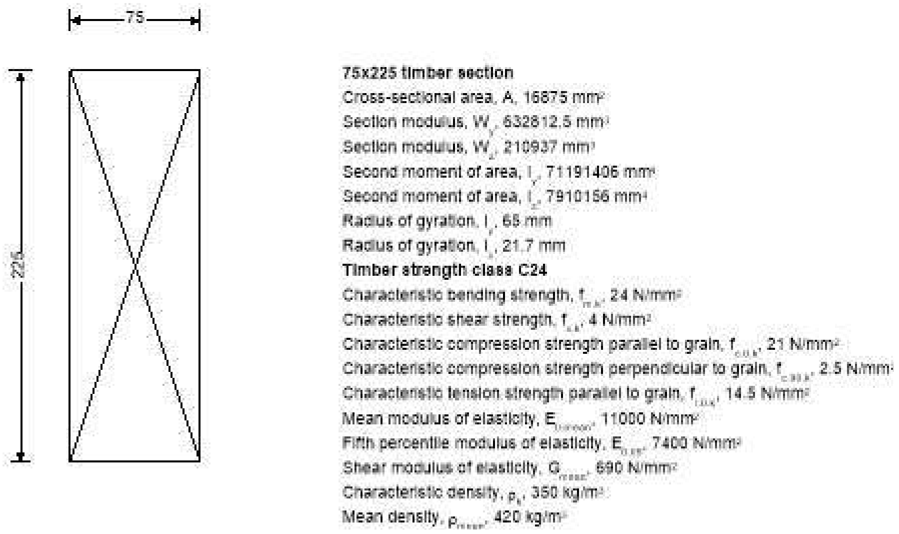
Timber section details

Number of timber sections in member N = **1**

Breadth of sections b = **75** mm

Depth of sections h = **225** mm

Timber strength class - EN 338:2016 Table 1 **C24**



Span details

Bearing length L_b = **75** mm

Member results summary	Unit	Capacity	Maximum	Utilisation	Result
Bearing stress	N/mm²	1.7	0.7	0.409	PASS
Bending stress	N/mm²	16.2	6.6	0.407	PASS
Shear stress	N/mm²	2.7	0.5	0.191	PASS
Deflection	mm	14	11.9	0.851	PASS

Consider Combination 1 - 1.35G + 1.50Q (Strength)

Modification factors

Duration of load and moisture content - Table 3.1 k_{mod} = **0.8**

Deformation factor - Table 3.2 k_{def} = **0.6**

Bending stress re-distribution factor - cl.6.1.6(2) k_m = **0.7**

Crack factor for shear resistance - cl.6.1.7(2) k_{cr} = **0.67**

System strength factor - cl.6.6 k_{sys} = **1.1**

Check design at start of span

Check compression perpendicular to the grain - cl.6.1.5

Design perpendicular compression - major axis $F_{c,y,90,d}$ = **3.893** kN

Effective contact length $L_{b,ef} = L_b$ = **75** mm

Design perpendicular compressive stress - exp.6.4 $\sigma_{c,y,90,d} = F_{c,y,90,d} / (b \times L_{b,ef})$ = **0.692** N/mm²

Design perpendicular compressive strength $f_{c,y,90,d} = k_{mod} \times k_{sys} \times f_{c.90.k} / \gamma_M$ = **1.692** N/mm²

$\sigma_{c,y,90,d} / (k_{c,90} \times f_{c,y,90,d})$ = **0.409**

PASS - Design perpendicular compression strength exceeds design perpendicular compression stress

Check shear force - Section 6.1.7

Design shear force $F_{y,d}$ = **3.893** kN

Design shear stress - exp.6.60 $\tau_{y,d} = 1.5 \times F_{y,d} / (k_{cr} \times b \times h)$ = **0.516** N/mm²

Design shear strength $f_{v,y,d} = k_{mod} \times k_{sys} \times f_{v.k} / \gamma_M$ = **2.708** N/mm²

$\tau_{y,d} / f_{v,y,d}$ = **0.191**

PASS - Design shear strength exceeds design shear stress

Check design 2150 mm along span

Check bending moment - Section 6.1.6

Design bending moment $M_{y,d}$ = **4.185** kNm

Design bending stress $\sigma_{m,y,d} = M_{y,d} / W_y$ = **6.613** N/mm²

Design bending strength $f_{m,y,d} = k_{mod} \times k_{sys} \times f_{m.k} / \gamma_M$ = **16.246** N/mm²

$\sigma_{m,y,d} / f_{m,y,d}$ = **0.407**

PASS - Design bending strength exceeds design bending stress

Check design at end of span

Check compression perpendicular to the grain - cl.6.1.5

Design perpendicular compression - major axis $F_{c,y,90,d}$ = **3.893** kN

Effective contact length $L_{b,ef} = L_b$ = **75** mm

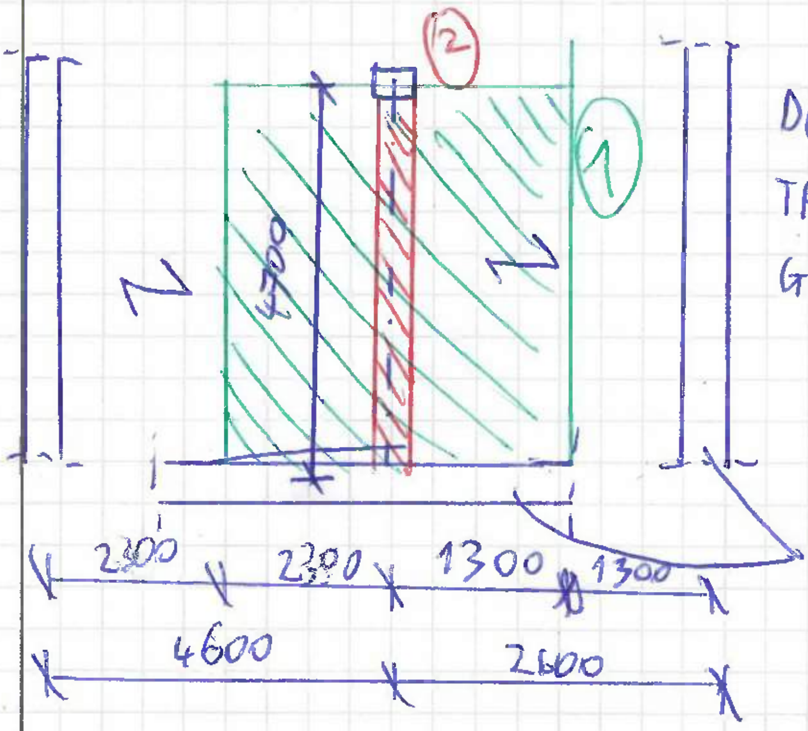
Design perpendicular compressive stress - exp.6.4 $\sigma_{c,y,90,d} = F_{c,y,90,d} / (b \times L_{b,ef})$ = **0.692** N/mm²

Design perpendicular compressive strength $f_{c,y,90,d} = k_{mod} \times k_{sys} \times f_{c.90.k} / \gamma_M$ = **1.692** N/mm²

$\sigma_{c,y,90,d} / (k_{c,90} \times f_{c,y,90,d})$ = **0.409**

Calculation Sheet | engineeria — structural and civil engineers

PROJECT TITLE: [redacted] HAMPTON WICK
PROJECT NUMBER: E0811 | SHEET NUMBER: CAL - 2210 | DATE: 19/07/23
BY: PS | CHECKED BY: MW | REV: P01
TITLE: SB01 DESIGN



DESIGN WITH THE WORST GEOMETRY POSSIBLY

LOADINGS

1 TIMBER FLOOR

DEAD $1{,}5\ kN/m^2 \cdot 3{,}60\ m = 5{,}40\ kN/m$

LIVE $2{,}5\ kN/m^2 \cdot 3{,}60\ m = 9{,}00\ kN/m$

2 ASSUMED 215mm BRICK WALL (WORST CASE)

DEAD - $50\ mm \cdot 25\ kN/m^3$ (RENDER) +

$0{,}215\ m \cdot 22\ kN/m^3$ (BRICK) $= \sim 6\ kN/m^2$ (HEAVY WALL FOR SAFETY)

(ASSUMED HEIGHT OF WALL)

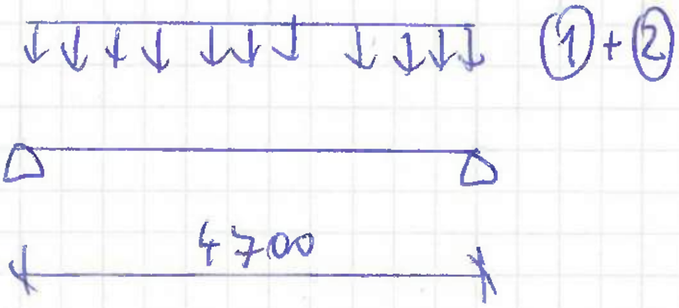
DEAD - $6\ kN/m^2 \cdot 3\ m = 18\ kN/m$

Calculation Sheet

engineeria
structural and civil engineers

PROJECT TITLE: HAMPTON WICK
PROJECT NUMBER: E0811 | SHEET NUMBER: CAL-2211 | DATE: 19/07/23
BY: PS | CHECKED BY: MW | REV: P01
TITLE: SB 01 DESIGN

STRUCTURAL MODEL



DL = 23,4 kN/m

LL = 9,0 kN/m

REFER TO TEDDS CALCULATION CAL-2212

USE 203x 203x 60 UC S355

PASS - Design perpendicular compression strength exceeds design perpendicular compression stress

Check shear force - Section 6.1.7

Design shear force $F_{y,d}$ = **3.893** kN

Design shear stress - exp.6.60 $\tau_{y,d} = 1.5 \times F_{y,d} / (k_{cr} \times b \times h)$ = **0.516** N/mm^2

Design shear strength $f_{v,y,d} = k_{mod} \times k_{sys} \times f_{v.k} / \gamma_M$ = **2.708** N/mm^2

$\tau_{y,d} / f_{v,y,d}$ = **0.191**

PASS - Design shear strength exceeds design shear stress

<u>Consider Combination 2 - 1.00G + 1.00Q (Service)</u>

<u>Check design 2150 mm along span</u>

Check y-y axis deflection - Section 7.2

Instantaneous deflection δ_y = **7.5** mm

Quasi-permanent variable load factor ψ_2 = **0.3**

Final deflection with creep $\delta_{y,Final} = \delta_y \times (1 + k_{def})$ = **11.9** mm

Allowable deflection $\delta_{y,Allowable} = Min(L_{m1\_s1} / 250, 14 mm)$ = **14** mm

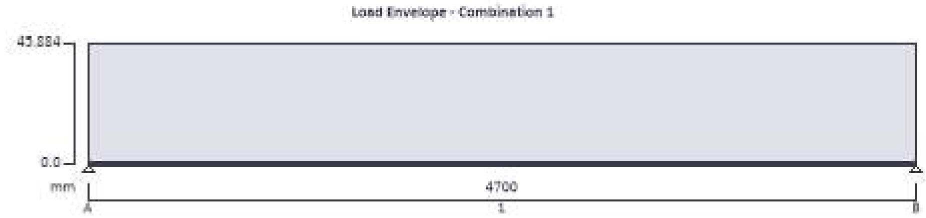
$\delta_{y,Final} / \delta_{y,Allowable}$ = **0.851**

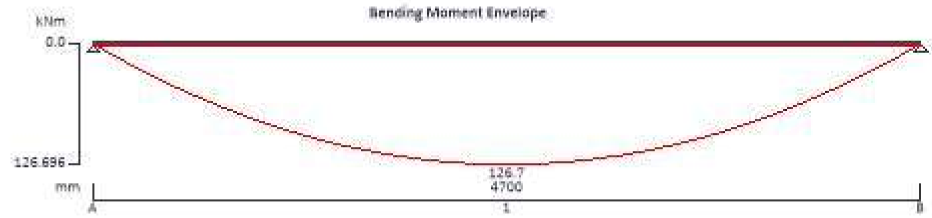
PASS - Allowable deflection exceeds final deflection

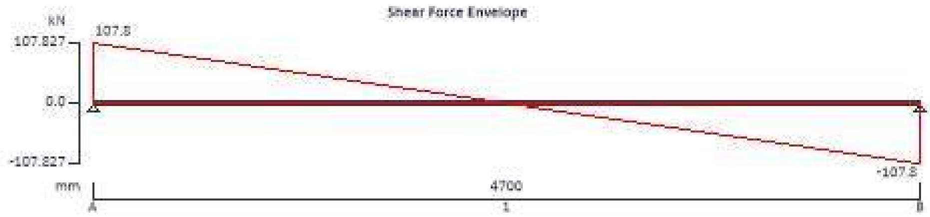
STEEL BEAM ANALYSIS & DESIGN (EN1993-1-1:2005)

In accordance with EN1993-1-1:2005 incorporating Corrigenda February 2006 and April 2009 and the UK national annex

TEDDS calculation version 3.0.14







Support conditions

Support A	Vertically restrained Rotationally free
Support B	Vertically restrained Rotationally free

Applied loading

Beam loads	Permanent self weight of beam × 1 Permanent full UDL 23.4 kN/m Variable full UDL 9 kN/m

Load combinations

Load combination 1	Support A	Permanent × 1.35 Variable × 1.50
		Permanent × 1.35 Variable × 1.50
	Support B	Permanent × 1.35 Variable × 1.50

Analysis results

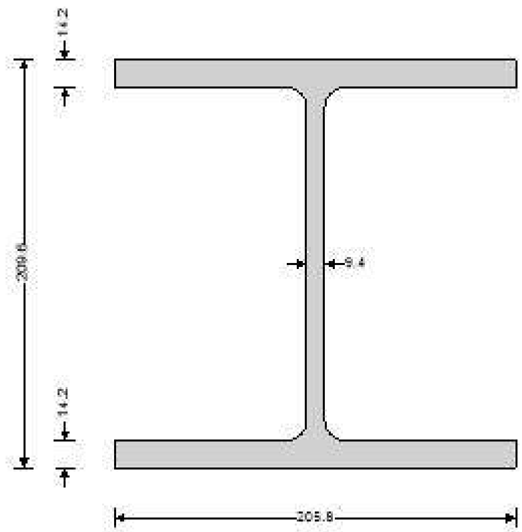
Maximum moment	M_{max} = **126.7** kNm	M_{min} = **0** kNm
Maximum shear	V_{max} = **107.8** kN	V_{min} = **-107.8** kN
Deflection	δ_{max} = **16.3** mm	δ_{min} = **0** mm
Maximum reaction at support A	R_{A_max} = **107.8** kN	R_{A_min} = **107.8** kN
Unfactored permanent load reaction at support A	$R_{A\_Permanent}$ = **56.4** kN	
Unfactored variable load reaction at support A	$R_{A\_Variable}$ = **21.2** kN	
Maximum reaction at support B	R_{B_max} = **107.8** kN	R_{B_min} = **107.8** kN
Unfactored permanent load reaction at support B	$R_{B\_Permanent}$ = **56.4** kN	
Unfactored variable load reaction at support B	$R_{B\_Variable}$ = **21.1** kN	

Section details

Section type	**UC 203x203x60 (BS4-1)**
Steel grade	**S355**

EN 10025-2:2004 - Hot rolled products of structural steels

Nominal thickness of element	$t = \max(t_f, t_w)$ = **14.2** mm
Nominal yield strength	f_y = **355** N/mm^2
Nominal ultimate tensile strength	f_u = **470** N/mm^2
Modulus of elasticity	E = **210000** N/mm^2



Partial factors - Section 6.1

Resistance of cross-sections	γ_{M0} = **1.00**
Resistance of members to instability	γ_{M1} = **1.00**
Resistance of tensile members to fracture	γ_{M2} = **1.10**

Lateral restraint

Span 1 has lateral restraint at supports only

Effective length factors

Effective length factor in major axis	K_y = **1.000**
Effective length factor in minor axis	K_z = **1.000**
Effective length factor for torsion	$K_{LT.A}$ = **1.000**
	$K_{LT.B}$ = **1.000**

Classification of cross sections - Section 5.5

$\varepsilon = \sqrt{[235 \text{ N/mm}^2 / f_y]} = \mathbf{0.81}$

Internal compression parts subject to bending - Table 5.2 (sheet 1 of 3)

Width of section — $c = d = \mathbf{160.8}$ mm

$c / t_w = 21.0 \times \varepsilon <= 72 \times \varepsilon$ — Class 1

Outstand flanges - Table 5.2 (sheet 2 of 3)

Width of section — $c = (b - t_w - 2 \times r) / 2 = \mathbf{88}$ mm

$c / t_f = 7.6 \times \varepsilon <= 9 \times \varepsilon$ — Class 1

Section is class 1

Check shear - Section 6.2.6

Height of web — $h_w = h - 2 \times t_f = \mathbf{181.2}$ mm

Shear area factor — $\eta = \mathbf{1.000}$

$h_w / t_w < 72 \times \varepsilon / \eta$

Shear buckling resistance can be ignored

Design shear force — $V_{Ed} = \max(\text{abs}(V_{max}), \text{abs}(V_{min})) = \mathbf{107.8}$ kN

Shear area - cl 6.2.6(3) — $A_v = \max(A - 2 \times b \times t_f + (t_w + 2 \times r) \times t_f, \eta \times h_w \times t_w) = \mathbf{2216}$ mm^2

Design shear resistance - cl 6.2.6(2) — $V_{c,Rd} = V_{pl,Rd} = A_v \times (f_y / \sqrt{[3]}) / \gamma_{M0} = \mathbf{454.1}$ kN

PASS - Design shear resistance exceeds design shear force

Check bending moment major (y-y) axis - Section 6.2.5

Design bending moment — $M_{Ed} = \max(\text{abs}(M_{s1\_max}), \text{abs}(M_{s1\_min})) = \mathbf{126.7}$ kNm

Design bending resistance moment - eq 6.13 — $M_{c,Rd} = M_{pl,Rd} = W_{pl.y} \times f_y / \gamma_{M0} = \mathbf{232.9}$ kNm

Slenderness ratio for lateral torsional buckling

Correction factor - Table 6.6 — $k_c = \mathbf{0.94}$

$C_1 = 1 / k_c^2 = \mathbf{1.132}$

Curvature factor — $g = \sqrt{[1 - (I_z / I_y)]} = \mathbf{0.814}$

Poissons ratio — $\nu = \mathbf{0.3}$

Shear modulus — $G = E / [2 \times (1 + \nu)] = \mathbf{80769}$ N/mm^2

Unrestrained length — $L = 1.0 \times L_{s1} = \mathbf{4700}$ mm

Elastic critical buckling moment — $M_{cr} = C_1 \times \pi^2 \times E \times I_z / (L^2 \times g) \times \sqrt{[I_w / I_z + L^2 \times G \times I_t / (\pi^2 \times E \times I_z)]} = \mathbf{460.4}$ kNm

Slenderness ratio for lateral torsional buckling — $\bar{\lambda}_{LT} = \sqrt{(W_{pl.y} \times f_y / M_{cr})} = \mathbf{0.711}$

Limiting slenderness ratio — $\bar{\lambda}_{LT,0} = \mathbf{0.4}$

$\bar{\lambda}_{LT} > \bar{\lambda}_{LT,0}$ - Lateral torsional buckling cannot be ignored

Design resistance for buckling - Section 6.3.2.1

Buckling curve - Table 6.5 — b

Imperfection factor - Table 6.3 — $\alpha_{LT} = \mathbf{0.34}$

Correction factor for rolled sections — $\beta = \mathbf{0.75}$

LTB reduction determination factor — $\phi_{LT} = 0.5 \times [1 + \alpha_{LT} \times (\bar{\lambda}_{LT} - \bar{\lambda}_{LT,0}) + \beta \times \bar{\lambda}_{LT}^2] = \mathbf{0.743}$

LTB reduction factor - eq 6.57 — $\chi_{LT} = \min(1 / [\phi_{LT} + \sqrt{(\phi_{LT}^2 - \beta \times \bar{\lambda}_{LT}^2)}], 1, 1 / \bar{\lambda}_{LT}^2) = \mathbf{0.864}$

Modification factor — $f = \min(1 - 0.5 \times (1 - k_c) \times [1 - 2 \times (\bar{\lambda}_{LT} - 0.8)^2], 1) = \mathbf{0.970}$

Modified LTB reduction factor - eq 6.58 — $\chi_{LT,mod} = \min(\chi_{LT} / f, 1) = \mathbf{0.890}$

Design buckling resistance moment - eq 6.55 — $M_{b,Rd} = \chi_{LT,mod} \times W_{pl.y} \times f_y / \gamma_{M1} = \mathbf{207.4}$ kNm

PASS - Design buckling resistance moment exceeds design bending moment

Check vertical deflection - Section 7.2.1

Consider deflection due to permanent and variable loads

Limiting deflection $\delta_{lim} = L_{s1} / 250 = \mathbf{18.8}$ mm

Maximum deflection span 1 $\delta = max(abs(\delta_{max}), abs(\delta_{min})) = \mathbf{16.297}$ mm

PASS - Maximum deflection does not exceed deflection limit

Calculation Sheet

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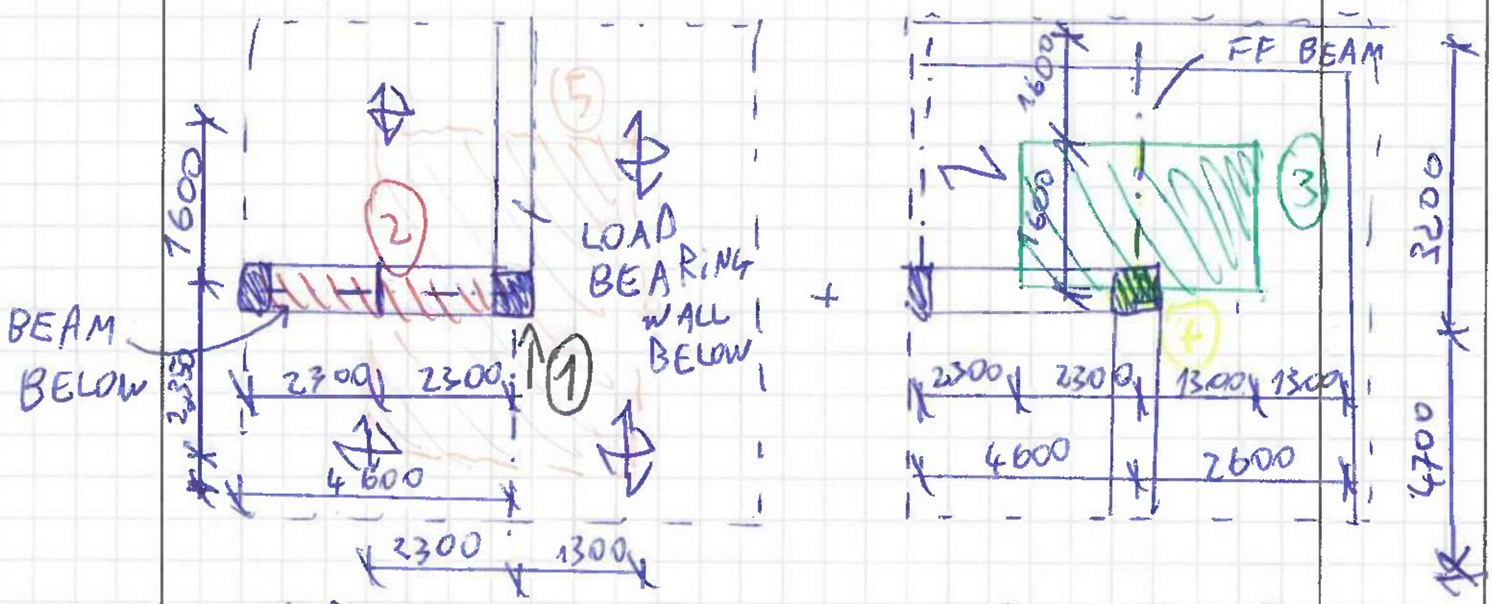
PROJECT TITLE: HAMPTON WICK
PROJECT NUMBER: E0811 | SHEET NUMBER: CAL-25010 | DATE: 19/07/23
BY: PS | CHECKED BY: MW | REV: P01
TITLE: SC01 DESIGN



GROUND FLOOR FIRST FLOOR

DESIGN WITH THE WORST GEOMETRY POSSIBLY

ASSUMED HEIGHT OF WALL + COLUMN = 3 m
(FF ; GF = 3 m)

LOADINGS

1 REACTION FROM SB01 (TEDDS ; UNFACTORED)

DEAD 56,4 kN
LIVE 21,2 kN

(WORST CASE)

2 ASSUMED HEAVY 215mm MASONRY WALL

DEAD = $6 kN/m^2 \cdot 3m \cdot 2,3m = 11,3 kN$

(FOR SAFETY + TO INCLUDE SELF-WEIGHT OF STEEL BEAM UNDER USE 12 kN

Calculation Sheet engineeria structural and civil engineers

PROJECT TITLE:	HAMPTON WICK				
PROJECT NUMBER:	E0811	SHEET NUMBER:	CAL - 2501	DATE:	19/07/23
BY:	PS	CHECKED BY:	MW	REV:	P01
TITLE:	SC01 DESIGN				

(3) TIMBER FLOOR

DEAD $1,5\,kN/m^2 \cdot 3,6\,m = 5,40\,kN/m$

LIVE $2,5\,kN/m^2 \cdot 3,6\,m = 9,00\,kN/m$

HALF OF STEEL BEAM → DEAD $5,4\,kN/m \cdot 1,6\,m = 8,64\,kN$

LIVE $9,0\,kN/m \cdot 1,6\,m = 14,4\,kN$

(4) SELF - WEIGHT OF STEEL COLUMN + BEAM

LETS ASSUME <u>2kN</u> $((3m + 1,6m) \cdot \sim 0,3\,kN/m)$

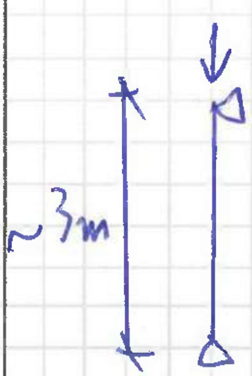
$= \sim 1,4\,kN$ WEIGHT OF STEEL

(5) GROUND FLOOR RC SLAB

DEAD $1,5\,kN/m^2 \cdot 3,6\,m \cdot 3,95\,m = 21,33\,kN$

LIVE $4,0\,kN/m^2 \cdot 3,6\,m \cdot 3,95\,m = 64,15\,kN$

STRUCTURAL MODEL (UNFACTORED)



$N_{Ek} = 100,4\,kN$ (DEAD) $+ 99,75$ (LIVE) kN

$\downarrow \cdot 1,35$ $\downarrow \cdot 1,5$

$N_{Ed} = 135\,kN + 150\,kN = \underline{285\,kN}$

BUCKLING LENGTH $\mu = 1,0$

$L_0 = 3m$

LOADINGS $N_{Ed} = 285\,kN$ (FACTORED)

$e = \max\left(\frac{L_0}{400}, \frac{h}{30}, 20mm\right)$ → $M_{Ed} = 0,02m \cdot 285\,kN = \sim 6\,kNm$

FOR SAFETY USE 300kN AND 10kNm

REFER TO TEDDS CALCULATION CAL-2502

<u>USE 100 x 100 x 8.0 SHS S355</u>

7 Ridgmount Street, WC1E 7AE, London, United Kingdom contact@engineeria.com (+44)207 580 4588 www.engineeria.com	Project HAMPTON WICK				Job no. E0801	
	Calcs for SC-61 DESIGN				Start page no./Revision CAL- 2502 P01	
	Calcs by PS	Calcs date 19/07/2023	Checked by DP	Checked date	Approved by DP	Approved date

STEEL COLUMN DESIGN

In accordance with EN1993-1-1:2005 incorporating Corrigenda February 2006 and April 2009 and the UK national annex

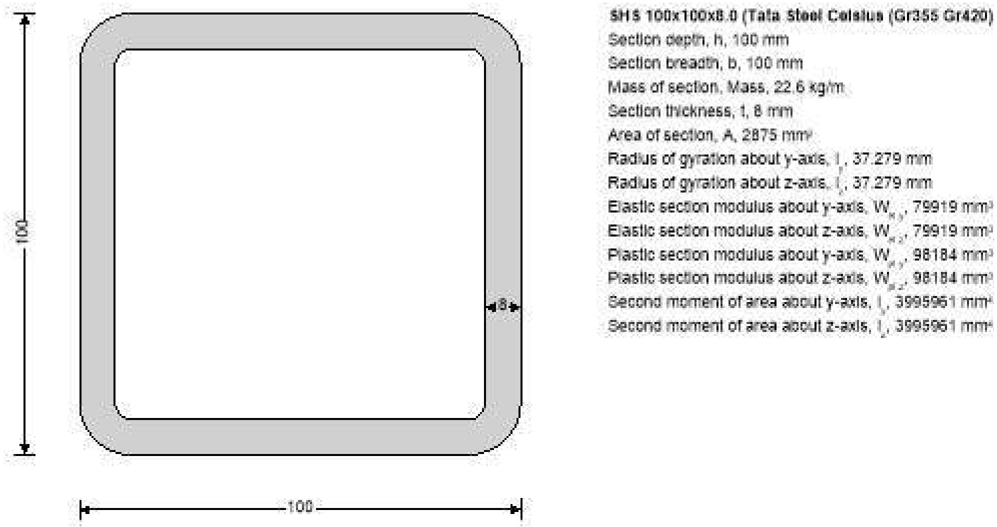
Tedds calculation version 1.1.06

Partial factors - Section 6.1

Resistance of cross-sections γ_{M0} = **1**

Resistance of members to instability γ_{M1} = **1**

Resistance of cross-sections in tension to fracture γ_{M2} = **1.1**



Column details

Column section **SHS 100x100x8.0**

Steel grade **S355H**

Yield strength f_y = **355** N/mm^2

Ultimate strength f_u = **470** N/mm^2

Modulus of elasticity E = **210** kN/mm^2

Poisson's ratio ν = **0.3**

Shear modulus G = E / [2 × (1 + ν)] = **80.8** kN/mm^2

Column geometry

System length for buckling - Major axis L_y = **3000** mm

System length for buckling - Minor axis L_z = **3000** mm

The column is not part of a sway frame in the direction of the minor axis

The column is not part of a sway frame in the direction of the major axis

Column loading

Axial load N_{Ed} = **300** kN (Compression)

Major axis moment at end 1 - Bottom $M_{y,Ed1}$ = **0.0** kNm

Major axis moment at end 2 - Top $M_{y,Ed2}$ = **10.0** kNm

Major axis bending is single curvature

Minor axis moment at end 1 - Bottom $M_{z,Ed1}$ = **0.0** kNm

Minor axis moment at end 2 - Top $M_{z,Ed2}$ = **10.0** kNm

Minor axis bending is single curvature

Major axis shear force $V_{y,Ed}$ = **0** kN

Minor axis shear force $V_{z,Ed}$ = **0** kN

Buckling length for flexural buckling - Major axis

End restraint factor K_y = **1.000**

Buckling length $L_{cr\_y} = L_y \times K_y$ = **3000** mm

Buckling length for flexural buckling - Minor axis

End restraint factor K_z = **1.000**

Buckling length $L_{cr\_z} = L_z \times K_z$ = **3000** mm

Web section classification (Table 5.2)

Coefficient depending on f_y $\varepsilon = \sqrt{(235 \text{ N/mm}^2 / f_y)}$ = **0.814**

Depth between fillets $c_w = h - 3 \times t$ = **76.0** mm

Ratio of c/t $ratio_w = c_w / t$ = **9.50**

Length of web taken by axial load $l_w = \min(N_{Ed} / (2 \times f_y \times t), c_w)$ =**52.8** mm

For class 1 & 2 proportion in compression $\alpha = (c_w/2 + l_w/2) / c_w$ = **0.847**

Limit for class 1 web $Limit_{1w} = (396 \times \varepsilon) / (13 \times \alpha - 1)$ = **32.16**

The web is class 1

Flange section classification (Table 5.2)

Depth between fillets $c_f = b - 3 \times t$ = **76.0** mm

Ratio of c/t $ratio_f = c_f / t$ = **9.50**

Conservatively assume uniform compression in flange

Limit for class 1 flange $Limit_{1f} = 33 \times \varepsilon$ = **26.85**

Limit for class 2 flange $Limit_{2f} = 38 \times \varepsilon$ = **30.92**

Limit for class 3 flange $Limit_{3f} = 42 \times \varepsilon$ = **34.17**

The flange is class 1

Overall section classification

The section is class 1

Resistance of cross section (cl. 6.2)

Compression (cl. 6.2.4)

Design force N_{Ed} = **300** kN

Design resistance $N_{c,Rd} = N_{pl,Rd} = A \times f_y / \gamma_{M0}$ = **1021** kN

$N_{Ed} / N_{c,Rd}$ = **0.294**

PASS - The compression design resistance exceeds the design force

Bending - Major axis (cl. 6.2.5)

Design bending moment $M_{y,Ed} = \max(\text{abs}(M_{y,Ed1}), \text{abs}(M_{y,Ed2}))$ = **10.0** kNm

Section modulus $W_y = W_{pl.y}$ = **98.2** cm^3

Design resistance $M_{c,y,Rd} = W_y \times f_y / \gamma_{M0}$ = **34.9** kNm

$M_{y,Ed} / M_{c,y,Rd}$ = **0.287**

PASS - The bending design resistance exceeds the design moment

Bending - Major axis(cl. 6.2.5)

Design bending moment $M_{z,Ed} = \max(\text{abs}(M_{z,Ed1}), \text{abs}(M_{z,Ed2}))$ = **10.0** kNm

Section modulus $W_z = W_{pl.z}$ = **98.2** cm^3

Design resistance $M_{c,z,Rd} = W_z \times f_y / \gamma_{M0}$ = **34.9** kNm

$M_{z,Ed} / M_{c,z,Rd}$ = **0.287**

PASS - The bending design resistance exceeds the design moment

Combined bending and axial force (cl. 6.2.9)

Ratio design axial to design plastic resistance — $n = abs(N_{Ed}) / N_{pl,Rd}$ = **0.294**

Ratio web area to gross area — $a_w = min(0.5, (A - 2 \times b \times t) / A)$ = **0.444**

Ratio flange area to gross area — $a_f = min(0.5, (A - 2 \times h \times t) / A)$ = **0.444**

Bending - Major axis (cl. 6.2.9.1)

Design bending moment — $M_{y,Ed} = max(abs(M_{y,Ed1}), abs(M_{y,Ed2}))$ = **10.0** kNm

Plastic design resistance — $M_{pl,y,Rd} = W_{pl.y} \times f_y / \gamma_{M0}$ = **34.9** kNm

Modified design resistance — $M_{N,y,Rd} = M_{pl,y,Rd} \times min(1, (1 - n) / (1 - 0.5 \times a_w))$ = **31.6** kNm

$M_{y,Ed} / M_{N,y,Rd}$ = **0.316**

PASS - Bending resistance in presence of axial load exceeds design moment

Bending - Minor axis (cl. 6.2.9.1)

Design bending moment — $M_{z,Ed} = max(abs(M_{z,Ed1}), abs(M_{z,Ed2}))$ = **10.0** kNm

Plastic design resistance — $M_{pl,z,Rd} = W_{pl.z} \times f_y / \gamma_{M0}$ = **34.9** kNm

Modified design resistance — $M_{N,z,Rd} = M_{pl,z,Rd} \times min(1, (1 - n) / (1 - 0.5 \times a_f))$ = **31.6** kNm

$M_{z,Ed} / M_{N,z,Rd}$ = **0.316**

PASS - Bending resistance in presence of axial load exceeds design moment

Biaxial bending

Exponent α — $\alpha = min(6, 1.66 / (1 - 1.13 \times n^2))$ = **1.84**

Exponent β — $\beta = min(6, 1.66 / (1 - 1.13 \times n^2))$ = **1.84**

Section utilisation at end 1 — $UR_{CS\_1} = [abs(M_{y,Ed1}) / M_{N,y,Rd}]^{\alpha} + [abs(M_{z,Ed1}) / M_{N,z,Rd}]^{\beta}$ = **0.000**

Section utilisation at end 2 — $UR_{CS\_2} = [abs(M_{y,Ed2}) / M_{N,y,Rd}]^{\alpha} + [abs(M_{z,Ed2}) / M_{N,z,Rd}]^{\beta}$ = **0.241**

PASS - The cross-section resistance is adequate

Buckling resistance (cl. 6.3)

Yield strength for buckling resistance — f_y = **355** N/mm²

Flexural buckling - Major axis

Elastic critical buckling force — $N_{cr,y} = \pi^2 \times E \times I_y / L_{cr\_y}^2$ = **920** kN

Non-dimensional slenderness — $\bar{\lambda}_y = \sqrt{(A \times f_y / N_{cr,y})}$ = **1.053**

Buckling curve (Table 6.2) — **a**

Imperfection factor (Table 6.1) — α_y = **0.21**

Parameter Φ — $\Phi_y = 0.5 \times [1 + \alpha_y \times (\bar{\lambda}_y - 0.2) + \bar{\lambda}_y^2]$ = **1.144**

Reduction factor — $\chi_y = min(1.0, 1 / [\Phi_y + \sqrt{(\Phi_y^2 - \bar{\lambda}_y^2)}])$ = **0.628**

Design buckling resistance — $N_{b,y,Rd} = \chi_y \times A \times f_y / \gamma_{M1}$ = **641.4** kN

$N_{Ed} / N_{b,y,Rd}$ = **0.468**

PASS - The flexural buckling resistance exceeds the design axial load

Flexural buckling - Minor axis

Elastic critical buckling force — $N_{cr,z} = \pi^2 \times E \times I_z / L_{cr\_z}^2$ = **920** kN

Non-dimensional slenderness — $\bar{\lambda}_z = \sqrt{(A \times f_y / N_{cr,z})}$ = **1.053**

Buckling curve (Table 6.2) — **a**

Imperfection factor (Table 6.1) — α_z = **0.21**

Parameter Φ — $\Phi_z = 0.5 \times [1 + \alpha_z \times (\bar{\lambda}_z - 0.2) + \bar{\lambda}_z^2]$ = **1.144**

Reduction factor — $\chi_z = min(1.0, 1 / [\Phi_z + \sqrt{(\Phi_z^2 - \bar{\lambda}_z^2)}])$ = **0.628**

Design buckling resistance — $N_{b,z,Rd} = \chi_z \times A \times f_y / \gamma_{M1}$ = **641.4** kN

$N_{Ed} / N_{b,z,Rd}$ = **0.468**

PASS - The flexural buckling resistance exceeds the design axial load

Minimum buckling resistance

Minimum buckling resistance $N_{b,Rd} = \min(N_{b,y,Rd}, N_{b,z,Rd}) = \mathbf{641.4}$ kN

$N_{Ed} / N_{b,Rd} = \mathbf{0.468}$

PASS - The axial load buckling resistance exceeds the design axial load

Buckling resistance moment (cl.6.3.2.1)

Square hollow section not subject to lateral torsional buckling therefore:-

Reduction factor $\chi_{LT} = \mathbf{1.0}$

Design buckling resistance moment $M_{b,Rd} = \chi_{LT} \times W_y \times f_y / \gamma_{M1} = \mathbf{34.9}$ kNm

Design bending moment $M_{y,Ed} = \max(\mathrm{abs}(M_{y,Ed1}), \mathrm{abs}(M_{y,Ed2})) = \mathbf{10.0}$ kNm

$M_{y,Ed} / M_{b,Rd} = \mathbf{0.287}$

PASS - The design buckling resistance moment exceeds the maximum design moment

Combined bending and axial compression (cl. 6.3.3)

Characteristic resistance to normal force $N_{Rk} = A \times f_y = \mathbf{1021}$ kN

Characteristic moment resistance - Major axis $M_{y,Rk} = W_{pl.y} \times f_y = \mathbf{34.9}$ kNm

Characteristic moment resistance - Minor axis $M_{z,Rk} = W_{pl.z} \times f_y = \mathbf{34.9}$ kNm

ψ_y = if(abs($M_{y,Ed1}$)<=abs($M_{y,Ed2}$), $M_{y,Ed1}$ / if($M_{y,Ed2}$>=0 kNm,max($M_{y,Ed2}$,0.0001 kNm),$M_{y,Ed2}$), $M_{y,Ed2}$ / if($M_{y,Ed1}$>=0 kNm,max($M_{y,Ed1}$,0.0001 kNm),$M_{y,Ed1}$)) = **0.000**

Moment distribution factor - Major axis $\psi_y = M_{y,Ed1} / M_{y,Ed2} = \mathbf{0.000}$

Moment factor - Major axis $C_{my} = \max(0.4, 0.6 + 0.4 \times \psi_y) = \mathbf{0.600}$

Moment distribution factor - Minor axis $\psi_z = M_{z,Ed1} / M_{z,Ed2} = \mathbf{0.000}$

Moment factor - Minor axis $C_{mz} = \max(0.4, 0.6 + 0.4 \times \psi_z) = \mathbf{0.600}$

Moment distribution factor for LTB $\psi_{LT} = M_{y,Ed1} / M_{y,Ed2} = \mathbf{0.000}$

Moment factor for LTB $C_{mLT} = \max(0.4, 0.6 + 0.4 \times \psi_{LT}) = \mathbf{0.600}$

Interaction factor k_{yy} $k_{yy} = C_{my} \times [1 + \min(0.8, \bar{\lambda}_y - 0.2) \times N_{Ed} / (\chi_y \times N_{Rk} / \gamma_{M1})] = \mathbf{0.825}$

Interaction factor k_{zy} $k_{zy} = 0.6 \times k_{yy} = \mathbf{0.495}$

Interaction factor k_{zz} $k_{zz} = C_{mz} \times [1 + \min(0.8, \bar{\lambda}_z - 0.2) \times N_{Ed} / (\chi_z \times N_{Rk} / \gamma_{M1})] = \mathbf{0.825}$

Interaction factor k_{yz} $k_{yz} = 0.6 \times k_{zz} = \mathbf{0.495}$

Section utilisation $UR_{B\_1} = N_{Ed} / (\chi_y \times N_{Rk} / \gamma_{M1}) + k_{yy} \times M_{y,Ed} / (\chi_{LT} \times M_{y,Rk} / \gamma_{M1}) + k_{yz} \times M_{z,Ed} / (M_{z,Rk} / \gamma_{M1})$

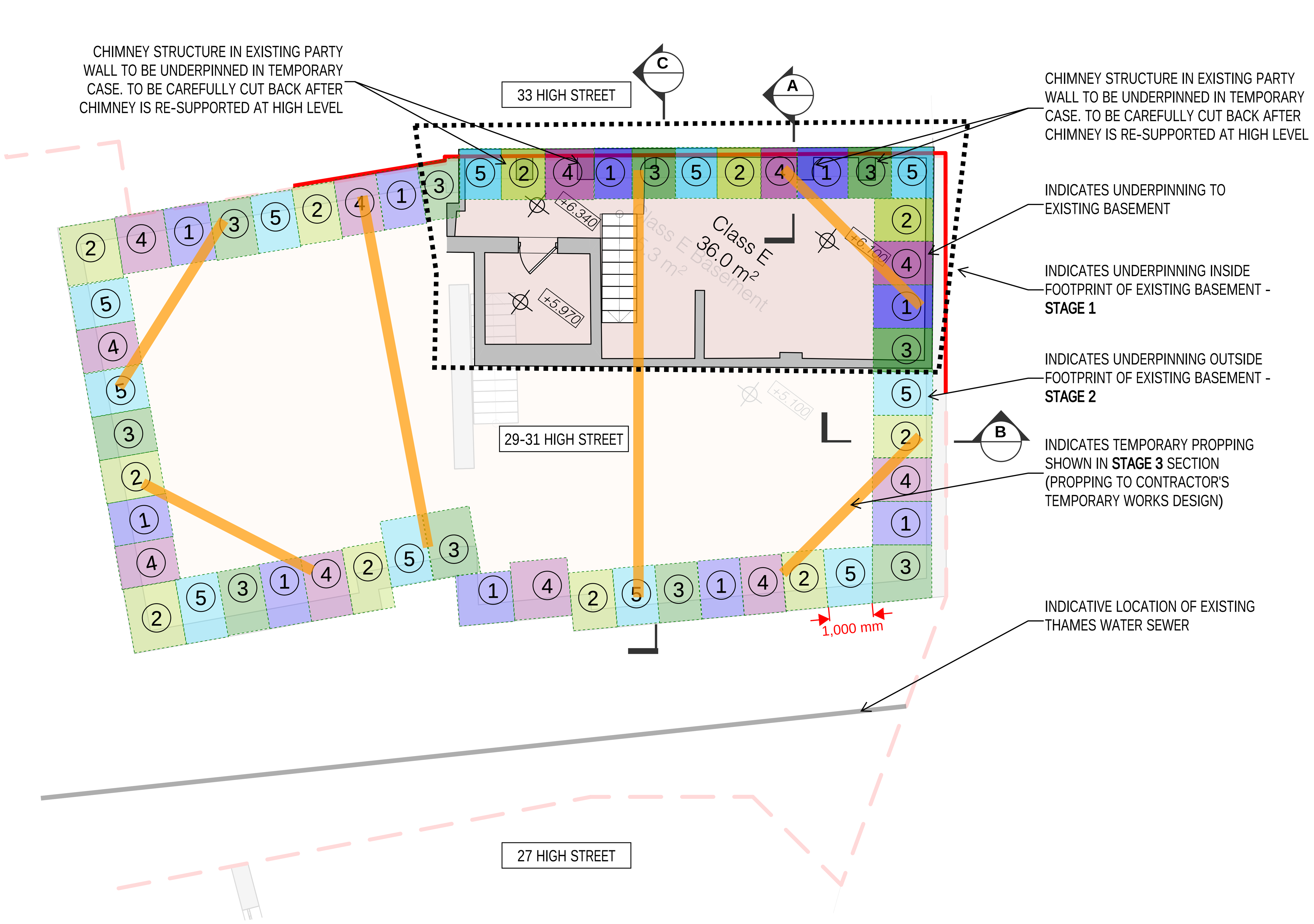
$UR_{B\_1} = \mathbf{0.846}$

$UR_{B\_2} = N_{Ed} / (\chi_z \times N_{Rk} / \gamma_{M1}) + k_{zy} \times M_{y,Ed} / (\chi_{LT} \times M_{y,Rk} / \gamma_{M1}) + k_{zz} \times M_{z,Ed} / (M_{z,Rk} / \gamma_{M1})$

$UR_{B\_2} = \mathbf{0.846}$

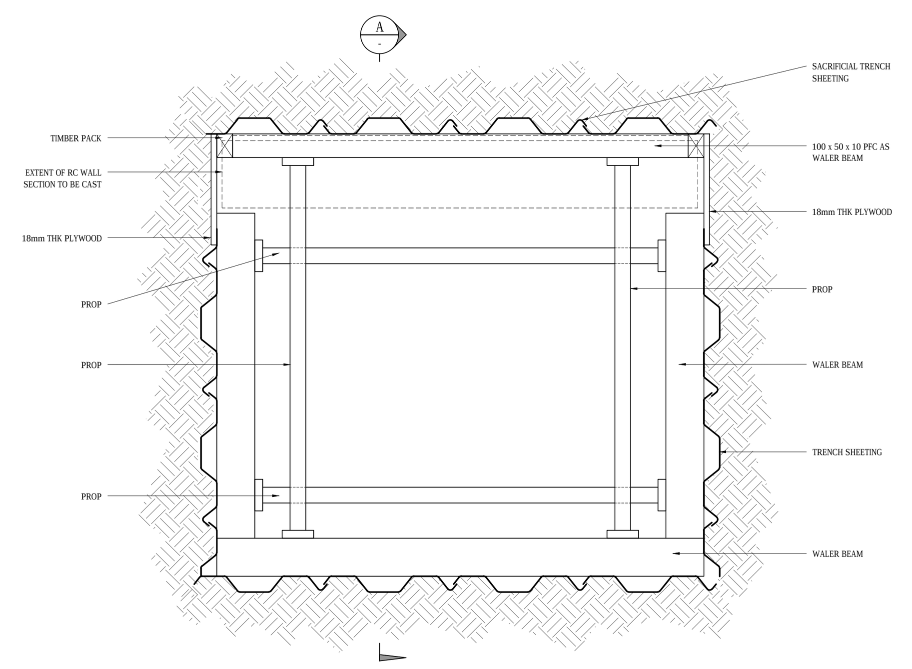
PASS - The buckling resistance is adequate

Appendix D - Proposed Construction Sequence



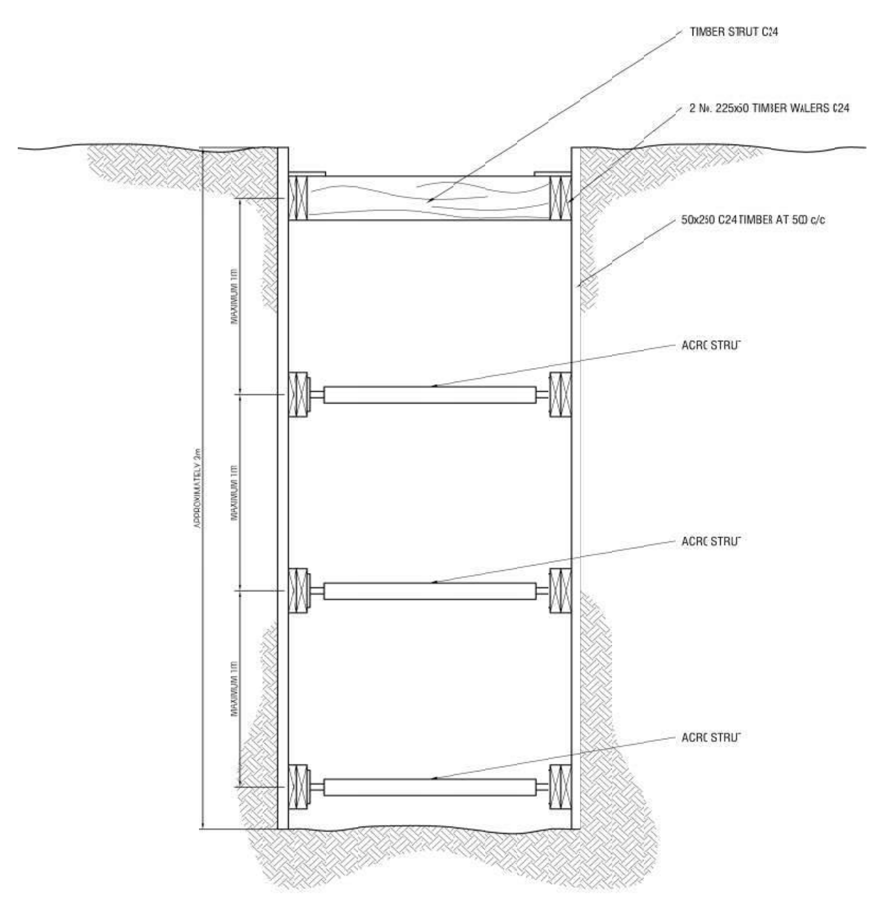
STAGES 1, 2, 3 - PLAN VIEW

N.T.S.

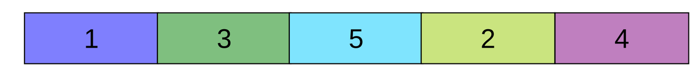


TYPICAL UNDERPIN EXCAVATION SUPPORT PLAN

N.T.S.



TYPICAL UNDERPINNING CONSTRUCTION SEQUENCING KEY:



TYPICAL UNDERPIN EXCAVATION SUPPORT SECTION

N.T.S.

1. THIS DRAWING IS TO BE READ IN CONJUNCTION WITH ALL ENGINEER'S AND ARCHITECT'S DRAWINGS, SPECIFICATIONS AND RISK REGISTERS.
2. DO NOT SCALE FROM THIS DRAWING. USE ONLY DIMENSIONS AS INDICATED. CHECK ALL SITE DIMENSIONS PRIOR TO PLACING ANY ORDER OR FABRICATION. WHERE A CONFLICT OF INFORMATION EXISTS SEEK CONFIRMATION FROM CONSULTANTS PRIOR TO PROCEEDING FURTHER WITH THE WORKS
3. TEMPORARY STABILITY OF THE EXISTING STRUCTURE AND ANY NEWLY CONSTRUCTED ELEMENTS OF PERMANENT WORKS DURING CONSTRUCTION IS SOLELY CONTRACTOR'S RESPONSIBILITY
4. ONLY DRAWINGS AND SPECIFICATIONS ISSUED FOR **CONSTRUCTION** CAN BE USED FOR THE WORKS. IT IS THE CONTRACTOR'S RESPONSIBILITY TO SEEK THE INFORMATION FROM CONSULTANTS.
5. ALL PROPRIETARY ITEMS TO BE INSTALLED STRICTLY IN ACCORDANCE WITH MANUFACTURER'S REQUIREMENTS AND SPECIFICATIONS
6. ALL WATERPROOFING SUCH AS TANKING DETAILS, DAMP PROOF MEMBRANES, DAMP PROOF COURSES, CAVITY TRAYS ETC. ARE TO BE INSTALLED AS PER ARCHITECT'S DETAILS

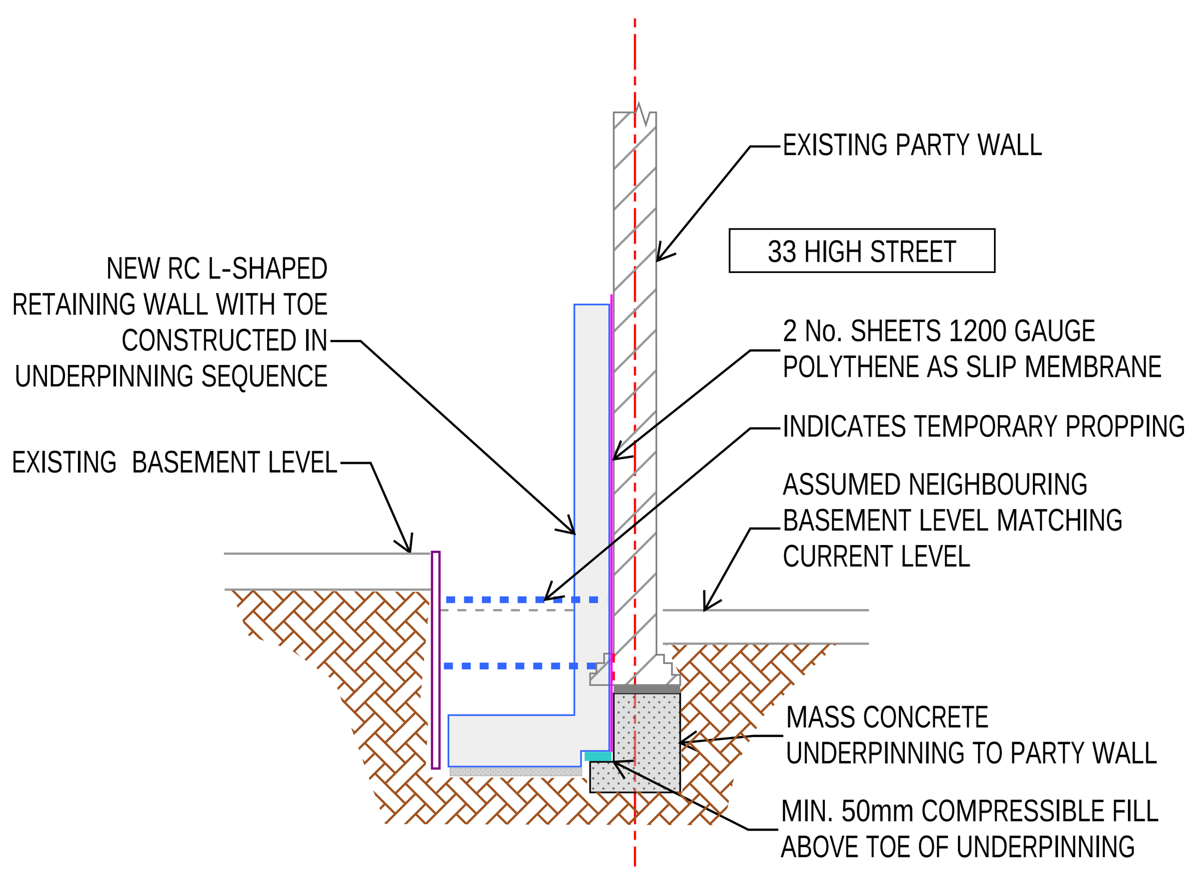
PROPOSED CONSTRUCTION SEQUENCE

STAGE 0:
- SITE SET UP

STAGE 1:
- USING HIT AND MISS UNDERPINNING SEQUENCE (REFER TO PLAN), DIG DOWN TO UNDERSIDE OF CORBEL LEVEL IN PINS MARKED "1".
- INSTALL MASS CONCRETE UNDERPIN TO 75mm BELOW UNDERSIDE OF EXISTING FOUNDATION. PROVIDE SHEAR KEY TO ADJACENT PINS.
- INSTALL DRY MORTAR PACK WITH NON-SHRINK ADDITIVE TO UNDERSIDE OF EXISTING FOUNDATION, WELL RAMMED IN.
- CAST RC WALL SECTIONS AND WALL TOE WITH CONTINUITY REBAR FOR FUTURE CONNECTION TO BASEMENT SLAB AND NEIGHBOURING SECTIONS
- BACKFILL EXCAVATION USING WELL COMPACTED GRANULAR MATERIAL OR LEAVE EXCAVATION SUPPORT IN PLACE
- REPEAT FOR REMAINING PINS, IN SEQUENCE INDICATED

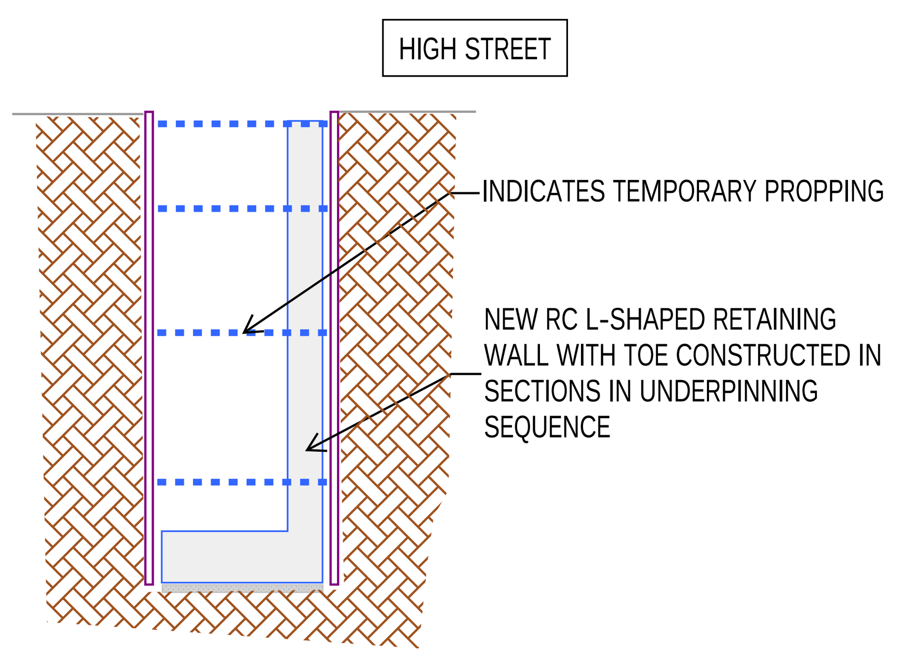
STAGE 2:
- EXCAVATE DOWN USING RC UNDERPINNING SEQUENCE, INSTALLING TRENCH SHEETING AND STRUTS/WALING BEAMS TO SUPPORT EXCAVATION. EXACT SIZE OF PINS TO SUIT CONTRACTOR'S TEMPORARY WORKS DESIGN.
- CAST RETAINING WALL SECTIONS AND WALL TOE WITH CONTINUITY REBARS FOR FUTURE CONNECTION TO BASEMENT SLAB

STAGE 3:
- EXCAVATE GROUND LEVEL WITHIN BASEMENT TO UNDERSIDE OF UPPER LEVEL OF HORIZONTAL PROPS (TO CONTRACTOR'S TEMPORARY WORKS DESIGN) AND INSTALL HORIZONTAL PROPS
- EXCAVATE TO UNDERSIDE OF LOWER LEVEL OF HORIZONTAL PROPS AND INSTALL PROPS BEFORE EXCAVATING TO FORMATION LEVEL
- PULL OUT CONTINUITY BARS FROM RETAININ WALL TOES AND CONSTRUCT REMAINING BASEMENT SLAB BETWEEN. THIS PROVIDES PERMANENT LOWER LEVEL HORIZONTAL PROP
- CONSTRUCT GROUND FLOOR SLAB TO PROVIDE PERMANENT HORIZONTAL PROP TO TOP OF RETAINING WALLS AND REMOVE TEMPORARY PROPPING



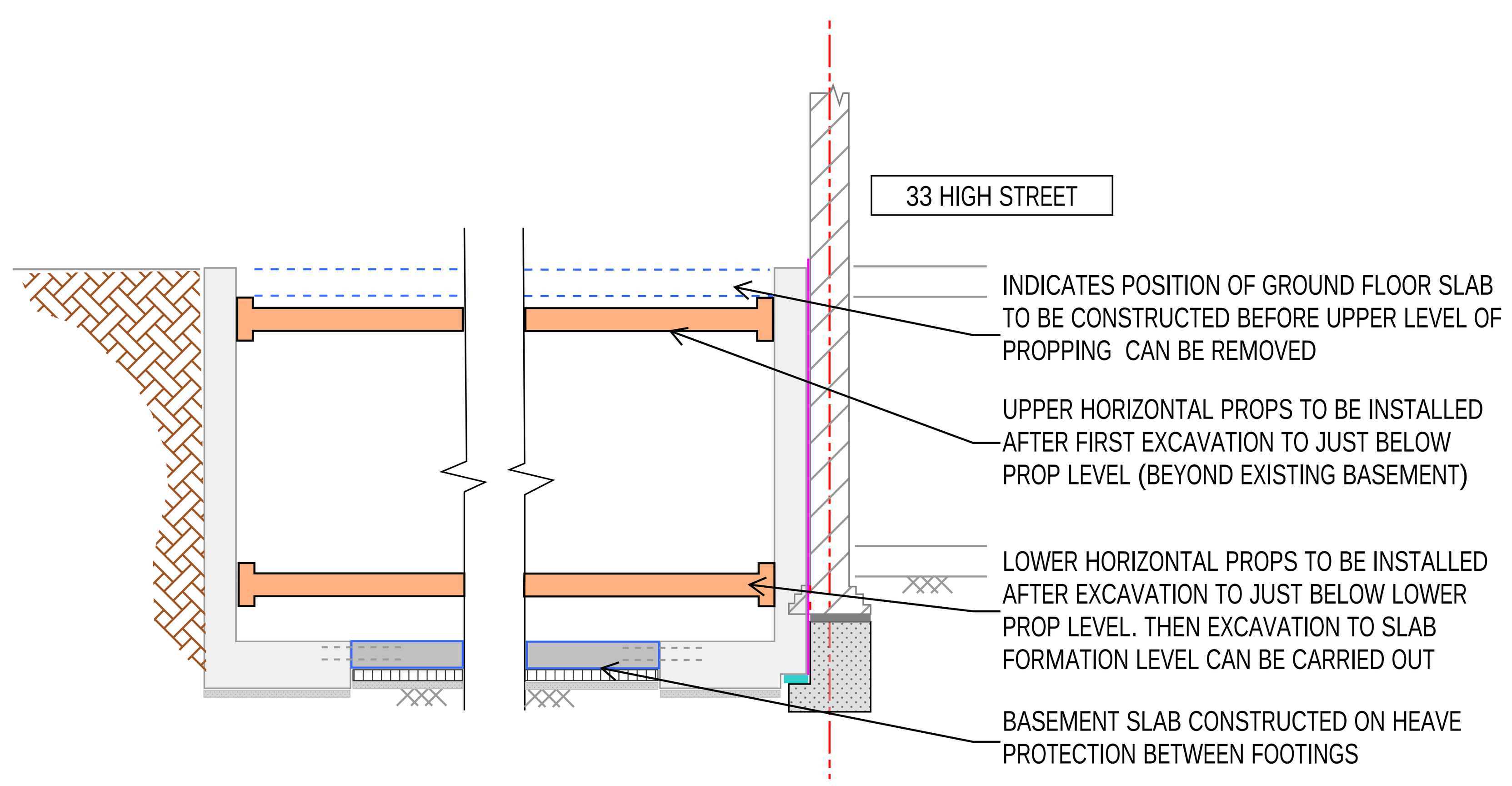
STAGE 1 - SECTION A

N.T.S.



STAGE 2 - SECTION B

N.T.S.



STAGE 3 - SECTION

N.T.S.

NOTE:

ALL UNDERPIN EXCAVATIONS TO BE PROVIDED WITH FULL TEMPORARY SUPPORT IN FORM OF TRENCH SHEETS, WALERS AND STRUTS. INSTALL WALERS AND STRUTS AT EVERY 1m VERTICALLY AND HORIZONTALLY. REPEAT UNTIL THE REQUIRED DEPTH IS GAINED.

CONTRACTOR TO CONSIDER WATER TABLE AND ALLOW FOR DE-WATERING OF EXCAVATIONS. WATER TABLE NOTED AS BELOW LEVEL OF BASEMENT, BUT SEASONAL CHANGES MAY CAUSE THIS TO VARY.

rev	date	description	by	checked
P01	2023.07.19	ISSUED FOR INFORMATION	AB	MW

PROJECT TITLE: 29-31 HIGH STREET, HAMPTON WICK - BASEMENT IMPACT ASSESSMENT

CLIENT: MR & MRS FROST

PROJECT No: E0811 | DRAWN: AB | CHECKED: MW

DRAWING TITLE: PROPOSED CONSTRUCTION SEQUENCE

DRAWING No: E0811-EEE-00-ZZ-DR-S-9050

STATUS DESCRIPTION: SUITABLE FOR INFORMATION

REV: P01 | SCALE: As Indicated

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