N.M.N PARTNERSHIP LMITED

Consulting Civil & Structural Engineers

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26 Amyand Park Road, Twickenham, TW1 3HE

CONSTRUCTION METHOD STATEMENT

November 2024

Project Ref: Nov/23 227

REVISION HISTORY

Rev	Purpose	Date	Issued By	Approved
Rev 0	Initial report	14.11.24	NM	NM
Rev 1	Updated	06.12.24	NM	NM

CONTENTS

CONSTRUCTION METHOD STATEMENT

This Construction Method Statement is produced for submission to the London Borough of Richmond planning department for planning application purposes only and should not be used for any other purposes, e.g. Party Wall Awards.

SCOPE OF WORKS

A new basement will be excavated under the entire footprint of the property, with a 3m front extension and a 1m rear extension. A lightwell will be included at the bay window for natural light and ventilation, while the rear will feature walk-on glass panels instead of lightwells, allowing natural light to penetrate the basement below. The basement will house a 2m wide, 12.5m long swimming pool, as well as a utility room and storage areas.

The ground floor will feature an RC slab with openings for the lightwell, stairs, and walk-on glass panels, providing lateral support to retaining walls. The basement walls, including those for the swimming pool, will be constructed using RC retaining walls designed to resist earth pressures and surcharges. Waterproofing measures will ensure the basement and pool remain dry.

Key design elements include reinforced edges for openings in the slab, waterproofed concrete for the pool and retaining walls, and robust, safe walk-on glass installations for the rear area. The construction will comply with building regulations, ensuring structural integrity, effective ventilation, and natural lighting.

DESCRIPTION OF THE PROPERTY AND ADJOINING PROPERTIES

The property is a three-storey mid-terrace house, constructed with masonry walls, which provide a solid foundation for the structure. The foundations themselves are made from corbeling bricks, a traditional method where bricks are laid in a stepped, overhanging pattern to support the weight of the building. This type of foundation is common in older properties and is designed to bear the load of the walls above while ensuring stability over time.

Internally, the floors across all levels are timber, contributing to the house's period charm and warmth. The roof is supported by timber rafters, forming a traditional roof structure with a lean-to design at the rear of the house, which adds a unique architectural feature. An infill side extension has been added at the rear of the property, featuring skylights that help bring natural light into the space, creating a bright and airy atmosphere.

The property appears to be in sound structural condition. The adjoining properties are of similar construction, and a visual inspection suggests that they too are in good condition. The overall stability of the property and its neighboring buildings indicates that they are well-maintained and secure, offering a solid foundation for any future development or renovation plans.

SOIL CONDITIONS

This Construction Method Statement is supported by our previous successful subterranean developments in the vicinity of the property. The ground conditions in the area consist of Kempton Park Gravels (clayey sands and gravel) overlaying the London Clay. The depth to the London Clay is approximately 6.5 meters below ground level.

Our prior excavation work reached similar depths to those proposed for this project, and we can confirm that no groundwater was encountered during these operations. The new basement design will limit ground bearing pressure to 150 kN/m², ensuring that the existing geological conditions at the proposed depth can adequately support the new imposed loads.

Although no groundwater was encountered during the previous excavations, the basement will be designed in accordance with the recommendations of BS8102:1990, "Protection of structures against water from the ground." Specifically, Clause 3.4 indicates that a water table should be assumed to be at 1.0 meter below ground level, which will be considered in the basement's waterproofing and drainage design.

Additional Measures:

In addition to the water management measures already outlined, the new pumps in the basement will be fitted with non-return valves to prevent flooding in the event of pump failure or blockage. To minimize the discharge to the existing sewers, water-efficient fixtures and fittings will be installed throughout the basement to reduce the overall flow.

Construction Drawings:

Please refer to **Drawings SK 102** and the Appendices for the underpinning layout, sequencing, and sections related to the party walls of the property.

Construction Sequence:

1. **Excavation Start:** Excavation will begin at the spine wall of the property, progressing towards the rear and the new 3-meter rear extension. A light well will be incorporated at the rear, passing the 3-meter extension.

2. **Temporary Ground Floor Removal:** A portion of the existing timber ground floor will be temporarily removed to allow for the loading of excavated material onto skips using a conveyor belt system. This will provide access to the basement area.

3. **Conveyor Belt Setup:** A conveyor belt will be set up through the front room and window to move the spoil from the excavation to a skip placed on the driveway for disposal.

4. **Underpinning Sequence:** The existing property will be underpinned using a 1, 3, 5, 2,

and 4 "hit and miss" sequence, as shown in **Drawing NMN/SK 102**. This underpinning sequence will ensure proper support during the basement excavation process.

5. **Horizontal Propping for Underpins:** Horizontal propping will be required at the toe and high level of the underpins until the basement slab is completed, and the underpinning pins gain the required strength.

6. **Removal of Ground Floor Elements:** As excavation progresses, the remaining ground floor joists and concrete slabs will be broken out and removed. Any existing foundations encountered during excavation will also be removed to allow space for the new basement.

7. **Demolition of Internal Walls and Floor Support:** Internal walls will be demolished, and the floors above will be temporarily supported with steel beams and props to ensure stability during the excavation and construction phases. Temporary support will be provided for the floors above using a top-down sequence. The rear wall will receive moment frame support via temporary needles and props at 800mm centers, supported off the newly cast underpin toes (see **SK101**).

8. **Rear Wall Excavation and Retaining Wall Construction:** Once the rear wall is exposed, excavation and installation of the reinforced concrete (RC) retaining wall for the rear extension will follow. This will be carried out in a similar manner to the underpinning of the party walls, with a 1-meter wide section excavated at a time. Temporary lateral supports will be used during construction.

9. **Construction of Basement Slab:** Once the retaining walls with toes are completed, a 250mm thick suspended basement slab will be constructed, spanning between the retaining wall toes as detailed.

10. **Ground Floor RC Slab for Patio:** After the rear extension basement is formed with the temporary supports in place, the ground floor RC slab will be constructed to form the patio area.

11. **Rear Bi-Fold Door Opening and Roof Installation:** The rear bi-fold door opening will be supported by a torsion beam, as detailed in the construction drawings. A new timber roof with a skylight will be installed as per the design.

12. **Installation of Second Level of Props:** Once excavation reaches approximately 500mm above the proposed basement level, a second level of horizontal props will be installed if required by the design.

13. **Excavation to Formation Level:** Excavation will continue down to the formation level, as specified in the project design.

14. **Drainage Installation:** Below-slab drainage systems for both foul and ground water,

as well as sumps and pumps, will be installed. The pumps will discharge water into a silt tank, and once approved, it will be directed into the existing sewer system at the front of the property.

15. **Construction of New Basement RC Slab:** The new ground-bearing basement RC slab will be cast using A393 mesh at both the top and bottom of the slab. The extension basement slab will be cast between the retaining wall toes, with bent-up bars from the toes.

16. **Removal of Horizontal Propping:** Once the new basement slab has gained sufficient strength, horizontal propping across the basement level will be removed. However, the propping below the ground floor will remain in place until the ground floor slab is cast and has cured.

17. **Excavation for Swimming Pool:** Excavation for the swimming pool will commence along the No 28 party wall line, utilizing the underpinning method with toes, similar to the basement underpinning. Raking props will be installed from the basement level off the toe, while the opposite side of the pool will be cast as one continuous retaining wall.

18. **Drained Cavity Layer Installation:** After the basement slab has cured, a drained cavity layer will be installed on both the slab and the walls of the basement.

19. **Insulation Installation:** A layer of insulation will be placed on top of the drained cavity layer on the slab and along the walls in front of the drained cavity layer.

20. **Screed Layer for Finished Floor:** Finally, a screed layer will be applied to form the finished basement floor, providing a level surface for use and finishing.

This sequence ensures a systematic approach to excavating, underpinning, and constructing the basement, while maintaining the integrity of the existing structure and minimizing disruption during the process.

POTENTIAL IMPACT ON THE PROPERTY AND ADJOINING PROPERTIES

The proposed basement will be formed using an underpinning method, designed to ensure the structural integrity of both the existing property and neighbouring structures. The underpinning will be carried out in sections, with each pin being no wider than 1000mm to minimize disruption to the surrounding ground and foundations. Additionally, to further reduce the risk of ground movement, no adjacent underpins will be constructed within a 72-hour period from the time of dry packing between the top of the pin and the underside of the existing foundation. This phased approach ensures that the ground has sufficient time to settle and stabilise between operations, thereby minimizing any potential impact on the surrounding area.

By adopting this method of construction, the amount of potential ground movement is significantly reduced, which in turn minimizes the effects of settlement on both the property undergoing the works and any adjoining structures. The careful sequencing and controlled technique employed in this underpinning process are essential in preventing any unintended consequences, such as subsidence or structural instability.

Furthermore, the proposed works, if executed properly and in strict accordance with the appointed Engineer’s detailed plans, guidelines, and procedures, will pose no significant threat to the structural stability of the property or the surrounding properties. The design has been specifically tailored to ensure that all potential risks are mitigated, and appropriate safeguards are put in place. With expert oversight and adherence to best practices, the works will proceed without compromising the safety and stability of the existing structures, ensuring a successful outcome for all parties involved.

POTENTIAL IMPACT ON EXISTING AND SURROUNDING UTILITIES, INFRASTRUCTURE AND MAN – MADE CAVITIES

Any local services that are located on the property’s land will be carefully maintained throughout the construction process. In cases where it becomes necessary, these services will be rerouted to ensure their continued functionality and to prevent any disruption. While the exact location of these services will not be fully known until the works begin, we anticipate that any potential impact on these services will be negligible, as they will be properly managed and maintained during the course of construction.

In the event that it is required to relocate or divert any utilities, the Contractor and the Design Team will be legally bound to notify the relevant utility owners in advance of undertaking any work. This notification is crucial as it allows the utility owners to assess the potential impact of the works on their infrastructure. Following this assessment, the utility owner will have the authority to either approve or deny the proposed alterations based on their findings. This ensures that all utilities are properly managed and that their operation is not compromised during construction.

Furthermore, it is important to note that there are no known man-made cavities, such as tunnels, in the vicinity of the proposed basement. This significantly reduces the likelihood of encountering unexpected underground voids or structures during excavation. The thorough planning and communication with utility owners, combined with the absence of known underground anomalies, ensure that the construction works can proceed without causing unforeseen complications related to services or sub-surface conditions.

POTENTIAL IMPACT ON DRAINAGE, SEWAGE, SURFACE AND GROUND WATER LEVELS AND FLOWS

All existing drainage and sewage connections will be meticulously maintained throughout the duration of the construction works, ensuring there is no disruption to the functionality of these vital systems. The proposed works are designed with minimal impact on the current infrastructure, and the property will remain a single residential unit throughout the construction process. As such, there will be no significant alteration to the existing drainage and sewage systems, and the overall discharge of wastewater to these systems will remain virtually unchanged. Consequently, the impact on the foul drainage system is expected to be minimal, with no major modifications or strain placed on it as a result of the basement construction.

Surface water management will also remain unaffected, as the scope of the proposed works is entirely subterranean, with no new "hard surfaces" being introduced at ground level. This means that the existing surface water drainage systems will continue to operate as originally designed, with no additional runoff generated from the proposed development. The absence of new impermeable surfaces ensures that there will be no significant increase in surface water discharge, thus minimizing the risk of localized flooding or overloading of the existing drainage systems.

The basement will be constructed at a level significantly above the local groundwater table, ensuring that the works will not impact or disrupt the natural flow of groundwater in the surrounding area, both during and after the construction process. The depth of the basement excavation has been carefully planned to avoid interference with any groundwater flows, and special attention will be given to ensuring that groundwater does not enter the excavation. In the rare event that any groundwater is encountered during excavation, measures will be taken to manage the situation appropriately. However, the pumping out of water will not be permitted on site. Instead, the procedure for managing any unexpected groundwater will involve the digging of containment holes to hold and manage the water until it can be safely addressed. This approach will ensure that the excavation process is managed responsibly, without any adverse impact on the surrounding environment or groundwater flow.

To ensure the potential risks associated with the construction works are fully understood and mitigated, a comprehensive Ground Investigation and Basement Impact Assessment has been carried out by Jomas Environmental Engineers. This thorough investigation has provided valuable insights into the local subsurface conditions, allowing for a detailed assessment of how the proposed works may interact with the existing infrastructure and environment. The findings from this assessment have been incorporated into the design and construction plans, ensuring that all necessary precautions and solutions are in place to prevent any unforeseen complications. The proactive approach taken in addressing groundwater, drainage, and other subsurface conditions guarantees that the construction will proceed smoothly, with minimal impact on the surrounding area and infrastructure.

Prepared By

Nathan Masilamani BEng MSc ICIOB
Senior Structural Engineer

NMN Partnership Ltd

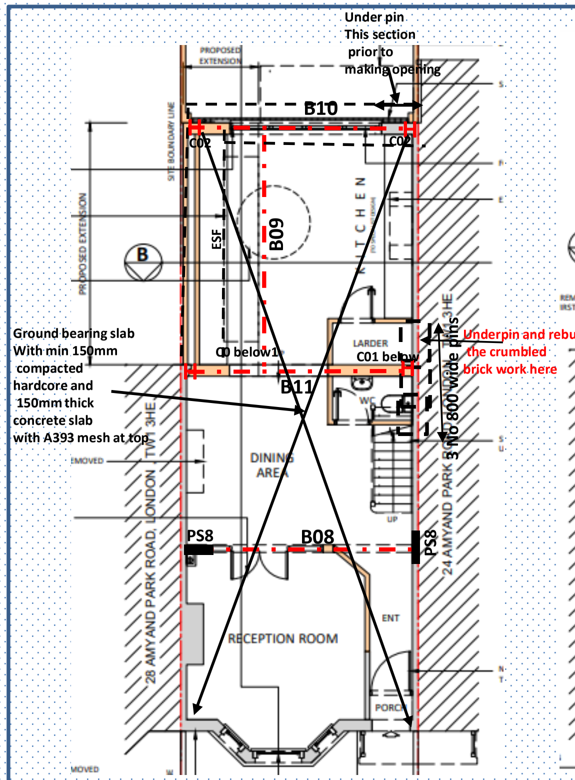
APPENDICES

The following appendices are included with this report.

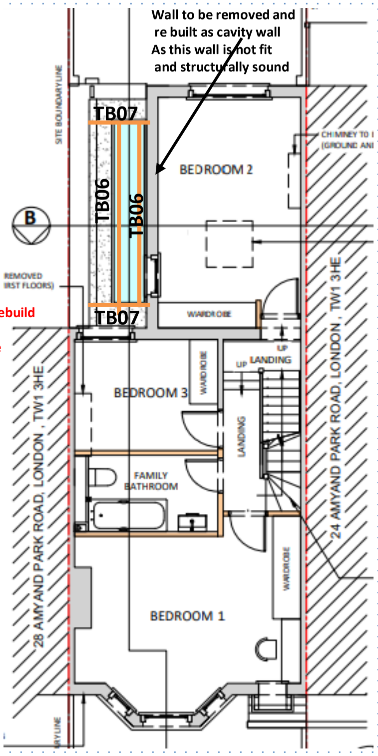
Appendix A - NMN Partnership Proposed Drawings and Construction Sequence

Appendix B - Soil Investigation and BIA Report by Jomas Environmental Engineers

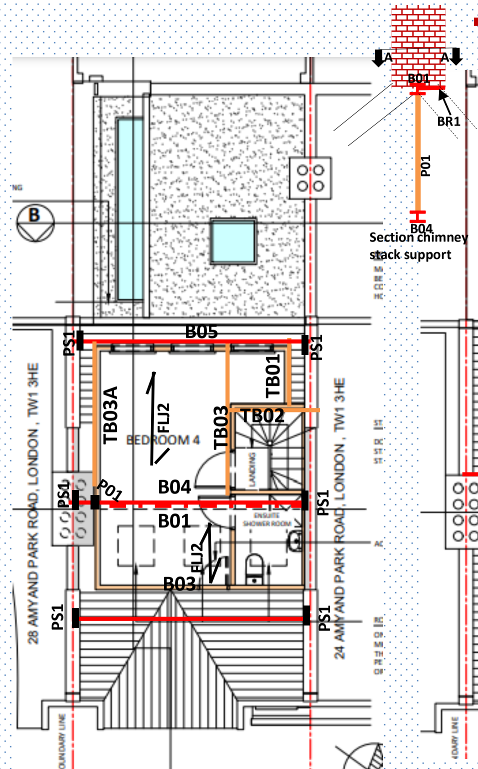
Appendix C – NMN Partnership Calculations



Proposed Ground Floor Layout

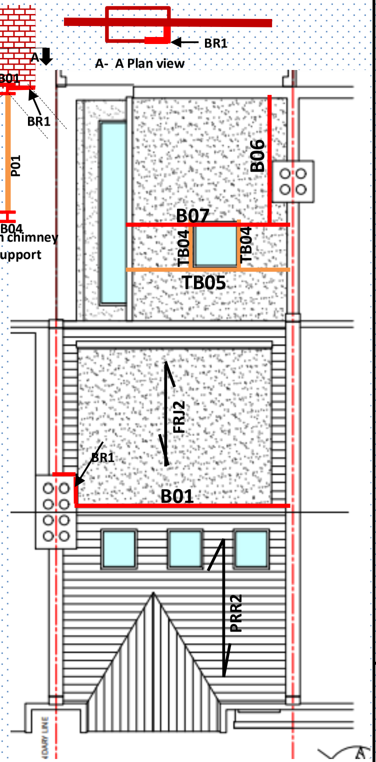


Proposed First Floor Layout



Proposed Loft Floor Layout

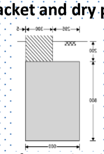
26 AMYAND PARK ROAD, LONDON , TW1 3HE



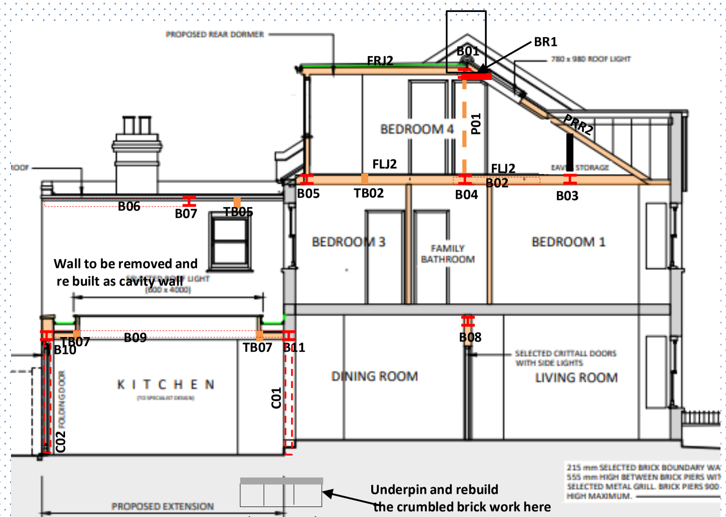
Proposed Roof Layout

Key

B01-152x152x37 UKC- 100mm bearing on 250x100x10 MS Plate and connected to P01 with 4No M12 bolts
B02- 127x76x13 UKB- connected to B03 and B04 with 6No M12 bolts and 2No 70x70x5 cleats
B03/B04/B05- 203x203x46 UKC- 100mm bearing on PS1
B06- 127x76x13 UKB- 100mm bearing on wall and connected to B07 with 4No M12 bolts(underslung if required)
B07-127x76x13 UKB- 100mm bearing on wall
B08- 152x152x37 UKC- 100mm bearing on PS8 by the stair and 200mm bearing on PS8 on the left
B09= 203x203x86 UKC- connected to B11 and B10 with 6No M16 bolts and 2No 100x100x10 cleats with top notch
B10-203x203x86 UKC cranked beam- Connected to C02 with 3 rows of 2 M24 bolts
B11-203x203x46 UKC–Connected to C01s with 3 rows of 2 M20 bolts
BR1- 150x75x18 UKPFC L bracket to support remaining stack – 100mm bearing on wall and
connected to B01 with 6mm end plate and 2No M12 bolt, place 6mm thick plate over the bracket and dry pack
FLJ2- 47x195 C24 timber joists at 400mm c/c with noggins at 3rd of span
FRJ2- 47x150 C24 timber joists at 400mm c/c with noggins at mid span and ends
PRR2- 47x125 C16 timber rafters at 400mm c/c with noggins at 3rd of span
PS1- 500x100x215 concrete pad stone
PS8-440x100x215 concrete pads tone
C01/C02- 203x203x46 UKC with 250x250x20 base plate and 400x250x15 Cap plate
P01- 1No 97x150 C24 C24 timber post- connected to B02 and B01 with 2No 70x70x5 cletas and 4No M12 bolts
TB01/TB02/TB04-1No 75x150 C24 timber
TB03- 2no 75x200 C24 timbers bolted together with M12 bolts at 500mm c/c
TB03A- 2No 47x200 C24 timbers bolted together with M12 bolts at 500mm c/c
TB05/TB06- 2no 75x150 C24 timbers bolted together with M12 bolts at 375mm c/c
TB07-2No 47x100 C24 timbers bolted together with M12 bolts at 250mm c/c
ESF- 600 wide 800 deep mass concrete strip footing founded at 1m below ground with max eccentricity of 145mm to
C/L of strip footing



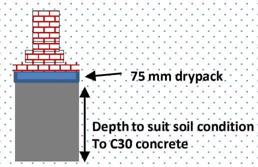
Section of Eccentric strip footing



Section

Notes:

1. This drawing is to read in conjunction with Architects drawing.
2. All steel to be grade S275, shot blasted and painted with 2coats of red oxide and 2 parts of phosphate epoxy
3. All bolts to be grade 8.8
4- Commencing work prior to Building regulation approval is the clients risk



Section of under pinning

Rev	Change	Date
A	Eccentric strip footing added	11.11.23
B	Notes and under pinning details added	15.04.24
C	CHIMNEY SUPPORT DETAILS ADDED	08.05.24
D	B11 and B10 changed, C01/C02 added	10.07.24
E	B10, B11, B09 changed , co1 moved	17.07.24

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Client:
Mariusz

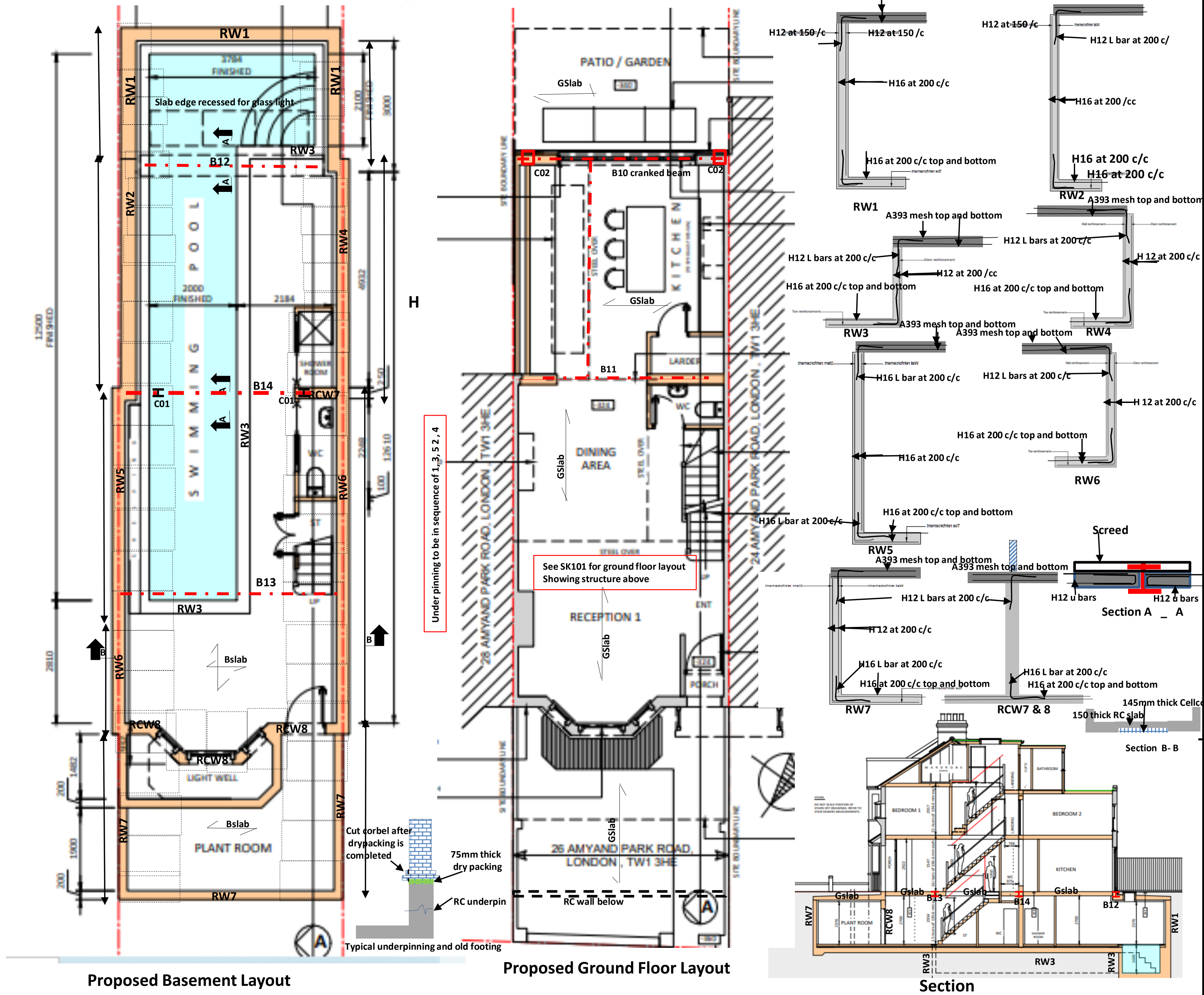
Project:
26 Amyand Park Road
Twickenham TW1 3HE

Title:
Proposed Structural Layout

Drawn: NM
Date: 10.11.2023

Scale: NTS
Job No: 23 227
Drawing No: SK 101 Rev E

Appendix A



Notes:

1. This drawing is to read in conjunction with Architects drawing.
2. All steel to be grade S275, shot blasted and painted with 2coats of red oxide and 2 parts of phosphate epoxy
3. All bolts to be grade 8.8
4- Commencing work prior to Building regulation approval is the clients risk
5. All temporary support during the demolition and construction is the responsibility of the contractor
6. All steel fabrication drawings to be submitted to the Engineer for Perusal and approval prior to construction

For Approval

RW1/RW2- 300 thick wall and toe - with H16 main bars and H12 lacer bars
RW3- 200 thick wall and toe with H12 at 200 c/c
RW4- 300 thick wall and toe with H16 at 150 c/c and H12 at 200 c/c
RW5- 300 thick wall and toe with H16 at 16 at 200 c/c
RW6- 300 thick wall and toe with H16 at 150 /c and H12 at 200 c/c
RW7/RCW7- 250 thck wall and 300 thick toe with H12 at 200 c/c
RCW8- 300 thick wall with H12 at 200 c/c
Bslab- 150 thick slab with A393 mesh top and bottom
Gslab- 200 thick RC slab with A393 mesh top and bottom
B12/B13- 203x203x86 UKC-100 mm bearing on RCwall
B14-203x203x71 UKC- 100mm bearing on RC wall
For ground floor high level beams see SK101

Rev	Change	Date
B	B14 changed	17.07.24

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Project:
26 Amyand Park Road
Twickenham TW1 3HE

Title:
Proposed Structural Layout
And Sections

Drawn: NM
Date: 04.05.2024

Scale: NTS
Job No: 23 227
Drawing No: SK 102 Rev B

JOMAS ENGINEERING ENVIRONMENTAL LAND REMEDIATION

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GROUND INVESTIGATION & BASEMENT IMPACT ASSESSMENT REPORT

26 AMYAND PARK ROAD
TWICKENHAM
TW1 3HE

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Jomas Associates Ltd Registered in England and Wales No. 7095350

Report Title: **Ground Investigation & Basement Impact Assessment for 26 Amyand Park Road, Twickenham, TW1 3HE**

Report Status: **Draft**

Job No: **P5802J3027/HAH**

Date: **25 November 2024**

QUALITY CONTROL - REVISIONS

Version	Date	Issued By	Comment

Prepared by: JOMAS ASSOCIATES LTD For: 05 GROUP LTD

Prepared by	Reviewed by	Approved by
Hamza Hashi MEng (Hons)	Josephine Whitehead MSci (Hons), FGS	Derek Grange BSc, MSc, CGeol, FGS, RoGEP - Specialist
Geotechnical Engineer	Senior Geotechnical Engineer	Senior Principal - Geotechnics
....	....	....

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APPENDIX 4 – CHEMICAL LABORATORY TEST RESULTS

APPENDIX 5 – GROUNDWATER MONITORING RESULTS

EXECUTIVE SUMMARY

05 Group Ltd commissioned Jomas Associates Ltd to prepare a Geotechnical Ground Investigation and Basement Impact Assessment at the site located at 26 Amyand Park Road, Twickenham, TW1 3HE .

The principal objectives of the study were as follows:

- To establish the geotechnical conditions pertaining to the site;
- To assess the data from the investigation to inform preliminary design advice with respect to foundation design, concrete specification and excavation stability.
- To undertake a Basement Impact Assessment (BIA) based on the methodologies outlined in London Borough of Richmond on Thames “Planning Advice Note: Good Practice Guide on Basement Developments” (2015) and “Basement Assessment User Guide” (2021), with additional reference to the guidance given in the London Borough of Camden document “Camden Planning Guidance Basements” (CPGB) (January 2021).

It should be noted that the table below is an executive summary of the findings of this report and is for briefing purposes only. Reference should be made to the main report for detailed information and analysis.

Site Information	
Current Site Use	Two-storey residential property undergoing refurbishment
Proposed Site Use	The proposed development for this site is understood to comprise a rear-side extension and creation of basement beneath the entire building footprint and extending partially beneath the front garden.
Summary of Stage 1 & 2 BIA	A Stage 1 & 2 Basement Impact Assessment report has been produced for the site and issued separately (Jomas, June 2024). A brief overview of the findings is presented below. Reference should be made to the full report for detailed information. On the earliest available map (1865), the site is shown as largely vacant except for a small building shown to be extending into the site from the north-west. By the map dated 1912, the site is shown to be situated within a row of terraced housing. No observational changes then occur to the site until the most recent map dated 2024. Historically, the surrounding area has comprised mainly residential properties, with the only significant land use identified as a railway 80m north of site and the River Crane beyond at approximately 176m from site. The British Geological Survey indicates that the site is directly underlain by superficial deposits of the Langley Silt Member. Superficial deposits of the Kempton Park Gravel Member are anticipated to underlie the Langley Silt Member. These superficial deposits overlie solid deposits of the London Clay Formation. The underlying Langley Silt Member and the London Clay Formation are identified as Unproductive. The Kempton Park Gravel Member is reported (off-site) as a Principal Aquifer. A review of the EnviroInsight Report indicates that there are no Environment Agency Zone 2 or Zone 3 flood zones within 250m of the site.

<table>
<tr><th colspan="2">Site Information</th></tr>
<tr><td></td><td>The River Crane is reported 176m north-west.
The screening and scoping assessments concluded the following:
• A ground investigation was recommended to confirm the ground conditions and groundwater levels (if any) beneath the site
• The ground investigation should also determine the presence of Made Ground and/or clay. Atterberg Limits of the underlying clay should be determined by the ground investigation to establish shrink/swell potential
• The proposed basement will underlie the existing building footprint/ hardstanding; there will be no significant change in surface water run-off
• As SuDS will be required by NPPF, PPG and LLFA policy requirements, where practicable, the remaining hard surfaces will likely be replaced with permeable paving. This will ensure that the proposed development will not increase the potential risk of flooding
• A SuDS/drainage strategy report was recommended
• A Ground Movement Assessment was considered prudent, but may not be a requirement of the London Borough of Richmond upon Thames</td></tr>
<tr><th colspan="2">Ground Investigation</th></tr>
<tr><td>Scope of Works</td><td>The ground investigation was undertaken on 10 October 2024, and consisted of the following:
• 1No cable percussive borehole, drilled to a depth of 10m below ground level (mbgl), with associated in-situ testing and sampling
• 1No groundwater monitoring well, installed to 7.5mbgl
• Laboratory analysis for chemical and geotechnical purposes
• 1No return visits to monitor groundwater levels has been carried out, and 1No further visit is due to be completed in February 2025</td></tr>
<tr><td>Ground Conditions</td><td>The results of the ground investigation revealed a ground profile comprising Made Ground to a depth of 1.9mbgl, underlain by granular deposits of the Kempton Park Gravel Member to 7.4mbgl, underlain by cohesive deposits of the London Clay Formation to a depth in excess of 10mbgl.
During the investigation, groundwater was reported within the borehole at a depth of 6.2mbgl, and by the time the drilling had concluded, was sat at a level of 6.45mbgl.
During return monitoring, groundwater was reported at 6.53mbgl. A second visit is due to take place in February 2025 and this report will be updated.</td></tr>
<tr><td>Foundations</td><td>Based upon the information obtained to date, it is considered that a cast in-situ cantilever retaining wall formed at approximately 3.5m below the existing ground level within the Kempton Park Gravel Member could be designed with an allowable bearing capacity of 200kPa. Total and differential settlements should be contained within tolerable limits.
It is unlikely that the foundations would need to be deepened further due to NHBC building near trees requirements.</td></tr>
</table>

Site Information	
Sulphates	Based on the results of chemical testing, for foundations formed with the Kempton Park Gravel Member, the required concrete class for the site is DS-1 assuming an Aggressive Chemical Environment for Concrete classification of AC-1 in accordance with the procedures outlined in BRE Special Digest 1. If foundations are to be formed within the London Clay Formation, higher concrete classes are considered necessary, as detailed in Section 6.4.
Ground Floor Slabs	If a cantilever retaining wall is utilised, then a ground bearing floor slab could be used. If a piled option is utilised then suspended floor slabs will be required.
Excavations	Temporary excavations are unlikely to remain stable and some form of temporary support or battering back to a safe angle and dewatering are likely to be required. Subject to seasonal variations, surface water/groundwater encountered during site works could likely be dealt with by conventional pumping from a sump used to collate waters.
Basement Impact Assessment	
Conclusions	The overall assessment of the site is that the creation of a basement for the proposed development should not adversely impact the site or its immediate environs, providing measures are taken to protect surrounding land and properties during construction. The proposed basement excavation will be within 5m of a public pavement. It is also laterally within 5m of neighbouring properties. Unavoidable lateral ground movements associated with the basement excavations must be controlled during temporary and permanent works so as not to impact adversely on the stability of the surrounding ground and any associated services. During the construction phase careful and regular monitoring will need to be undertaken to ensure that the neighbouring properties are not adversely affected. This may mean that structures will need to be suitably propped and supported.
Recommended Further Works	
Recommendations	• A drainage/SuDS strategy report is recommended to outline how betterment of the flood risk will be achieved through development of the site • A Ground Movement Assessment is also considered prudent, though may not be a specific requirement of the London Borough of Richmond upon Thames

1 INTRODUCTION

1.1 Terms of Reference

1.1.1 05 Group Ltd ("The Client") has commissioned Jomas Associates Ltd ('Jomas'), to undertake an investigation of the geotechnical factors pertaining to the proposed redevelopment and to prepare a Basement Impact Assessment at a site referred to as 26 Amyand Park Road, Twickenham, TW1 3HE .

1.1.2 To this end a Stage 1 & 2 (Screening and Scoping) Basement Impact Assessment has been produced for the site and issued separately (Jomas, June 2024), followed by an intrusive investigation (detailed in this report).

1.1.3 Details of the previous report are provided below in Table 1.1:

Table 1.1: Previous Reports - Jomas

Title	Author	Reference	Date
Stage 1 & 2 Basement Impact Assessment (Screening and Scoping) for 26 Amyand Park Road, Twickenham, TW1 3HE	Jomas Associates Ltd	P5802J3027/HAH	20 June 2024

1.1.4 The intrusive investigation was undertaken in accordance with Jomas' proposal dated 17 September 2024.

1.2 Proposed Development

1.2.1 The proposed development for this site is understood to comprise a rear-side extension and creation of basement beneath the entire building footprint and extending partially beneath the front garden.

1.2.2 Plans of the proposed development are included in Appendix 1.

1.2.3 For the purpose of geotechnical assessment, it is considered that the project could be classified as a Geotechnical Category (GC) 2 site in accordance with BS EN 1997.

1.3 Objectives

1.3.1 An intrusive investigation is proposed to establish geotechnical conditions pertaining to the site.

1.3.2 The data from the geotechnical investigation is to form the basis of preliminary design advice with respect to foundation design, concrete specification and excavation stability.

1.3.3 A Basement Impact Assessment will assess the potential impacts that the proposal may have on ground stability, the hydrogeology and hydrology on the site and its environs.

1.4 Scope of Works

1.4.1 The following tasks were undertaken to achieve the objectives listed above:

- An intrusive investigation to assess the underlying ground conditions;
- Undertaking of laboratory chemical and geotechnical testing upon samples obtained;
- Return groundwater monitoring;
- Carrying out a Basement Impact Assessment (BIA);
- The compilation of this report, which collects and discusses the above data, and presents an assessment of the site conditions, conclusions and recommendations.

1.5 Scope of Basement Impact Assessment

1.5.1 The site lies within the remit of the London Borough of Richmond upon Thames. The council has published the documents "Planning Advice Note: Good Practice Guide on Basement Developments" (2015) and "Basement Assessment User Guide" (2021). These documents provide detail on the issues relevant to basements within London Borough of Richmond upon Thames and describe how these issues should be assessed.

1.5.2 Jomas has also used the guidance given in the London Borough of Camden document "Camden Planning Guidance Basements" (CPGB) (January 2021) as this is generally accepted as the best available guidance on the practicalities regarding how to undertake a BIA.

1.5.3 Jomas' BIA covers most items required under CPGB, with the exception of;

- Plans and sections to show foundation details of adjacent structures.
- Programme for enabling works, construction and restoration
- Evidence of consultation with neighbours
- Ground Movement Assessment (GMA), to include assessment of significant adverse impacts and Specific mitigation measures required, as well as a confirmatory and reasoned statement identifying likely damage to nearby properties according to Burland Scale
- Construction Sequence Methodology
- Proposals for monitoring during construction.
- Drainage assessment

1.5.4 This Jomas BIA also takes into account the Campbell Reith pro forma BIA produced on behalf of and published by the London Borough of Camden as guidance for applicants to ensure that all of the required information is provided.

1.5.5 A number of the requirements set out in the London Borough of Camden document CPGB will need to be addressed in a construction management plan, this stage is not within the scope of work that Jomas Associates have been commissioned.

1.6 Supplied Documentation

1.6.1 Jomas Associates have not been supplied with any previously produced reports at the time of writing this report.

1.7 Limitations

1.7.1 Jomas Associates Ltd ('Jomas') has prepared this report for the sole use of 05 Group Ltd, in accordance with the generally accepted consulting practices and for the intended purposes as stated in the agreement under which this work was completed. This report may not be relied upon by any other party without the explicit written agreement of Jomas. No other third-party warranty, expressed or implied, is made as to the professional advice included in this report. This report must be used in its entirety.

1.7.2 The records search was limited to information available from public sources; this information is changing continually and frequently incomplete. Unless Jomas has actual knowledge to the contrary, information obtained from public sources or provided to Jomas by site personnel and other information sources, have been assumed to be correct. Jomas does not assume any liability for the misinterpretation of information or for items not visible, accessible or present on the subject property at the time of this study.

1.7.3 Whilst every effort has been made to ensure the accuracy of the data supplied, and any analysis derived from it, there may be conditions at the site that have not been disclosed by the investigation, and could not therefore be taken into account. As with any site, there may be differences in soil conditions between exploratory hole positions. Furthermore, it should be noted that groundwater conditions may vary due to seasonal and other effects and may at times be significantly different from those measured by the investigation. No liability can be accepted for any such variations in these conditions.

1.7.4 This report is not an engineering design and the figures and calculations contained in the report should be used by the Structural Engineer, taking note that variations may apply, depending on variations in design loading, in techniques used, and in site conditions. Our recommendations should therefore not supersede the Engineer's design.

2 EXISTING INFORMATION

2.1 Site Information

2.1.1 The site location plan is appended to this report in Appendix 1.

Table 2.1: Site Information

Name of Site	-
Address of Site	26 Amyand Park Road, Twickenham, Richmond upon Thames, TW1 3HE
Approx. National Grid Ref.	516307 173599
Site Area (Approx.)	0.01 hectares
Site Occupation	Residential
Local Authority	London Borough of Richmond upon Thames

2.2 Summary of Stage 1 & 2 Basement Impact Assessment

2.2.1 As detailed in Table 1.1, a report has been produced for the site by Jomas dated 20 June 2024, and issued separately. A brief overview of the findings is presented below. Reference should be made to the full report for detailed information.

Site Setting

2.2.2 On the earliest available map (1865), the site is shown as largely vacant except for a small building shown to be extending into the site from the north-west. By the map dated 1912, the site is shown to be situated within a row of terraced housing. No observational changes then occur to the site until the most recent map dated 2024.

2.2.3 Historically, the surrounding area has comprised mainly residential properties, with the only significant land use identified as a railway 80m north of site and the River Crane beyond at approximately 176m from site.

2.2.4 The British Geological Survey indicates that the site is directly underlain by superficial deposits of the Langley Silt Member. Superficial deposits of the Kempton Park Gravel Member are anticipated to underlie the Langley Silt Member. These superficial deposits overlie solid deposits of the London Clay Formation.

2.2.5 The underlying Langley Silt Member and the London Clay Formation are identified as Unproductive. The Kempton Park Gravel Member is reported (off-site) as a Principal Aquifer.

2.2.6 A review of the EnviroInsight Report indicates that there are no Environment Agency Zone 2 or Zone 3 flood zones within 250m of the site.

2.2.7 The River Crane is reported 176m north-west.

Basement Impact Assessment (Screening and Scoping)

2.2.8 Screening identifies the area that require further (usually intrusive) investigation whilst scoping is the activity of defining in further detail the matters to be investigated as part of the BIA process. Scoping comprises of the definition of the required investigation needed in order to determine in detail the nature and significance of the potential impacts identified during screening.

2.2.9 These issues are summarised below:

2.2.10 The site predominantly comprises hardstanding cover which includes the existing building on site, a driveway area and a rear external patio. Areas of gravel and small plants are present adjacent to the building. The proposed plans show that there will be a reduction in hardstanding area to the front of the building through provision of a new garden area, though the majority of this will be underlain by the basement.

2.2.11 The site was considered to be at low risk of flooding based on historic flooding.

2.2.12 No risk of flooding to the site from artificial sources was identified.

2.2.13 The published geological maps indicate that the site is directly underlain by superficial deposits of the Langley Silt Member and the Kempton Park Gravel Member. These superficial deposits are underlain by solid deposits of the London Clay Formation. This should be confirmed by an intrusive investigation. Geotechnical laboratory testing of soils should also be undertaken to establish their shrink/swell properties.

2.2.14 The proposed basement excavation will be within 5m of a public pavement, and within 5m of neighbouring properties.

2.2.15 Unavoidable lateral ground movements associated with the basement excavations must be controlled during temporary and permanent works so as not to impact adversely on the stability of the surrounding ground, any associated services and structures.

2.2.16 It is recommended that the site is supported by suitably designed temporary support with a basement box construction. This will ensure that the adjacent land is adequately supported in the temporary and permanent construction. Alternatively, the excavation should proceed in a manner that maintains the integrity of the ground on all sides.

2.2.17 Careful and regular monitoring of the structure will need to be undertaken during the construction phase to ensure that vertical movements do not adversely affect the above property and neighbouring structures. If necessary, the works may have to be carried out in stages with the above structure suitably propped and supported.

2.2.18 Full details of the suitable engineering design of the scheme in addition to an appropriate construction method statement should be submitted by the developer to the London Borough of Richmond upon Thames.

2.2.19 The overall assessment of the site is that the creation of a basement for the existing development will not adversely impact the site or its immediate environs, providing measures are taken to protect surrounding land and properties during construction.

2.3 Previous Ground Investigations

2.3.1 Jomas is not aware of any previous intrusive investigation works that have been undertaken on the site.

3 GROUND INVESTIGATION

3.1 Scope of Works

3.1.1 A ground investigation was undertaken on the 10 October 2024.

3.1.2 A summary of the fieldwork carried out at the site, with justifications for exploratory hole positions, is presented in Table 3.1 below.

Table 3.1: Scope of Intrusive Investigation

Investigation Type	Number of Exploratory Holes Achieved	Exploratory Hole Designation	Depth Achieved	Justification
Cable Percussion Borehole	1	BH1	10mbgl	Obtain samples for laboratory geotechnical testing. To allow in-situ geotechnical testing.
Monitoring Well	1	BH1	7.5mbgl	Groundwater monitoring wells.

3.1.3 The ground investigation was undertaken in accordance with British Standard BS5930:2015+A1:2020 "Code of practice for ground investigations", British Standard BS10175:2011+A2:2017 "Investigation of potentially contaminated sites - code of practice", NHBC Standards, Chapter 4.1 and AGS Guidelines for Good Practice in Site Investigations.

3.1.4 The exploratory hole position is shown on the exploratory hole location plan presented in Figure 2, Appendix 1. The exploratory hole record is included in Appendix 2.

3.2 Geotechnical Testing

In-situ

3.2.1 In-situ geotechnical testing included Standard Penetration Tests (SPTs). The determined N-values have been used to determine the relative density of granular materials and have been used with standard correlations to infer various other derived geotechnical parameters including the undrained shear strength of the cohesive strata. The results of the individual tests are on the appropriate exploratory hole logs in Appendix 2.

Laboratory

3.2.2 Soil samples were obtained and submitted to the UKAS accredited laboratory of K4 Soils Ltd for a series of analyses.

3.2.3 This testing was designed to classify the samples; and to obtain parameters (either directly or sufficient to allow relevant correlations to be used) relevant to the technical objectives of the investigation.

3.2.4 The following laboratory geotechnical testing was carried out:

Table 3.2 Laboratory Geotechnical Analysis

Methodology	Test Description	Number of tests
BS1377:1990	Moisture Content Determination	2
BS1377:1990	Liquid and Plastic Limit Determination (Atterberg Limits)	2
BS1377:1990	Particle Size Distribution - Sieving	3
BS1377:1990	Determination of the undrained shear strength in triaxial compression with single-stage loading and without measurement of pore pressure	1

3.2.5 The geotechnical laboratory test results are included in Appendix 3.

3.2.6 In addition, 5No soil samples were sent to the UKAS and MCerts accredited laboratory of Derwentside Environmental Testing Services Ltd and analysed for a modified BRE Special Digest 1 suite (acid and water soluble sulphate, total sulphur and pH) to assist with the ACEC classification for buried concrete. The results of this chemical testing are included in Appendix 4.

4 ENCOUNTERED CONDITIONS

4.1 General

4.1.1 A factual record of the conditions encountered during the physical investigation of the site is presented in the following section.

4.1.2 For further details of the ground conditions, reference should be made to the exploratory hole location plan presented in Appendix 1, exploratory hole log presented in Appendix 2, and the laboratory testing results in Appendix 3 and 4.

4.2 Ground Conditions

4.2.1 The ground conditions encountered were broadly consistent with those anticipated, i.e. a thickness of Made Ground overlying the Langley Silt Member over the Kempton Park Gravel Member over the London Clay Formation, and are summarised in Table 4.1 below.

Table 4.1: Ground Conditions Encountered

Stratum and Description	Encountered from (mbgl)	Base of strata (mbgl)	Thickness range (m)
Concrete over (dark) brown clayey silty gravelly sand. Sand is fine to coarse. Gravel consists of fine to coarse, angular to rounded flint, brick and concrete. (MADE GROUND)	0.0	1.9	1.9
Dense to very dense orangish brown slightly clayey very sandy GRAVEL. Sand is fine to coarse. Gravel consists of fine to coarse, angular to rounded flint. (KEMPTON PARK GARVEL MEMBER)	1.9	7.4	5.5
Firm to stiff consistency** dark grey CLAY. (LONDON CLAY FORMATION)	7.4	>10.0 [base not proven]	>2.6 [thickness not proven]

***Consistency estimated using semi-empirical correlations with SPT N-values, Plasticity Indices and published literature*

4.2.2 No visual or olfactory evidence of potential contamination was identified within the investigation positions.

4.3 Hydrogeology

4.3.1 Groundwater strikes and groundwater monitoring are summarised below.

Table 4.2: Groundwater Strikes During Investigation

Exploratory Hole ID	Depth Encountered (mbgl)	Depth Post-Drilling (mbgl)	Stratum
BH1	6.20	6.45	Kempton Park Gravel Member

4.3.2 1No return groundwater monitoring visit was undertaken on 18 October 2024, the results are presented in Appendix 5 and are summarised below. A second visit is due to take place in February 2025.

Table 4.3: Groundwater Monitoring Summary

Exploratory Hole ID	Depth Encountered (mbgl)	Well response zone as installed (mbgl)	Depth base of well (mbgl)	Stratum targeted by response zone
BH1	6.53	1.00 – 7.50	8.02	Made Ground and Kempton Park Gravel Member

4.3.3 While the monitoring well is understood to have been installed to 7.5mbgl, the depth to the base of the well measured during the return monitoring visit was 8.02mbgl. This is potentially due to an error when measuring the pipe for installation, and/or the top of the monitoring well being located below ground level.

4.3.4 It should be noted that changes in groundwater levels can occur for a number of reasons including seasonal effects and variations in drainage. Such fluctuations may only be recorded by the measurement of the groundwater level within a standpipe or piezometer installed within appropriate response zones. Changes in groundwater level can have a direct effect on excavation stability and dewatering requirements, and cohesive soils can soften under rising or high groundwater levels.

4.4 Limitations

4.4.1 During the intrusive ground investigation, no impenetrable obstructions were encountered. However, the possible presence of natural and/or manmade obstructions on site cannot be discounted.

5 DERIVATION OF GEOTECHNICAL PARAMETERS

5.1 Introduction

5.1.1 A summary of ground conditions obtained from the ground investigation and the derived geotechnical parameters is provided below.

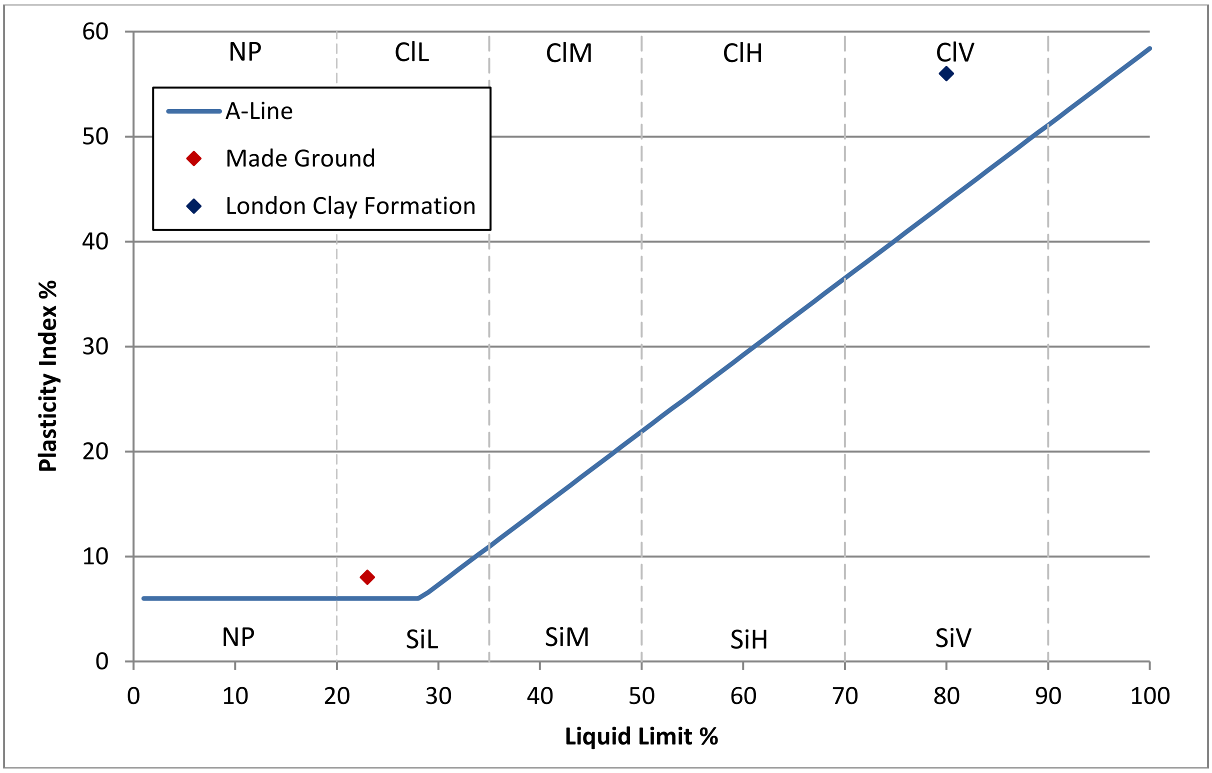
5.2 Plasticity of Cohesive Materials

5.2.1 Atterberg Limit determination was undertaken on 1No sample of Made Ground at a depth of 1.7mbgl, and 1No sample of the London Clay Formation at a depth of 9.5mbgl.

5.2.2 Within the Made Ground, the plasticity index value was 8% and was indicative of low plasticity, as illustrated in Figure 5.1 below. The modified plasticity index value was 4.96%, indicating that these soils are non-shrinkable.

5.2.3 The plasticity index value within the London Clay Formation was 56% and was indicative of very high plasticity. The modified plasticity index value was 53.2%, indicating soils with high volume change potential.

Figure 5.1: Plasticity Chart

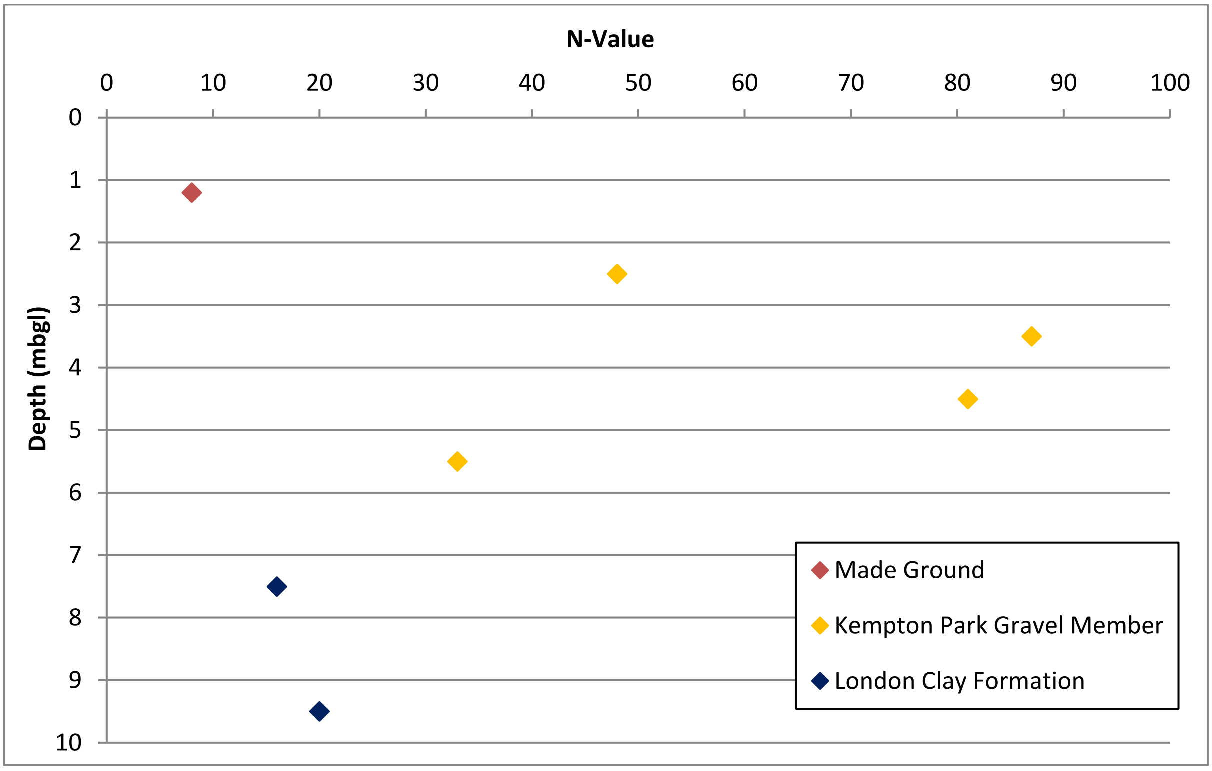


5.3 Standard Penetration Tests

5.3.1 Standard Penetration Tests were undertaken at regular intervals throughout the cable percussive borehole. The results of the SPTs are plotted against depth in Figure 5.2 below.

5.3.2 N_{equi} results have been calculated where the full 300mm of penetration could not be achieved for 50 or more blows

Figure 5.2: SPT N-Value v Depth



5.4 Undrained Shear Strength

5.4.1 As discussed above, the N values recorded in the clay vary with depth, this infers that the undrained shear strength of the clay similarly varies. Figure 5.3 below shows the undrained shear strength inferred by the correlation suggested by Stroud (1974);

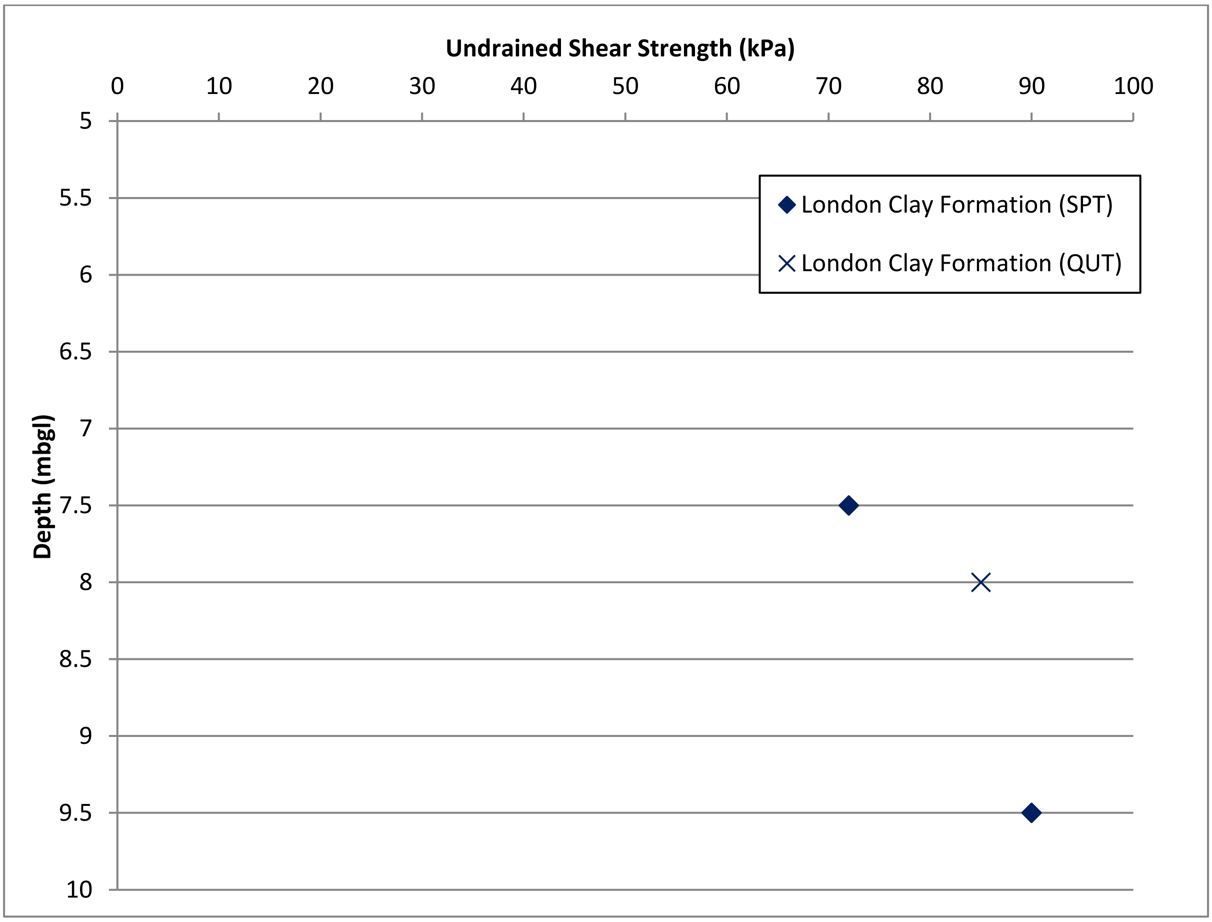
$c_u = f_1 \times N$ can be applied,

in which
c_u= mass shear strength (kN)
f_1 = constant
N= SPT value achieved during boring operations

5.4.2 In the above equation f_1 is dependent on the plasticity of the material that the SPT is being carried out in. As the plasticity indices were shown to be greater than 25% a value for f_1 of 4.5 has been adopted after Tomlinson (2001).

5.4.3 The graph below shows the shear strength profile of the encountered cohesive materials at the site, based on the SPT to shear strength correlation described above, as well as the results of quick undrained triaxial (QUT) testing on undisturbed samples taken from the borehole.

Figure 5.3: Undrained Shear Strength v Depth



5.4.4 As shown above, a general trend of increasing undrained shear strength with depth can be seen within the limited results from the London Clay Formation.

5.5 Coefficient of Compressibility

5.5.1 Stroud and Butler (1974) developed a relationship between the coefficient of compressibility (m_v) and SPT N-value.

$m_v = 1/ f_2 \times N$ can be applied,

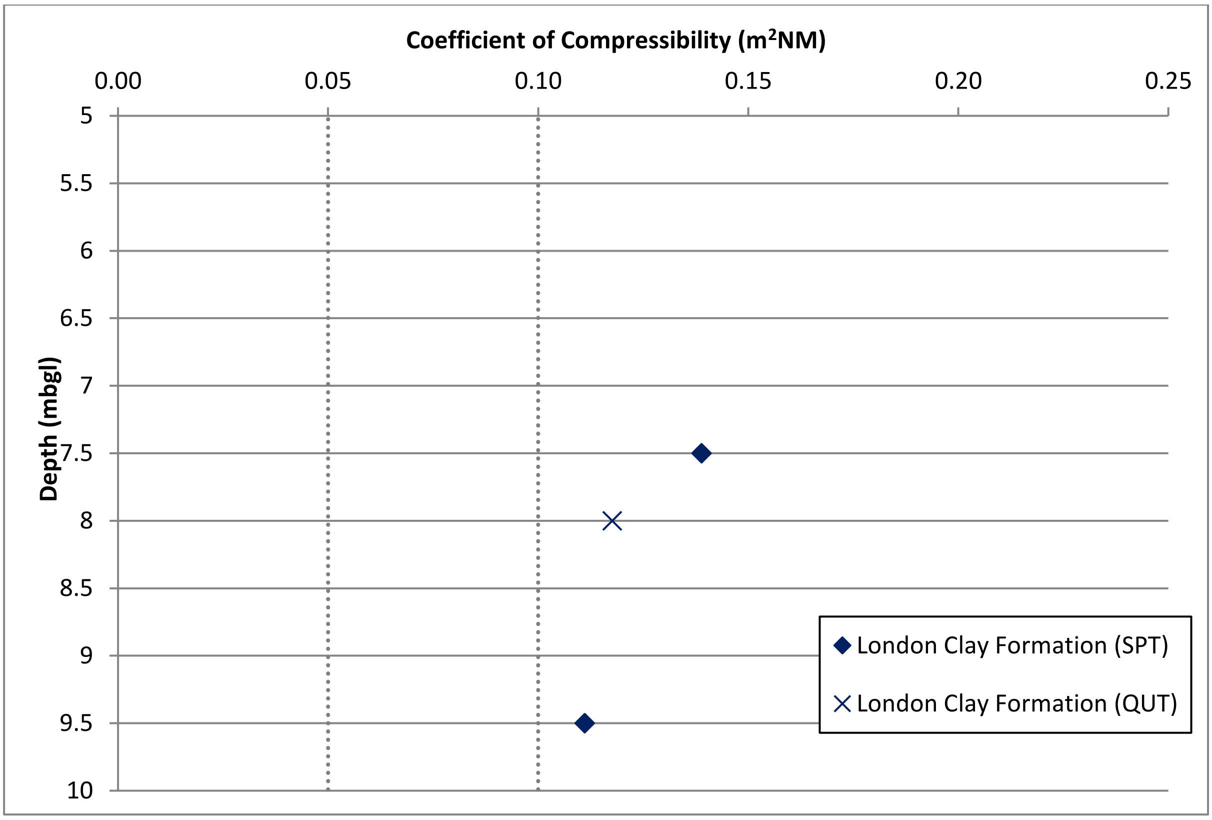
in which
m_v = coefficient of compressibility (m^2/MN)

f_2 = constant dependent on the plasticity index
N = SPT value achieved during boring operations

5.5.2 Using the plasticity indices obtained and the graphs provided in Tomlinson (2001) a value of f_2 of 0.45 has been taken and used with the SPT N-values to infer coefficient of compressibility (m_v).

5.5.3 Where the undrained shear strength of the clays was measured using the quick undrained triaxial methodology, the m_v value was calculated by rearranging the equations for f_1 and f_2 and substituting in the measured undrained shear strength.

Figure 5.4: Coefficient of Volume Compressibility (mv) v Depth



5.5.4 As shown above, the results from the London Clay Formation are of "medium compressibility".

5.6 Density

5.6.1 In order to calculate the undrained shear strength using the quick undrained triaxial methodology, the bulk density of the materials has to be calculated, which are provided on the testing certificates in Appendix 4. These values can be converted to a unit weight value in kN/m^3.

5.6.2 In the absence of geotechnical laboratory test results, the correlations and suggested unit weight values for both cohesive and granular materials given in BS8004:2015 have been used.

5.6.3 The derived unit weights are summarised below in Table 5.1.

Table 5.1: Derived Unit Weights

Strata	Unit Weight (kN/m^3)
Made Ground	17
Kempton Park Gravel Member	20
London Clay Formation	19.5

5.7 Effective Angle of Shearing Resistance / Angle of Friction

5.7.1 In cohesive soils, the effective angle of shearing resistance can be derived from the plasticity index of the soil, using the following equation presented in BS8004:2015.

$$\emptyset' = 42 - (12.5xLOG10(PI))$$

Where PI = Plasticity Index.

5.7.2 Values have been calculated for all available Plasticity Index results and are presented in Table 5.2.

Table 5.2: Derived Angles of Shearing Resistance

Sample	Stratum	Derived Angle of Shearing Resistance (°)
BH1 – 1.7m	Made Ground	30.7
BH1 – 9.5m	London Clay Formation	20.1

5.7.3 In granular materials, the effective angle of friction can be derived directly from shear box testing, or indirectly using the methodology outlined in Table 1 of BS8004:2015, using a combination of the SPT N-values, Particle Size Distribution of the soil, and the field descriptions of angularity of the gravel fraction. This method assumes that the fines content of the material is less than 15%. An alternative method is to refer to the correlation between angle of friction and SPT N-values postulated by Peck *et al* (1967) and reproduced in Tomlinson (2001).

5.8 Stiffness Moduli

5.8.1 In cohesive soils of the London Clay Formation, the undrained stiffness modulus (Young's Modulus) can be derived using the correlation with undrained shear strength as postulated by Jardine et al. (1985):

$$\underline{Eu\ = 400 * Cu(kPa)}$$

5.8.2 The drained Young's Modulus for the London Clay Formation can then be derived from E_u, as follows:

$$E' = 0.6 * Eu$$

5.8.3 In granular materials, the drained Young's Modulus can be derived using the following correlation:

$$E' = N$$

5.9 Summary of Derived General Properties

5.9.1 Based on the analysis of the ground investigation data and past experience with similar deposits, the following derived general parameters are given in Table 5.3.

Table 5.3: Derived General Parameters

Property	Made Ground	Kempton Park Gravel Member	London Clay Formation
Unit Weight	17[1)]	20[1)]	19.5[2)]
Drained Friction, ϕ' (◦)	30.7[3)]	36[4)]	20.1[3)]
Drained Cohesion, c' (kPa)	0	-	0
SPT N-value	8	31 – 87	16 – 20
Undrained Young's Modulus, E_u (MPa)[5)]	-	-	28.8 – 36
Drained Young's Modulus E' (MPa)	-	31.0 – 87.0[6)]	17.3 – 21.6[7)]
Undrained Shear Strength, c_u (kPa)[8)]	-	-	72 – 90
Undrained Shear Strength, c_u (kPa)[9)]	-	-	85
Plasticity Index (%)	8	-	56
Modified Plasticity Index (%)	5	-	53.2
Volume Change Potential [NHBC]	Non-shrinkable	-	High
Modulus of Volume Compressibility, m_v (m^2/MN)[10)]	-	-	0.111 – 0.139

1) Derived from Figures 1 and 2 of BS8004:2015
2) Calculated from bulk density, measured during quick undrained triaxial (QUT) testing
3) Calculated from: ϕ' = (42°- 12.5log10I_p) for 5% ≤ I_p ≤ 100% Where, I_p is the soil's plasticity index (BS8004:2015)
4) Calculated from Table 1 of BS8004:2015
5) Calculated from E_u = 0.4 x c_u MPa, based on the guidance given in Jardine et al 1985
6) Calculated from: E' = 1.0 x N MPa, based on the guidance given in CIRIA Report 143
7) Calculated from E' = 0.6 x Eu MPa, based on the guidance given in Jardine et al 1985
8) The undrained shear strength (c_u) of the cohesive soils was correlated to the SPT N-values using Stroud (1974), where $c_u=f_1N$ and f_1 is factor related to the Plasticity Index (PI) of the clay (a value of f_1 equal to 5.0 for PI ≤ 25% and a value of f_1 value equal to 4.5 for PI>25)
9) These values have been determined from the unconsolidated undrained triaxial compression testing in accordance with BS1377: Part 7: 1990, Clause 8
10) Calculated from: $m_v = 1/f_2$ N m^2/MN, f_2 is a coefficient proposed by Stroud and Butler (1975) and varies with Plasticity Index (PI) as presented in Figure 27 of CIRIA Report 27 or 10/c_u

6 GEOTECHNICAL ENGINEERING RECOMMENDATIONS

6.1 General

6.1.1 Subsequent to intrusive investigation of the site and receipt of the laboratory test results, the following geotechnical assessments have been made.

6.2 Proposed Foundations

General

6.2.1 All topsoil is to be stripped from beneath proposed structures ahead of development.

6.2.2 The Made Ground is not considered to provide suitable bearing strata due to its variability and the unacceptable risk of total and differential settlement.

6.2.3 All foundations should be deepened beneath these deposits, soft clay, root or desiccated zones, or disturbed ground, and founded within underlying competent strata.

Conventional Foundations

6.2.4 Based on drawings provided, it is anticipated that the finished floor level of the basement would be approximately 3m below existing ground level and therefore formation level is anticipated to be ~3.5mbgl.

6.2.5 Based upon the information obtained to date, it is considered that a cast in-situ cantilever retaining wall formed at approximately 3.5m below the existing ground level within the Kempton Park Gravel Member could be designed with an allowable bearing capacity of 200kPa. Total and differential settlements should be contained within tolerable limits.

6.2.6 It is unlikely that the foundations would need to be deepened further due to NHBC building near trees requirements.

6.2.7 Where foundations need to change levels, the foundations should be stepped and reinforced. These steps should be no deeper than half of the width of the foundation and each step should not exceed 0.5m.

6.2.8 If foundations span different strata, e.g. sand and clay, they should either be deepened to terminate in a single soil stratum, or suitable reinforcement included (to be detailed by the Structural Engineer).

6.2.9 Foundations greater than 2.50m deep require structure-specific design by a structural engineer.

6.2.10 It is recommended that excavations to form the foundations should be undertaken using a toothless bucket to reduce the potential for disturbance of the underlying Kempton Park Gravel Member.

6.2.11 Foundations should not be formed in the granular materials until the granular materials have been proof compacted. Given the depth and likely size of these foundations, it is considered that this could be undertaken using a hydraulic "elephants foot" or if the whole basement founding layer is compacted at the same time a vibrating roller or "whacker plate" if the machinery can be easily taken into the excavation and the stability of the excavation/safety of any workers entering the excavation can be assured.

6.2.12 Where any unexpected or soft ground conditions are encountered during the groundworks, works in that area should cease and the advice of a suitably qualified geotechnical engineer sought.

6.3 Retaining Walls

6.3.1 It is anticipated that retaining structure(s) will be required.

6.3.2 Based on the analysis of the available site investigation data and past experience with similar deposits the parameters in Table 6.1 are considered appropriate for the potential retaining structure(s).

Table 6.1: Geotechnical Parameters for Retaining Wall Design

	Kempton Park Gravel Member	London Clay Formation
Critical state angle of shearing resistance (ϕ')°	36	20
Effective Cohesion kN/m²	-	0
Saturated Bulk Weight (γ_{sat}) kN/m³	21	19.5

6.3.3 In addition, the specialist contractor should ensure the stability of the cut-face during the temporary works.

6.3.4 As an alternative to cantilever retaining walls, fully embedded retaining walls comprising a contiguous/secant piled basement box could be formed. The piles would need to act as retaining walls as well as carry the structural loadings. The piles should be designed to withstand the earth pressures, and still meet the required structural requirements regarding issues such as deflection, deformation and bending.

6.3.5 To provide sufficient support for the excavation, it is recommended that un-propped piles are formed to at least three times the depth of excavation.

6.3.6 If these piles can be suitably propped, then this depth may be reduced. Suitable propping could be provided by the basement floor and the ground floor if they are suitably tied into the piles and suitably reinforced. This may require specialist construction techniques.

6.4 Aggressive Ground Conditions

6.4.1 Sulphate attack on building foundations occurs where sulphate solutions react with the various products of hydration in Ordinary Portland Cement (OPC) or converted High-Alumina Cement (HAC). The reaction is expansive, and therefore disruptive, not only due to the formation of minute cracks, but also due to loss of cohesion in the matrix.

6.4.2 In accordance with BRE Special Digest 1, the characteristic values of sulphate used to determine the concrete classification are determined using the methodology summarised in the table below.

Table 6.2: Concrete in the Ground Characteristic Value Determination

No Samples in the dataset	Method for determining the sulphate characteristic value
1 - 4	Highest value
5 - 9	Mean of the top 2No highest results
10 or greater	Mean of the top 20% highest results

6.4.3 Table 6.3 summarises the analysis of the aggressive nature of the ground for each of the strata encountered within the ground investigation.

Table 6.3: Concrete in the Ground Classes

Stratum	No Samples	pH range	Characteristic WS Sulphate (mg/l)	Characteristic Total Potential Sulphate (%)1)	Design Sulphate Class	ACEC Class
Made Ground	2	8 – 8.7	80	N/A	DS-1	AC-1
Kempton Park Gravel Member	2	8.4 – 8.7	<10	N/A	DS-1	AC-1
London Clay Formation	1	8.4	173	0.87	DS-3	AC-3

1) Applies to soils containing more than 0.3% of oxidisable sulphides, calculated in accordance with BRE SD-1

6.4.4 Analysis of the results indicates that the London Clay Formation contains significant concentrations of oxidisable sulphides (e.g. pyrite), which can be oxidised to form additional sulphate on disturbance and exposure to air as outlined in BRE SD-1:2005. The total potential sulphate must therefore also be considered in the designation of a Design Class, in cases where the London Clay Formation is to be disturbed and exposed to air.

6.4.5 Where these deposits are not likely to be disturbed and exposed, but foundations are formed within them (such as piles), then a Design Class of DS-2 is recommended, with an Aggressive Chemical Environment for Concrete (ACEC) Classification of AC-2.

6.4.6 The concrete structures, including foundations, will need to be designed in accordance with BS EN 1992-1-1:2004+A1:2014. It is recommended that the advice of this publication be taken for the design and specification of all sub-surface concrete.

6.5 Floor Slabs

6.5.1 It is anticipated that finished floor level of the proposed basement will be approximately 3m below the existing ground floor level.

6.5.2 If a cantilever retaining wall is utilised, then a ground bearing floor slab could be used. Given the material at these depths, it is considered likely that such floor slabs could be constructed on the in-situ natural granular materials. In this case, formations of the structures should be inspected by a competent person. Any loose or soft material should be removed and replaced with well-graded, properly compacted granular fill or lean mix concrete. The formation should be blinded if left exposed for more than a few hours or if inclement weather is experienced.

6.5.3 If a piled option is utilised then suspended floor slabs will be required. The loadings from the suspended floor slab will need to be carried by the foundations, which will need to be designed to not only carry the structural loadings but the additional floor loadings.

6.5.4 All floor slabs would also need to be suitably reinforced, not only to distribute the structural loading but also to ensure that the floor slab can prop the retaining walls and does not buckle from the lateral pressures imposed by the cantilever retaining walls.

6.5.5 The floor slab (and basement walls) would need to be constructed to conform to BS: 8102 (2009).

6.6 Excavations

6.6.1 Temporary excavations within the Made Ground and granular soils are unlikely to remain stable and some form of temporary support or battering back to a safe angle and dewatering are likely to be required.

6.6.2 Temporary excavations within the cohesive soils are likely to remain relatively stable in the short term though some spalling may be anticipated.

6.6.3 Cantilever retaining walls should be installed in short sections to aid stability of the excavation during construction of the basement.

6.6.4 Ground works should always be designed in such a manner to avoid entry into excavations by construction or maintenance personnel. However, in the event that such works cannot be avoided or designed out, they should only be undertaken in accordance with a safe system of work, following an appropriate risk assessment and in accordance with any legislative requirements, e.g. Confined Spaces Regulations.

6.7 Groundwater Control

6.7.1 During the investigation, groundwater was reported within the borehole at a depth of 6.2mbgl, and by the time the drilling had concluded, was sat at a level of 6.45mbgl.

6.7.2 During return monitoring, groundwater was reported at 6.53mbgl. A second visit is due to take place in February 2025 and this report will be updated.

6.7.3 Subject to seasonal variations, any groundwater encountered during site works could be readily dealt with by conventional pumping from a sump used to collate waters.

6.7.4 Surface water or rainfall ingress is likely to freely drain through the granular materials. If this does not occur, then they too could be dealt with by traditional sump and pump.

7 BASEMENT IMPACT ASSESSMENT

7.1 Geological Impact

7.1.1 The published geological maps indicate that the site is directly underlain solid deposits of the Langley Silt Member and Kempton Park Gravel Member. These superficial deposits are underlain by solid deposits of the London Clay Formation

7.1.2 The ground conditions were confirmed by a ground investigation and comprise Made Ground to a depth of 1.9mbgl, underlain by granular deposits of the Kempton Park Gravel Member to 7.4mbgl, underlain by cohesive deposits of the London Clay Formation to a depth in excess of 10mbgl. The proposed basement will be founded within the Kempton Park Gravel Member at a depth of ca. 3.5mbgl.

7.1.3 Laboratory testing indicates that the London Clay Formation is of high volume change potential. However, with consideration of the depth of these deposits, it is not considered that they will have an impact on the proposed basement.

7.2 Hydrology and Hydrogeology Impact

7.2.1 Based on all the information available at the time of writing, the risk of flooding from groundwater is considered to be low to moderate. The site was shown on mapping to not be located within an area where there is increased potential for elevated groundwater due to permeable surface deposits. The site was identified to be located within an area with a susceptibility to groundwater flooding of <25%.

7.2.2 During the investigation, groundwater was reported at depths of between 6.2mbgl and 6.53mbgl. At this stage, on this basis, it is considered that the proposed basement is unlikely to have a detectable impact on the groundwater regime. However, an additional groundwater monitoring visit is due to be conducted in February 2025, and this report will be updated on receipt of the results.

7.2.3 Appropriate water proofing measures should be included within the whole of the proposed basement wall/floor design as a precaution.

7.2.4 The Kempton Park Gravel Member is classed as a Secondary A Aquifer but the creation of the basement is considered unlikely to have any impact upon the hydrogeology of the area.

7.2.5 The proposed development will lie outside of flood risk zones and is therefore assessed as being at low probability of fluvial flooding.

7.2.6 The River Crane is reported 176m north-west of the site.

7.2.7 The information available suggests that the site lies in an area that is at low risk of surface water flooding.

7.2.8 The proposed basement construction is unlikely to result in an increase in impermeable areas in the post development scenario.

7.2.9 No risk of flooding to the site from artificial sources has been identified.

7.3 Other Impacts

7.3.1 Impacts such as changes to areas of external hardstanding, past flooding, and impacts to adjacent properties and pavement are addressed within the Stage 1 & 2 (Screening and Scoping) Basement Impact Assessment for 26 Amyand Park Road, Twickenham, TW1 3HE (Jomas Associates Ltd, P5802J3027/HAH, June 2024).

7.3.2 Full details of the suitable engineering design of the scheme in addition to an appropriate construction method statement should be submitted by the Developer to the London Borough of Richmond upon Thames.

7.4 Cumulative Impacts

7.4.1 The above individual effects could potentially interact to form a greater issue.

7.4.2 The site has been identified as being directly underlain by a Secondary A Aquifer (Kempton Park Gravel Member).

7.4.3 However, no sensitive uses have been identified in the surrounding area.

7.4.4 Furthermore, the modest size of the proposed basement will not significantly alter the existing groundwater regime.

7.4.5 The development of the basement will therefore not significantly affect the groundwater flow on or surrounding the site.

7.5 Conclusion

7.5.1 The overall assessment of the site is that the creation of a basement for the existing development will not adversely impact the site or its immediate environs, providing measures are taken to protect surrounding land and properties during construction.

7.5.2 The proposed development is not expected to cause significant problems to the subterranean drainage.

8 REFERENCES

AGS Guidelines for Good Practice in Geotechnical Ground Investigation, 2016

BRE Report BR 470: Working platforms for tracked plant, 2004. BRE: Watford

BRE Special Digest 1: Concrete in Aggressive Ground, 2005. BRE: Watford

British Standards Institution BS 10175:2011+A2:2017 Code of practice for the investigation of potentially contaminated sites. BSI: London

British Standards Institution BS 5930:2015+A1:2020 Code of practice for ground investigations. BSI:London

British Standards Institution BS 8002:2015 Code of practice for earth retaining structures. BSI: London

British Standards Institution BS 8004:2015 Code of practice for foundations. BSI: London

British Standards Institution BS EN 1997-1:2004+A1:2013 Eurocode 7. Geotechnical design. General rules. BSI: London

CIRIA Report R143 The standard penetration test (SPT): methods and use, 1995: CIRIA: London

Ministry of Housing, Communities & Local Government: National Planning Policy Framework. February 2019.

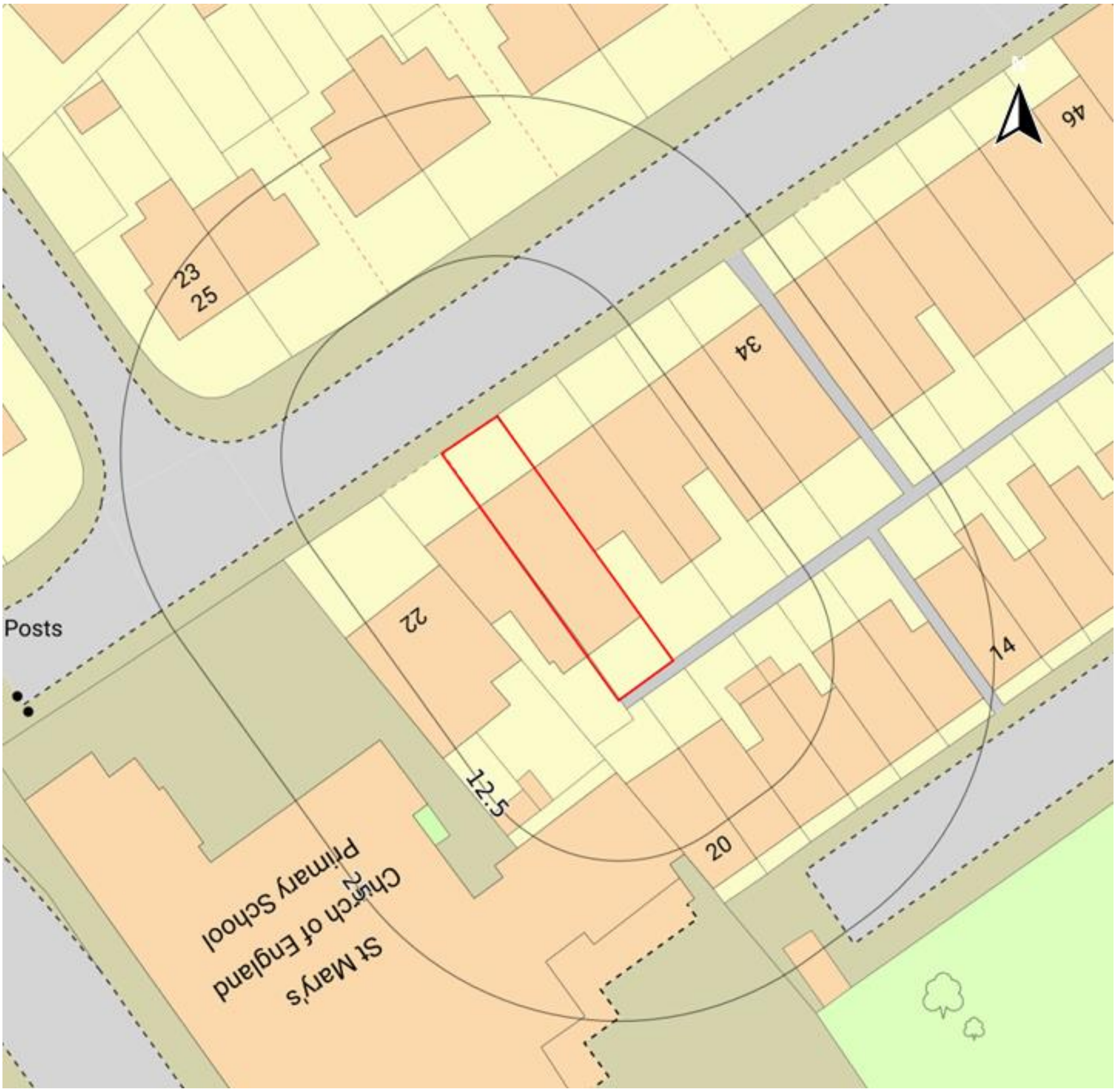
NHBC Standards 2023. NHBC, Milton Keynes

Tomlinson M.J (2001): Foundation Design and Construction 7th Edition. Pearson prentice Hall: Harlow

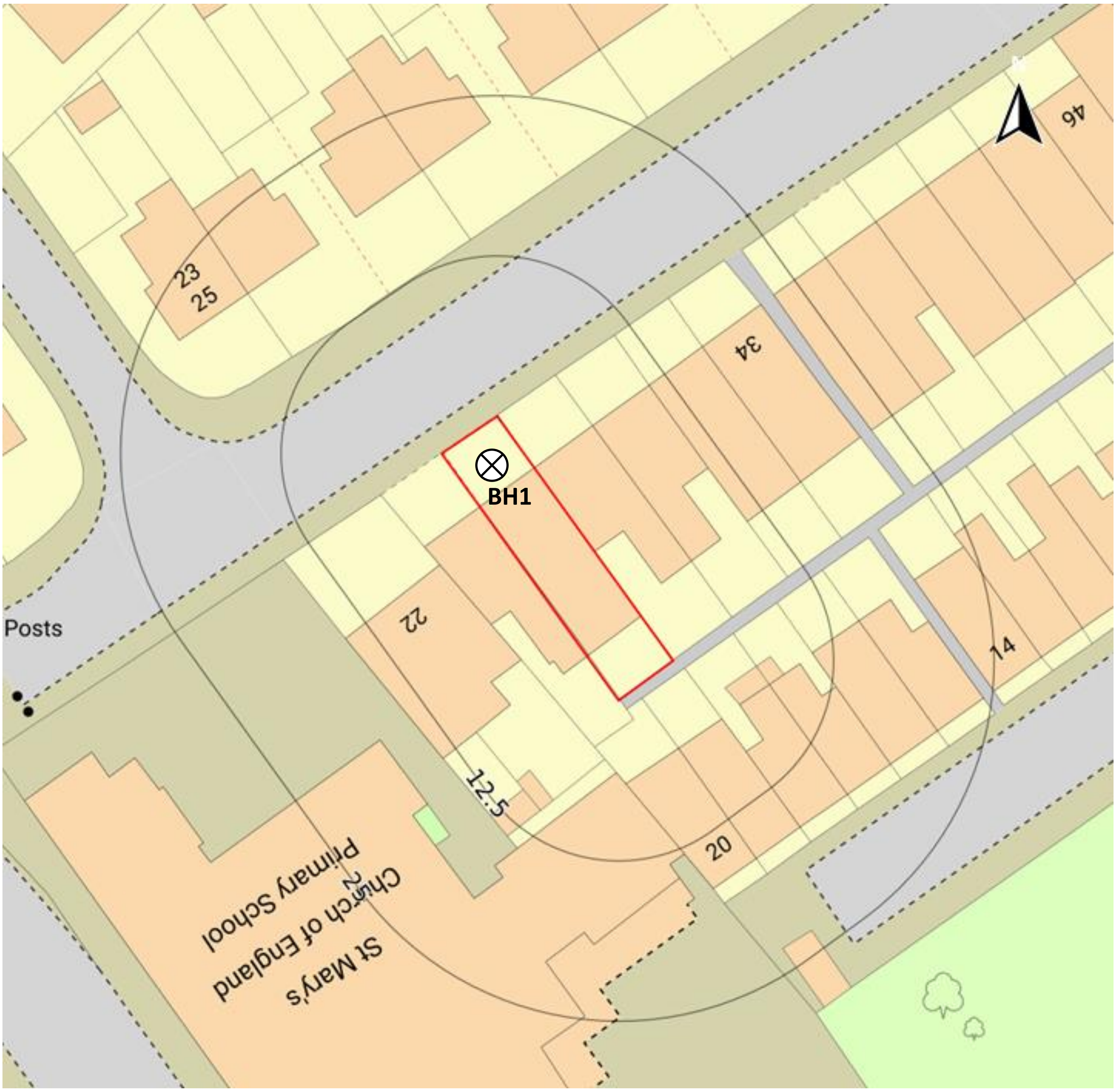
APPENDICES

APPENDIX 1 – FIGURES

PROJECT NAME	26 Amyand Park Road, TW1 3HE	**CLIENT**	05 Group Ltd
TITLE	Site Location Plan	**PROJECT NO.**	P5802J3027
DATE	June 2024	**FIGURE NO.**	1



PROJECT NAME	26 Amyand Park Road, TW1 3HE	**CLIENT**	05 Group Ltd
TITLE	Completed Exploratory Hole Plan	**PROJECT NO.**	P5802J3027
DATE	October 2024	**FIGURE NO.**	2



PROJECT NAME	26 Amyand Park Rd, TW1 3HE	CLIENT	05 Group Ltd
TITLE	Walkover Photo Plan	FIGURE	3

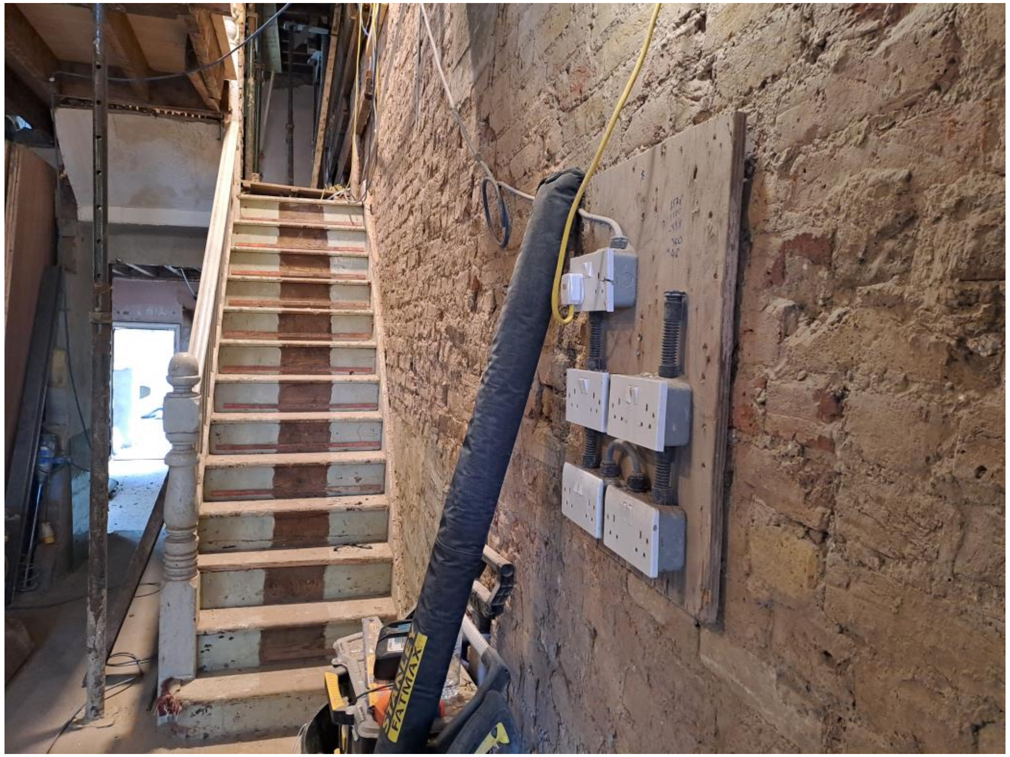
Photo 1: Overview of front of site.

Photo 2: Overview of front garden of site.

PROJECT NAME	26 Amyand Park Rd, TW1 3HE	**CLIENT**	05 Group Ltd
TITLE	Walkover Photo Plan	**FIGURE**	3

Photo 3: Main living room of site.

Photo 4: Site is connected to electrics.



PROJECT NAME	26 Amyand Park Rd, TW1 3HE	**CLIENT**	05 Group Ltd
TITLE	Walkover Photo Plan	**FIGURE**	3

Photo 5: Internal doorway leading to kitchen area of site.

Photo 6: Back doors of site.

PROJECT NAME	26 Amyand Park Rd, TW1 3HE	**CLIENT**	05 Group Ltd
TITLE	Walkover Photo Plan	**FIGURE**	3

Photo 7: Toilet of site.

Photo 8: Back garden of site from the doorway.

PROJECT NAME	26 Amyand Park Rd, TW1 3HE	**CLIENT**	05 Group Ltd
TITLE	Walkover Photo Plan	**FIGURE**	3

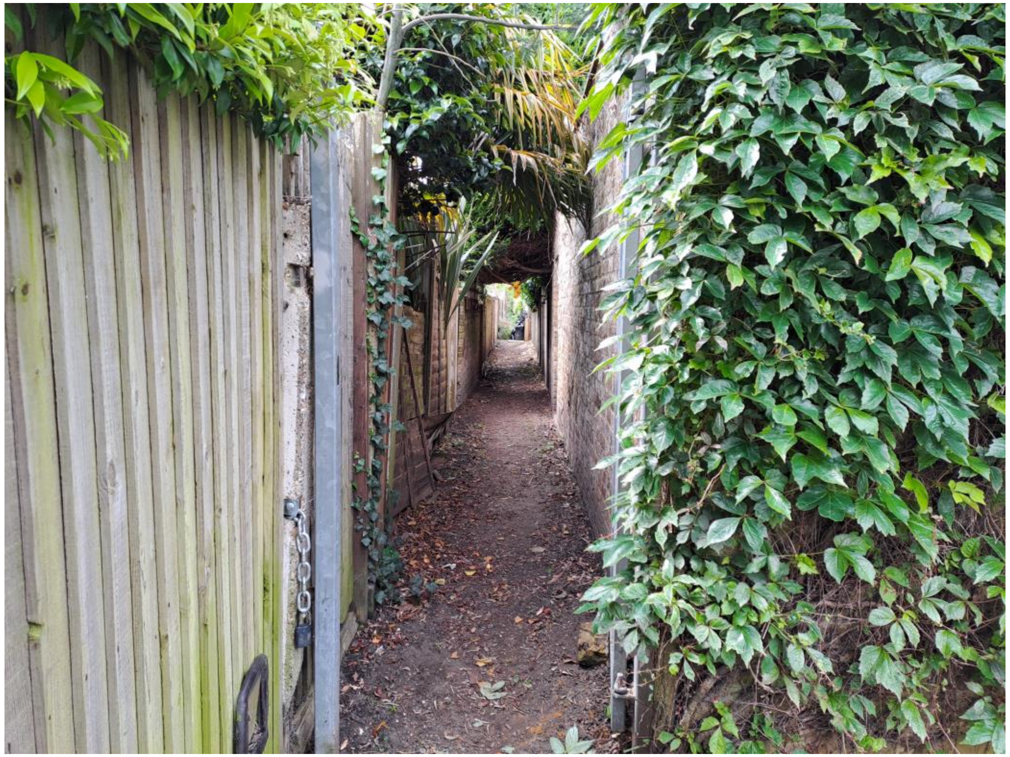
Photo 9: Back garden of site from gate.

Photo 10: External water supply by front door.

PROJECT NAME	26 Amyand Park Rd, TW1 3HE	**CLIENT**	05 Group Ltd
TITLE	Walkover Photo Plan	**FIGURE**	3

Photo 11: Drainage in back garden.

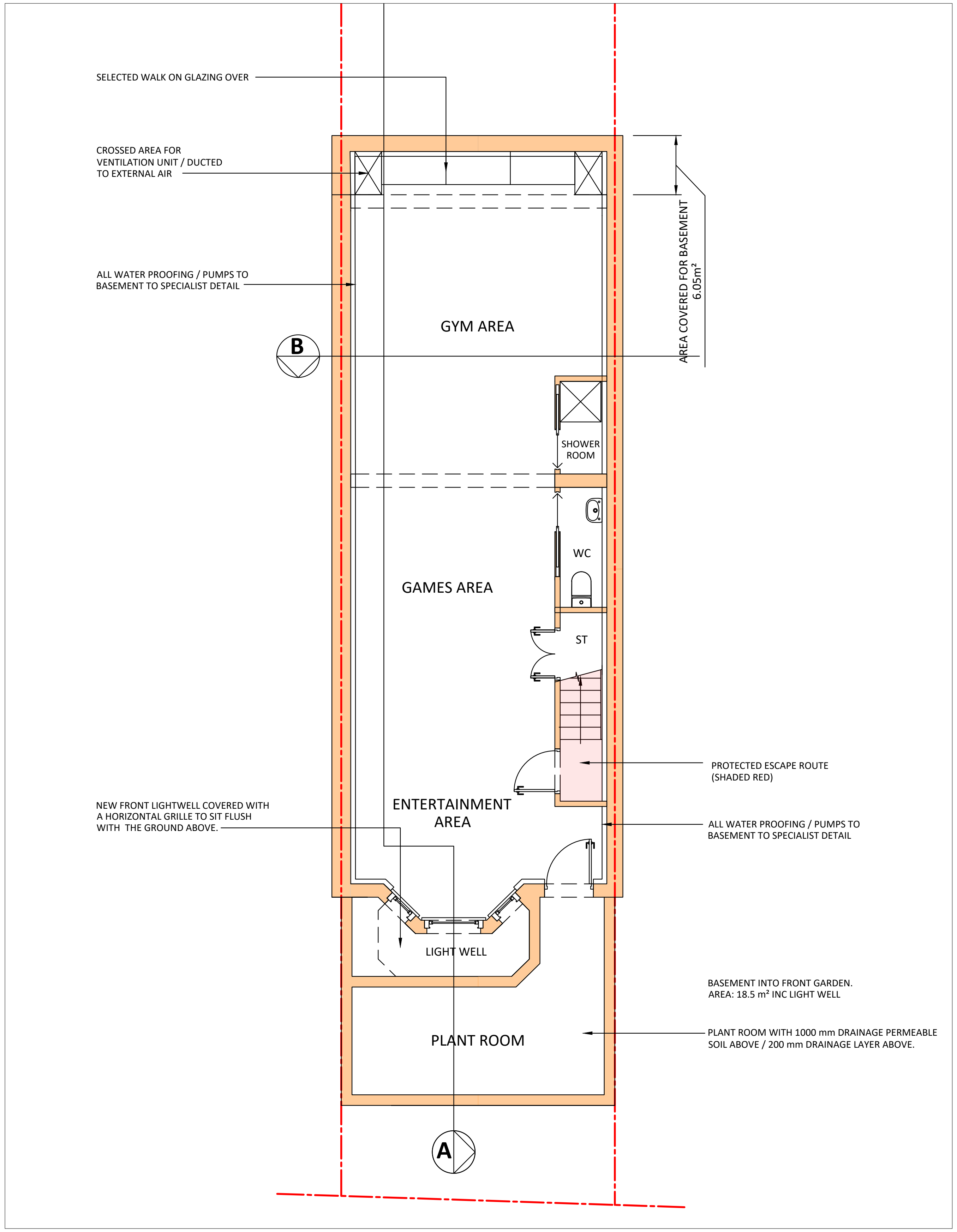
Photo 12: Alleyway leading to back gate.



PROJECT NAME	26 Amyand Park Rd, TW1 3HE	**CLIENT**	05 Group Ltd
TITLE	Walkover Photo Plan	**FIGURE**	3

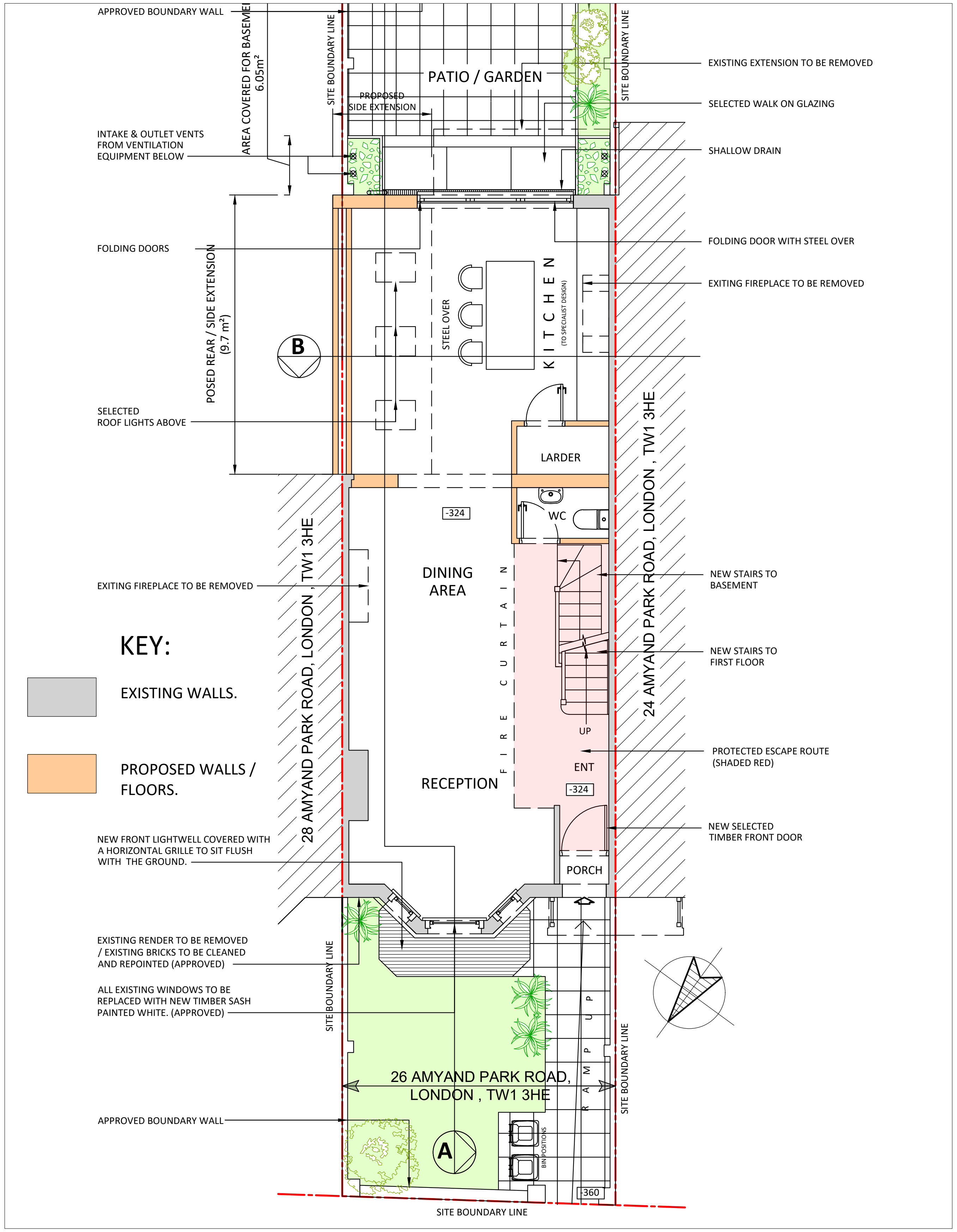
Photo 13: Back gate of site from alleyway.

Figure 4: Proposed Development Plan (Basement and Ground Floors)



PROPOSED BASEMENT FLOOR

SCALE 1:100 @ A3



PROPOSDED GROUND FLOOR PLAN

SCALE 1:100 @ A3

NOTES

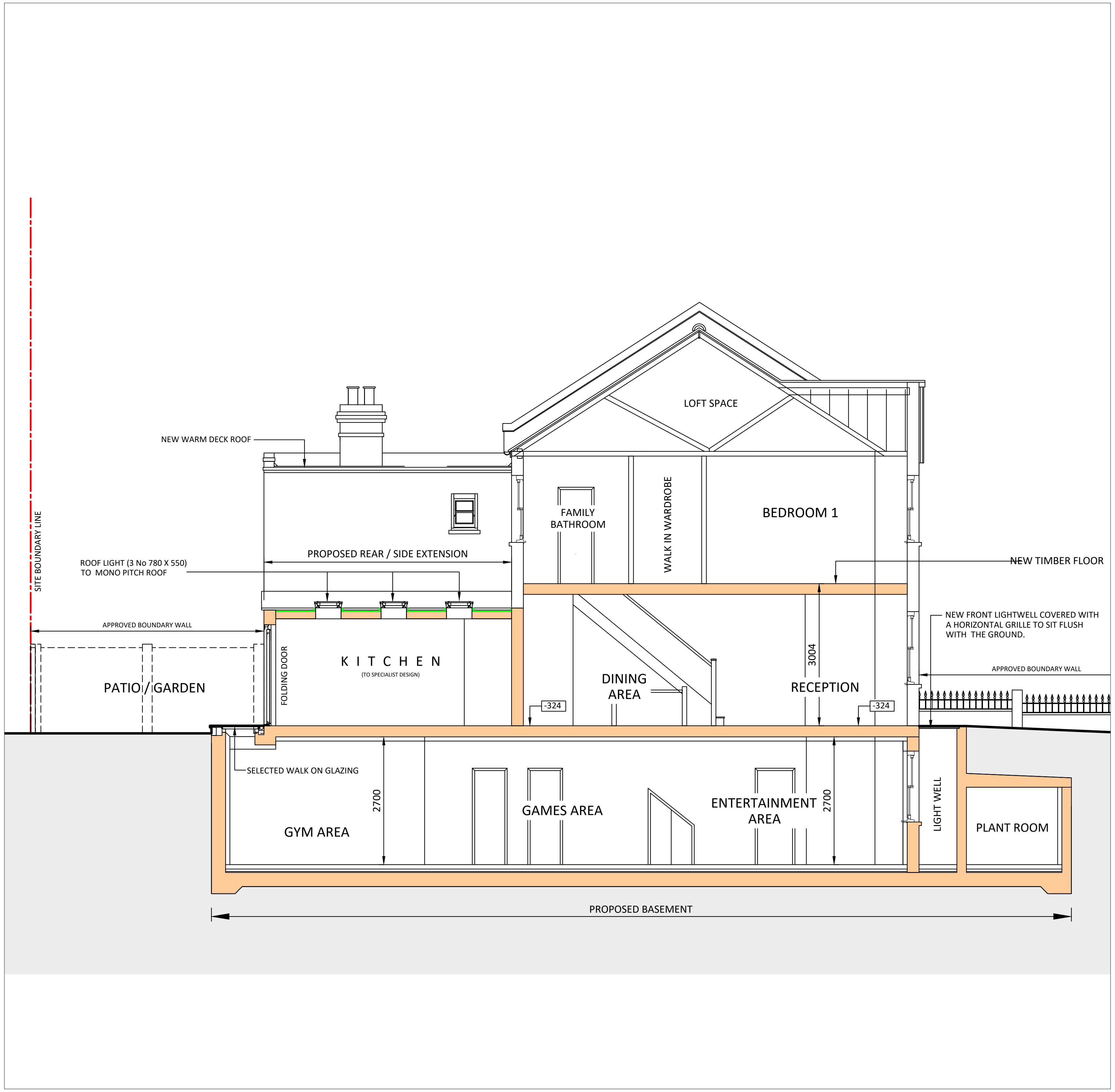
1. ALL DIMENSIONS TO BE CHECKED ON SITE.
2. THIS DRAWING HAS BEEN DRAWN TO SCALE, AS SHOWN, FOR THE PURPOSE OF OBTAINING LOCAL AUTHORITY APPROVAL.

BAR SCALE: 1:100 @ A3 — 1 CM, 0, 1 CM, 2 CM, 3 CM, 4 CM / 1.0M, 0, 1.0M, 2.0M, 3.0M, 4.0M

REVISIONS:

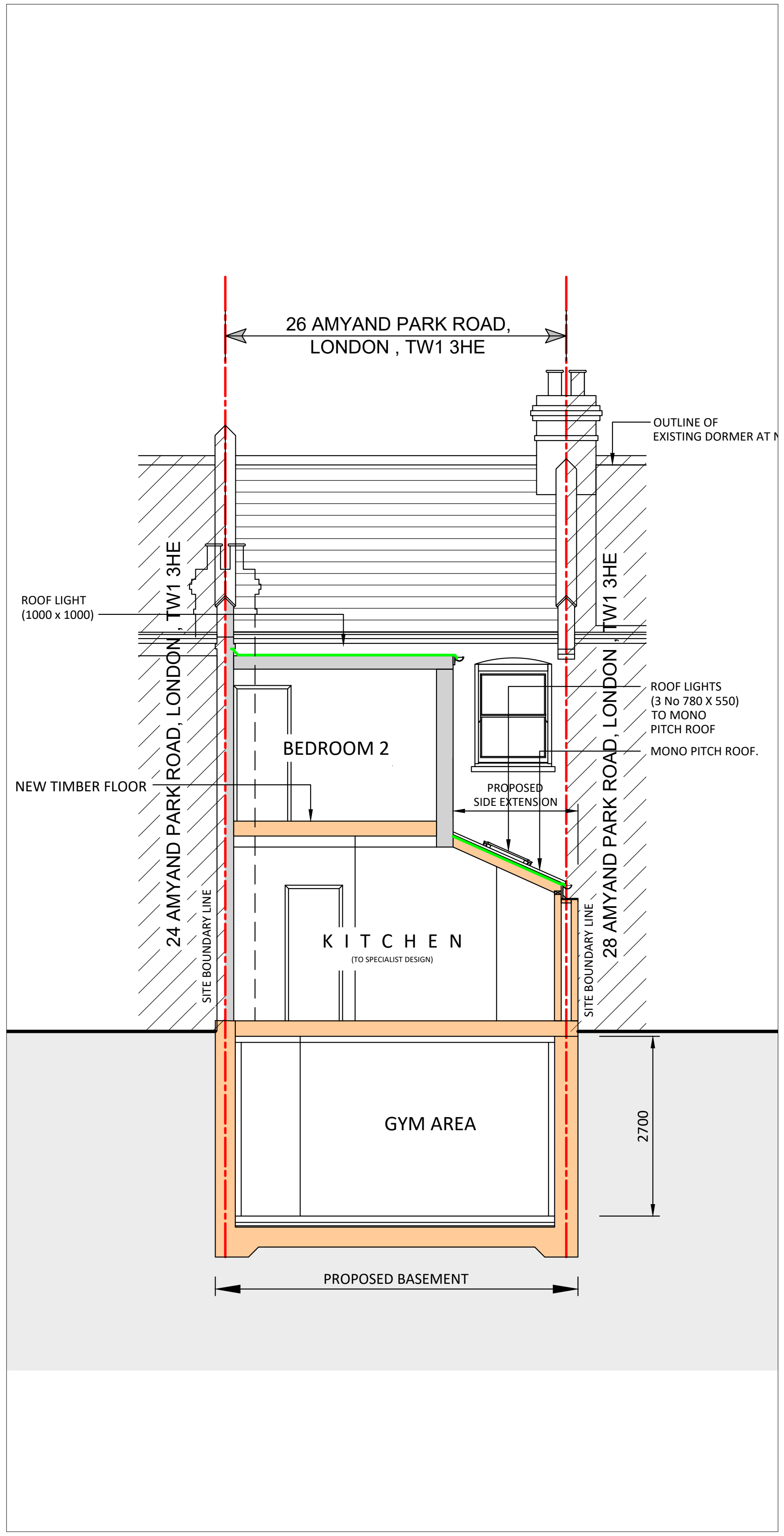
Drawing Title:	PROPOSED BASEMENT & GROUND FLOOR PLANS.
Property Address:	26 AMYAND PARK ROAD, LONDON, TW1 3HE.
Date:	MAY 2024
Scale @ A3:	1:100
Drawing Number:	SC 23111 / AP / BA01

Figure 5: Proposed Development Plan (Sections)



PROPOSED SECTION AT 'A'

SCALE 1:100 @ A3



PROPOSED SECTION AT 'B'

SCALE 1:100 @ A3

NOTES

1. ALL DIMENSIONS TO BE CHECKED ON SITE.
2. THIS DRAWING HAS BEEN DRAWN TO SCALE, AS SHOWN, FOR THE PURPOSE OF OBTAINING LOCAL AUTHORITY APPROVAL.

BAR SCALE:

1:100 @ A3

1 CM 0 1 CM 2 CM 3 CM 4 CM

1.0M 0 1.0M 2.0M 3.0M 4.0M

REVISIONS:

Drawing Title:	PROPOSED SECTIONS AT 'A' & 'B'
Property Address:	26 AMYAND PARK ROAD, LONDON, TW1 3HE.
Date:	MAY 2024
Scale @ A3:	1:100
Drawing Number:	SC 23111 / AP / BA03

APPENDIX 2 – EXPLORATORY HOLE RECORDS

CABLE PERCUSSION RECORD

JOMAS ENGINEERING ENVIRONMENTAL LAND REMEDIATION

Borehole Number **BH1**

Project Name: 26 Amyand Park Road	Client: 05 Group Ltd	Date: 10/10/2024
Location: Twickenham, TW1 3HE	Logged by: HAH/BD	
Project No. : P5802J3027	Crew Name: RD	Drilling Equipment: Cable Percussion Drilling Equipment

Log Status	Hole Type	Level	Approved By	Scale	Page Number
FINAL	CP		SC	1:50	Sheet 1 of 1

Sample Depth (m)	Type	Results
0.25	B	
0.50	B	
1.00	B	
1.00	D	
1.20	SPT	N=8 (1,0/1,2,1,4)
1.70	B	
1.70	D	
2.50	B	
2.50	D	
2.50	SPT	N=48 (3,5/9,11,14,14)
3.50	B	
3.50	SPT	50 (7,11/50 for 172mm)
4.50	B	
4.50	D	
4.50	SPT	50 (8,12/50 for 185mm)
5.50	B	
5.50	SPT	N=33 (3,4/5,9,9,10)
7.50	B	
7.50	SPT	N=16 (2,3/3,4,4,5)
8.00	U	
9.50	B	
9.50	D	
9.50	SPT	N=20 (3,3/4,5,5,6)

Water Strikes: water strike symbol at approx. 6.2 m

Depth (m)	Level (m)	Stratum Description
0.20	-0.20	Concrete. (MADE GROUND)
0.40	-0.40	Dark brown silty gravelly sand. Sand is fine to coarse. Gravel consists of fine to coarse, angular to rounded flint, brick and concrete. (MADE GROUND)
1.40	-1.40	Dark brown clayey gravelly sand. Sand is fine to medium. Gravel consists of fine to coarse, angular to rounded flint and brick. (MADE GROUND)
1.90	-1.90	Brown clayey slightly gravelly sand. Sand is fine. Gravel consists of fine to medium, sub-angular to rounded flint, with occasional brick fragments. (MADE GROUND)
7.40	-7.40	Dense to very dense orangish brown slightly clayey very sandy GRAVEL. Sand is fine to coarse. Gravel consists of fine to coarse, angular to rounded flint. (KEMPTON PARK GRAVEL MEMBER)
10.00	-10.00	Firm to stiff consistency** dark grey CLAY. (LONDON CLAY FORMATION)

End of Borehole at 10.00m

Remarks:

*Field description

**Consistency estimated using semi-empirical correlations with SPT N-values, Plasticity Indices and published literature.

Groundwater reported at 6.2mbgl during drilling and at 6.45mbgl post-drilling.

Casing Diameter by Depth			Chiselling		
Depth Top	Depth Base	Diameter	Depth Top	Depth Base	Duration

APPENDIX 3 – GEOTECHNICAL LABORATORY TEST RESULTS

Summary of Natural Moisture Content, Liquid Limit and Plastic Limit Results

Job No.	Project Name	Programme	
36184	26 Amyand Park Rd TW1 3HE	Samples received	14/10/2024
		Schedule received	14/10/2024
Project No.	Client	Project started	15/10/2024
J3027	Jomas Associates	Testing Started	23/10/2024

Hole No.	Sample				Soil Description	NMC	Passing 425µm	LL	PL	PI	Remarks
	Ref	Top m	Base m	Type		%	%	%	%	%	
BH1	-	1.70	-	B	Brown slightly gravelly very sandy silty CLAY (gravel is fmc and angular to sub-rounded)	19	62	23	15	8	Sample washed to obtain test fraction
BH1	-	9.50	-	U	High strength dark grey silty CLAY	26	95	80	24	56	

UKAS TESTING 2519

Test Methods: BS1377: Part 2: 1990:
Natural Moisture Content : clause 3.2
Atterberg Limits: clause 4.3, 4.4 and 5.0
These results only apply to the items tested

NOTE: The report shall not be reproduced except in full without authority of the laboratory

Test Report by K4 SOILS LABORATORY
Unit 8 Olds Close Olds Approach
Watford Herts WD18 9RU

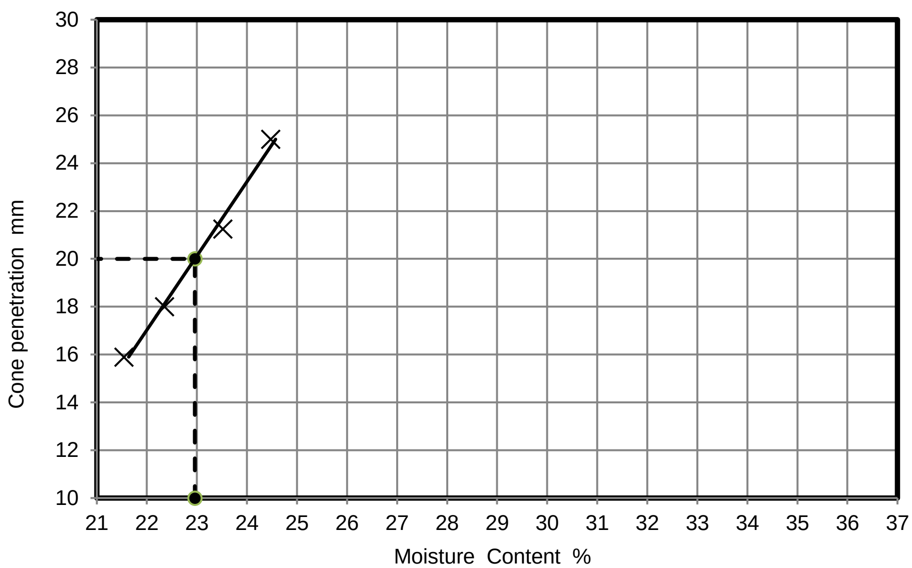
Tel: 01923 711 288
Email: James@k4soils.com

Checked and Approved
Initials **J.P**
Date: 25/10/2024

Approved Signatories: K.Phaure (Tech.Mgr) J.Phaure (Lab.Mgr)

MSF-5-R1

	LIQUID LIMIT, PLASTIC LIMIT AND PLASTICITY INDEX			Job No.	36184
				Borehole/Pit No.	BH1
Site Name	26 Amyand Park Rd TW1 3HE			Sample No.	-
Project No.	J3027	Client	Jomas Associates	Depth Top	1.70 m
Soil Description	Brown slightly gravelly very sandy silty CLAY (gravel is fmc and angular to sub-rounded)			Depth Base	- m
				Sample Type	B
				Samples received	14/10/2024
				Schedules received	14/10/2024
				Project Started	15/10/2024
				Date Tested	23/10/2024

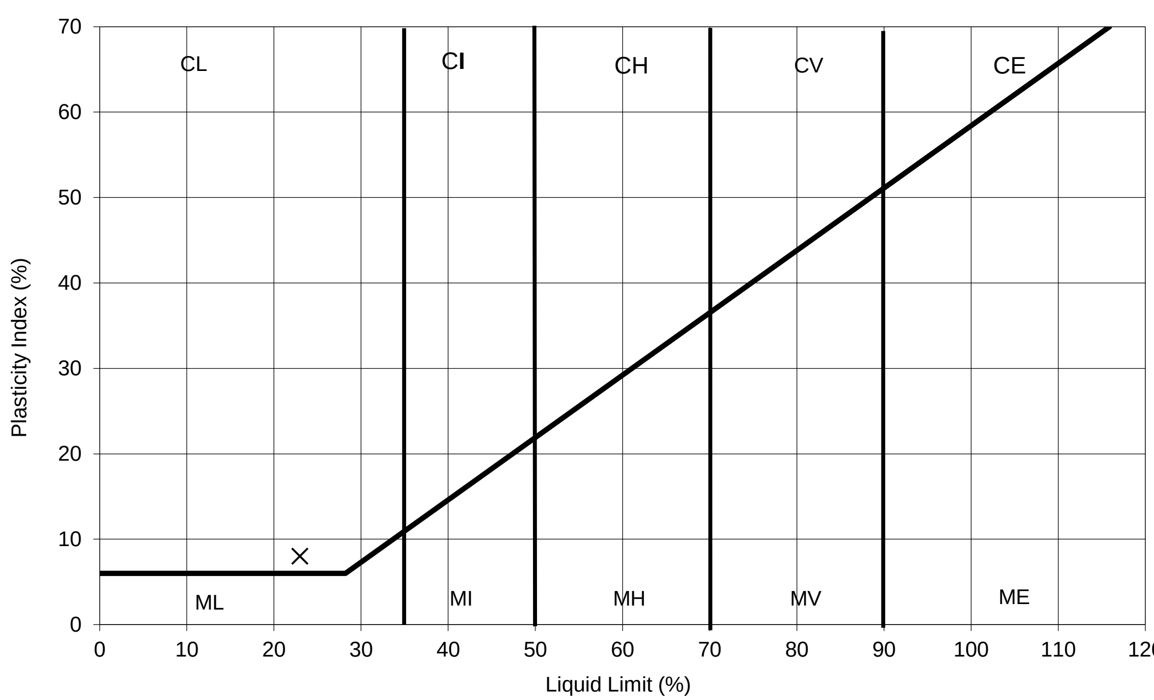


NATURAL MOISTURE CONTENT	**19**	**%**
% PASSING 425µm SIEVE	**62**	**%**
LIQUID LIMIT	**23**	**%**
PLASTIC LIMIT	**15**	**%**
PLASTICITY INDEX	**8**	**%**

Remarks

Sample washed to obtain test fraction

PLASTICITY INDEX



These results only apply to the items tested. The report shall not be reproduced except in full without authority of the laboratory

UKAS TESTING 2519

TEST METHOD
BS1377: Part 2 :Clause 4.3 : 1990 Determination of the liquid limit by the cone penetrometer method
BS1377: Part 2 :Clause 5.0 : 1990: Determination of the plastic limit and plasticity index
BS1377: Part 2 :Clause 3.2 : 1990:Determination of the moisture content by the oven drying

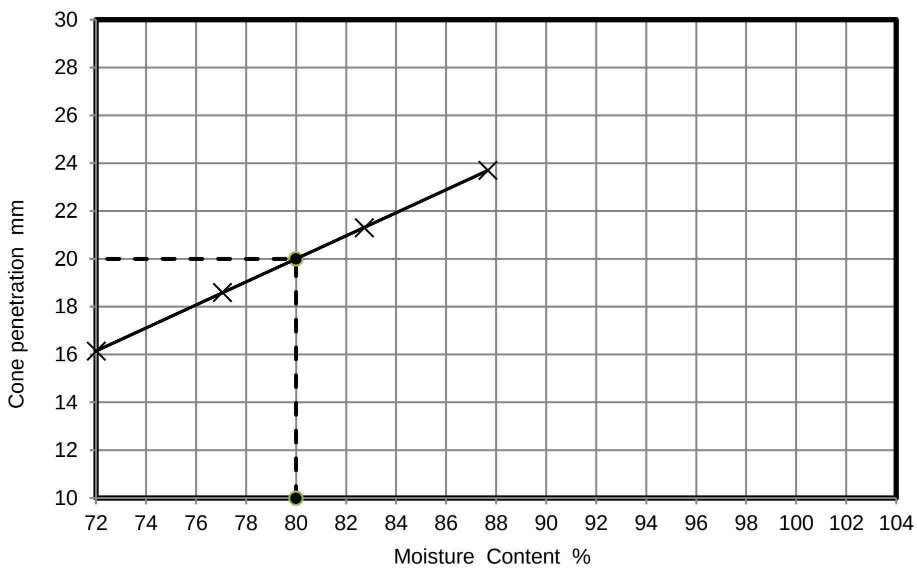
Test Report by K4 SOILS LABORATORY Unit 8 Olds Close Olds Approach Watford Herts WD18 9RU
Tel: 01923 711 288 Email: James@k4soils.com

Checked and Approved
Initials: J.P
Date: 25/10/2024

Approved Signatories: K.Phaure (Tech.Mgr) J.Phaure (Lab.Mgr)

MSF-5 R2

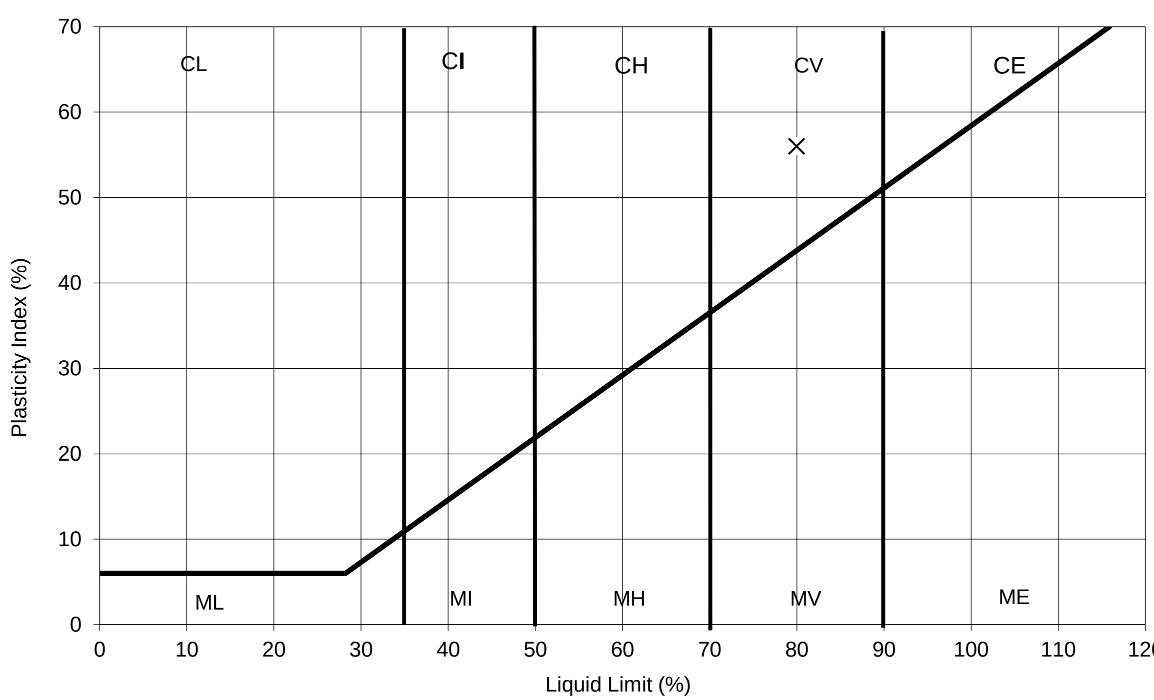
	LIQUID LIMIT, PLASTIC LIMIT AND PLASTICITY INDEX	Job No.	36184
		Borehole/Pit No.	BH1
Site Name	26 Amyand Park Rd TW1 3HE	Sample No.	-
Project No.	J3027 / Client: Jomas Associates	Depth Top	9.50 m
Soil Description	High strength dark grey silty CLAY	Depth Base	- m
		Sample Type	U
		Samples received	14/10/2024
		Schedules received	14/10/2024
		Project Started	15/10/2024
		Date Tested	23/10/2024



NATURAL MOISTURE CONTENT	26	%
% PASSING 425µm SIEVE	95	%
LIQUID LIMIT	80	%
PLASTIC LIMIT	24	%
PLASTICITY INDEX	56	%

Remarks

PLASTICITY INDEX



These results only apply to the items tested. The report shall not be reproduced except in full without authority of the laboratory

UKAS TESTING 2519

TEST METHOD
BS1377: Part 2 :Clause 4.3 : 1990 Determination of the liquid limit by the cone penetrometer method
BS1377: Part 2 :Clause 5.0 : 1990: Determination of the plastic limit and plasticity index
BS1377: Part 2 :Clause 3.2 : 1990:Determination of the moisture content by the oven drying

Test Report by K4 SOILS LABORATORY Unit 8 Olds Close Olds Approach Watford Herts WD18 9RU
Tel: 01923 711 288 Email: James@k4soils.com

Approved Signatories: K.Phaure (Tech.Mgr) J.Phaure (Lab.Mgr)

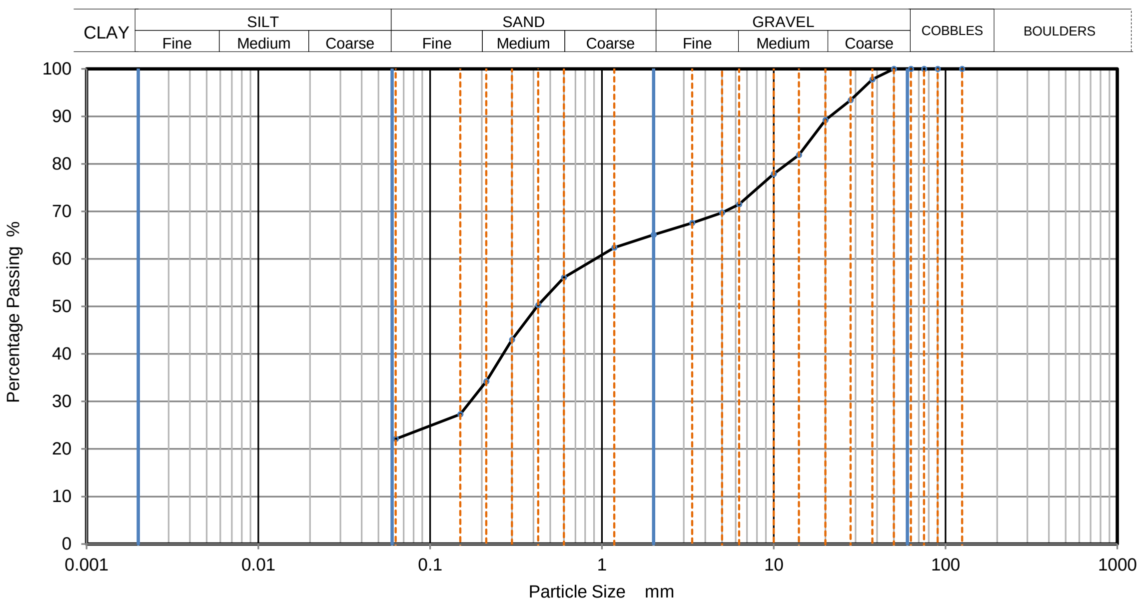
Checked and Approved
Initials: J.P
Date: 25/10/2024

MSF-5 R2

PARTICLE SIZE DISTRIBUTION

Site Name	26 Amyand Park Rd TW1 3HE			Job Ref	36184
				Borehole/Pit No.	BH1
				Sample No.	-
Project No.	J3027	Client	Jomas Associates	Depth Top	0.50 m
Soil Description	Greyish brown silty clayey very gravelly SAND with fmc brick and concrete fragments (gravel is fm and sub-rounded)			Depth Base	- m
				Sample Type	B
				Samples received	14/10/2024
				Schedules received	14/10/2024
Test Method	BS1377:Part 2: 1990, clause 9.0			Project started	15/10/2024
				Date tested	22/10/2024

These results only apply to the items tested



Sieving		Sedimentation	
Particle Size mm	% Passing	Particle Size mm	% Passing
125	100		
90	100		
75	100		
63	100		
50	100		
37.5	98		
28	93		
20	89		
14	82		
10	78		
6.3	72		
5	70		
3.35	68		
2	65		
1.18	62		
0.6	56		
0.425	50		
0.3	43		
0.212	34		
0.15	27		
0.063	22		

Sample Proportions	% dry mass
Very coarse	0
Gravel	35
Sand	43
Fines <0.063mm	22

Grading Analysis	
D100 mm	
D60 mm	0.915
D30 mm	0.172
D10 mm	
Uniformity Coefficient	
Curvature Coefficient	

Remarks

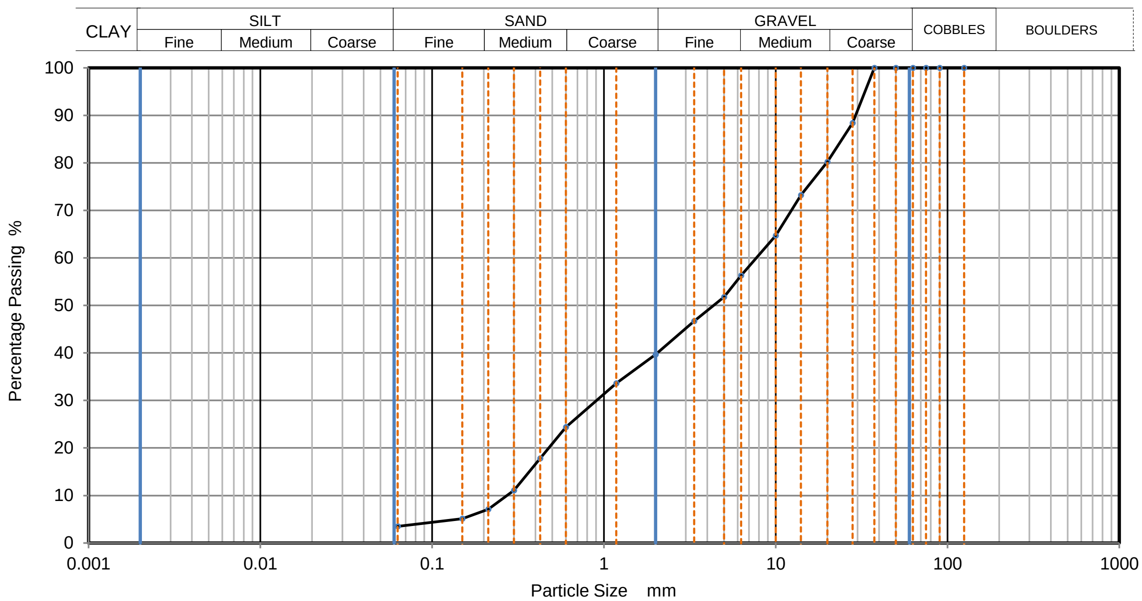
Preparation and testing in accordance with BS1377 unless noted below

NOTE: The report shall not be reproduced except in full without approval of the laboratory

PARTICLE SIZE DISTRIBUTION

Job Ref	36184		
Borehole/Pit No.	BH1		
Site Name	26 Amyand Park Rd TW1 3HE	Sample No.	-
Project No.	J3027	Client	Jomas Associates
Depth Top	2.50 m		
Depth Base	- m		
Soil Description	Brown slightly clayey very sandy GRAVEL (gravel is fmc and sub-angular to sub-rounded)	Sample Type	B
Samples received	14/10/2024		
Schedules received	14/10/2024		
Test Method	BS1377:Part 2: 1990, clause 9.0	Project started	15/10/2024
Date tested	22/10/2024		

These results only apply to the items tested



Sieving		Sedimentation	
Particle Size mm	% Passing	Particle Size mm	% Passing
125	100		
90	100		
75	100		
63	100		
50	100		
37.5	100		
28	88		
20	80		
14	73		
10	65		
6.3	56		
5	52		
3.35	47		
2	40		
1.18	34		
0.6	24		
0.425	18		
0.3	11		
0.212	7		
0.15	5		
0.063	4		

Sample Proportions	% dry mass
Very coarse	0
Gravel	60
Sand	36
Fines <0.063mm	4

Grading Analysis		
D100	mm	
D60	mm	7.72
D30	mm	0.904
D10	mm	0.272
Uniformity Coefficient		28
Curvature Coefficient		0.39

Remarks

Preparation and testing in accordance with BS1377 unless noted below

UKAS TESTING 2519

K4 Soils Laboratory

Unit 8, Olds Close, Watford, Herts, WD18 9RU

Email: james@k4soils.com

Tel: 01923 711288

Checked and Approved

Initials: J.P

Date: 25/10/2024

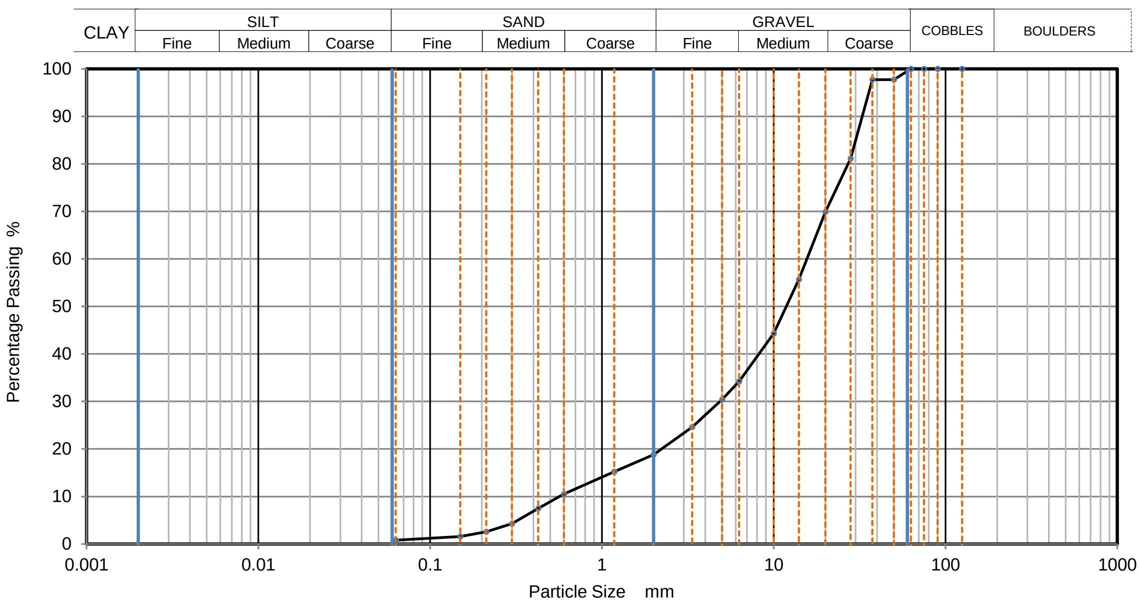
Approved Signatories: K.Phaure (Tech.Mgr) J.Phaure (Lab.Mgr)

MSF-5-R3

PARTICLE SIZE DISTRIBUTION

Site Name	26 Amyand Park Rd TW1 3HE			Job Ref	36184
Project No.	J3027	Client	Jomas Associates	Borehole/Pit No.	BH1
Soil Description	Brown slightly clayey sandy GRAVEL (gravel is fmc and sub-angular to sub-rounded)			Sample No.	-
				Depth Top	4.50 m
				Depth Base	- m
				Sample Type	B
				Samples received	14/10/2024
				Schedules received	14/10/2024
Test Method	BS1377:Part 2: 1990, clause 9.0			Project started	15/10/2024
				Date tested	22/10/2024

These results only apply to the items tested



Sieving		Sedimentation	
Particle Size mm	% Passing	Particle Size mm	% Passing
125	100		
90	100		
75	100		
63	100		
50	98		
37.5	98		
28	81		
20	70		
14	56		
10	44		
6.3	34		
5	30		
3.35	25		
2	19		
1.18	15		
0.6	11		
0.425	8		
0.3	4		
0.212	3		
0.15	2		
0.063	1		

Sample Proportions	% dry mass
Very coarse	0
Gravel	81
Sand	18
Fines <0.063mm	1

Grading Analysis	
D100 mm	
D60 mm	15.6
D30 mm	4.88
D10 mm	0.565
Uniformity Coefficient	28
Curvature Coefficient	2.7

Remarks

Preparation and testing in accordance with BS1377 unless noted below

NOTE: The report shall not be reproduced except in full without approval of the laboratory

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MSF-5-R3

Unconsolidated Undrained Triaxial Compression tests without measurement of pore pressure
Summary of Results

Tests carried out in accordance with BS1377:Part 7 : 1990 clause 8 or 9 as appropriate to test

Job No.	Project Name	Programme	
36184	26 Amyand Park Rd TW1 3HE	Samples received	14/10/2024
		Schedule received	14/10/2024
Project No.	Client	Project started	15/10/2024
J3027	Jomas Associates	Testing Started	21/10/2024

Hole No.	Sample				Soil Description	Test Type	Density		w	Length	Diameter	σ3	At failure				Remarks
	Ref	Top	Base	Type			bulk	dry					Axial strain	σ1 - σ3	cu	Mode	
		m	m				Mg/m3		%	mm	mm	kPa	%	kPa	kPa		
BH1	-	8.00	-	U	High strength dark grey silty CLAY	UU	1.97	1.56	26	198	102	190	20	170	85	C	

Legend
UU - single stage test (single and multiple specimens) — σ3 Cell pressure — Mode of failure ; B - Brittle
UUM - Multistage test on a single specimen — σ1 - σ3 Maximum corrected deviator stress — P - Plastic
suffix R - remoulded or recompacted — cu Undrained shear strength, ½ (σ1 - σ3) — C - Compound

UKAS TESTING 2519

Test Report by K4 SOILS LABORATORY
Unit 8 Olds Close Olds Approach Watford Herts WD18 9RU
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Email: james@k4soils.com

Approved Signatories: K.Phaure (Tech.Mgr) J.Phaure (Lab.Mgr)

Checked and Approved
Initials: J.P
Date: 25/10/2024

MSF-5-R7b

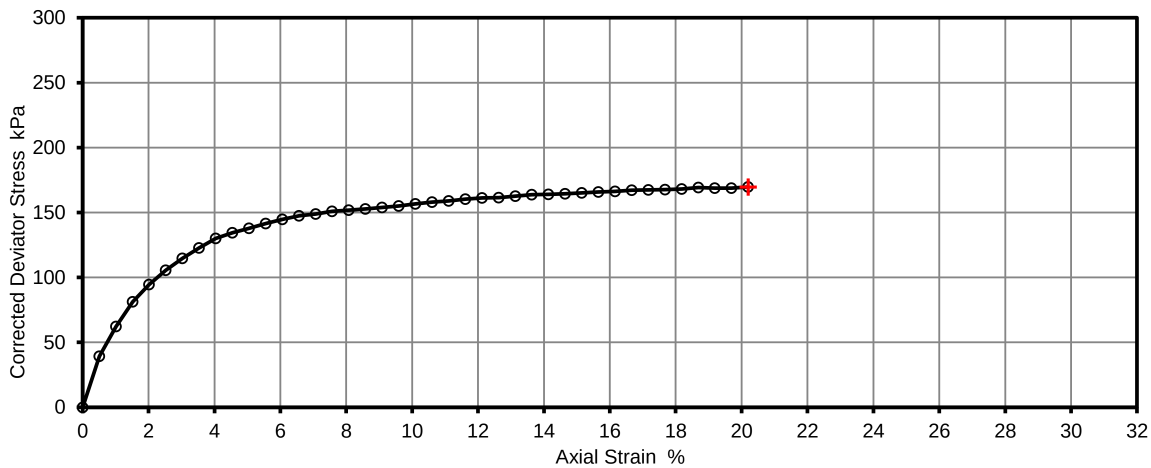
	Unconsolidated Undrained Triaxial Compression Test without measurement of pore pressure - single specimen			Job Ref	36184
				Borehole/Pit No.	BH1
Site Name	26 Amyand Park Rd TW1 3HE			Sample No.	-
Project No.	J3027	Client	Jomas Associates	Depth Top	8.00 m
Soil Description	High strength dark grey silty CLAY			Depth Base	- m
				Sample Type	U
				Samples received	14/10/2024
				Schedules received	14/10/2024
Test Method	BS1377:Part 7 : 1990 clause 8			Date of test	21/10/2024

Remarks

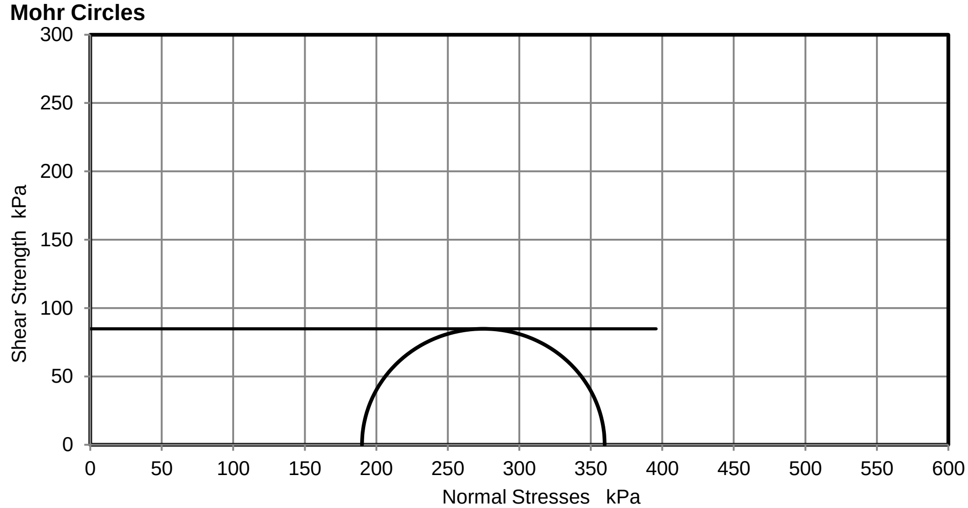
Position within sample

Test Number	1	
Length	198.0	mm
Diameter	102.0	mm
Bulk Density	1.97	Mg/m3
Moisture Content	26	%
Dry Density	1.56	Mg/m3
Rate of Strain	2.0	%/min
Cell Pressure	190	kPa
Axial Strain	20	%
Deviator Stress, (σ1 - σ3)f	170	kPa
Undrained Shear Strength, cu	85	kPa ½(σ1 - σ3)f
Mode of Failure	Compound	

Deviator Stress v Axial Strain



Mohr Circles



Deviator stress corrected for area change and membrane effects

Mohr circles and their interpretation is not covered by BS1377. This is provided for information only.

Test Report by K4 SOILS LABORATORY
Unit 8 Olds Close Olds Approach
Watford Herts WD18 9RU
Tel: 01923 711 288 Email: James@k4soils.com

Checked and Approved
Initials: J.P
Date 25/10/2024

Approved Signatories: K.Phaure (Tech.Mgr) J.Phaure (Lab.Mgr)

APPENDIX 4 – CHEMICAL LABORATORY TEST RESULTS

Hamza Hashi
Jomas Associates Limited
24 Sarum Complex
Salisbury Road
Uxbrdge
UB8 2RZ

Normec DETS Limited
Unit 1
Rose Lane Industrial Estate
Rose Lane
Lenham Heath
Kent
ME17 2JN
t: 01622 850410

DETS Report No: 24-12235

Site Reference: 26 Amyand Park Road, TW1 3HE

Project / Job Ref: J3027

Order No: P5802J3027.5

Sample Receipt Date: 15/10/2024

Sample Scheduled Date: 15/10/2024

Report Issue Number: 1

Reporting Date: 21/10/2024

Authorised by:

S. Kn

Steve Knight
Customer Support Manager

Dates of laboratory activities for each tested analyte are available upon request.

Normec DETS Limited
Unit 1, Rose Lane Industrial Estate
Rose Lane
Lenham Heath
Maidstone
Kent ME17 2JN
Tel : 01622 850410

Soil Analysis Certificate						
DETS Report No: 24-12235	**~Date Sampled**	10/10/24	10/10/24	10/10/24	10/10/24	10/10/24
Jomas Associates Limited	**~Time Sampled**	None Supplied	None Supplied	None Supplied	None Supplied	None Supplied
~Site Reference: 26 Amyand Park Road, TW1 3HE	**~TP / BH No**	BH1	BH1	BH1	BH1	BH1
~Project / Job Ref: J3027	**~Additional Refs**	Jar	Jar	Jar	Jar	Jar
~Order No: P5802J3027.5	**~Depth (m)**	1.00	1.70	2.50	4.50	9.50
Reporting Date: 21/10/2024	**DETS Sample No**	743975	743976	743977	743978	743979

Determinand	**Unit**	**RL**	**Accreditation**			(n)	(n)	
pH	pH Units	N/a	**MCERTS**	8.7	8.0	8.7	8.4	8.4
Total Sulphate as SO_4	mg/kg	< 200	**MCERTS**	1217	493	< 200	< 200	618
Total Sulphate as SO_4	%	< 0.02	**MCERTS**	0.12	0.05	< 0.02	< 0.02	0.06
W/S Sulphate as SO_4 (2:1)	mg/l	< 10	**MCERTS**	80	34	< 10	< 10	173
W/S Sulphate as SO_4 (2:1)	g/l	< 0.01	**MCERTS**	0.08	0.03	< 0.01	< 0.01	0.17
Total Sulphur	%	< 0.02	NONE	0.05	0.03	< 0.02	< 0.02	0.29

Analytical results are expressed on a dry weight basis where samples are assisted-dried at less than 30°C. The Method Description page describes if the test is performed on the dried or as-received portion
Subcontracted analysis (S)
~Sample details provided by customer and can affect the validity of results
(n) Please note we are only MCERTS accredited (UK soils only) for sand, loam and clay and any other matrix is outside our scope of accreditation

Normec DETS Limited
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Rose Lane
Lenham Heath
Maidstone
Kent ME17 2JN
Tel : 01622 850410

Soil Analysis Certificate - Sample Descriptions	
DETS Report No: 24-12235	
Jomas Associates Limited	
~Site Reference: 26 Amyand Park Road, TW1 3HE	
~Project / Job Ref: J3027	
~Order No: P5802J3027.5	
Reporting Date: 21/10/2024	

DETS Sample No	~TP / BH No	~Additional Refs	~Depth (m)	Moisture Content (%)	Sample Matrix Description
743975	BH1	Jar	1.00	13.4	Brown sandy clay with stones and brick
743976	BH1	Jar	1.70	15.9	Brown sandy clay with stones
743977	BH1	Jar	2.50	6.2	Brown sandy gravel with stones
743978	BH1	Jar	4.50	5.2	Brown sandy gravel with stones
743979	BH1	Jar	9.50	22	Brown clay

Moisture content is part of procedure E003 & is not an accredited test

Insufficient Sample I/S

Unsuitable Sample U/S

~Sample details provided by customer and can affect the validity of results

Normec DETS Limited
Unit 1, Rose Lane Industrial Estate
Rose Lane
Lenham Heath
Maidstone
Kent ME17 2JN
Tel : 01622 850410

Soil Analysis Certificate - Methodology & Miscellaneous Information
DETS Report No: 24-12235
Jomas Associates Limited
~Site Reference: 26 Amyand Park Road, TW1 3HE
~Project / Job Ref: J3027
~Order No: P5802J3027.5
Reporting Date: 21/10/2024

Matrix	Analysed On	Determinand	Brief Method Description	Method No
Soil	D	Boron - Water Soluble	Determination of water soluble boron in soil by 2:1 hot water extract followed by ICP-OES	E012
Soil	AR	BTEX	Determination of BTEX by headspace GC-MS	E001
Soil	D	Cations	Determination of cations in soil by aqua-regia digestion followed by ICP-OES	E002
Soil	D	Chloride - Water Soluble (2:1)	Determination of chloride by extraction with water & analysed by ion chromatography	E009
Soil	AR	Chromium - Hexavalent	Determination of hexavalent chromium in soil by extraction in water then by acidification, addition of 1,5 diphenylcarbazide followed by colorimetry	E016
Soil	AR	Cyanide - Complex	Determination of complex cyanide by distillation followed by colorimetry	E015
Soil	AR	Cyanide - Free	Determination of free cyanide by distillation followed by colorimetry	E015
Soil	AR	Cyanide - Total	Determination of total cyanide by distillation followed by colorimetry	E015
Soil	D	Cyclohexane Extractable Matter (CEM)	Gravimetrically determined through extraction with cyclohexane	E011
Soil	AR	Diesel Range Organics (C10 - C24)	Determination of hexane/acetone extractable hydrocarbons by GC-FID	E004
Soil	AR	Electrical Conductivity	Determination of electrical conductivity by addition of saturated calcium sulphate followed by electrometric measurement	E022
Soil	AR	Electrical Conductivity	Determination of electrical conductivity by addition of water followed by electrometric measurement	E023
Soil	D	Elemental Sulphur	Determination of elemental sulphur by solvent extraction followed by GC-MS	E020
Soil	AR	EPH (C10 – C40)	Determination of acetone/hexane extractable hydrocarbons by GC-FID	E004
Soil	AR	EPH Product ID	Determination of acetone/hexane extractable hydrocarbons by GC-FID	E004
Soil	AR	EPH TEXAS (C6-C8, C8-C10, C10-C12, C12-C16, C16-C21, C21-C40)	Determination of acetone/hexane extractable hydrocarbons by GC-FID for C8 to C40. C6 to C8 by headspace GC-MS	E004
Soil	D	Fluoride - Water Soluble	Determination of Fluoride by extraction with water & analysed by ion chromatography	E009
Soil	D	Fraction Organic Carbon (FOC)	Determination of TOC by combustion analyser.	E027
Soil	D	Organic Matter (SOM)	Determination of TOC by combustion analyser.	E027
Soil	D	TOC (Total Organic Carbon)	Determination of TOC by combustion analyser.	E027
Soil	AR	Exchangeable Ammonium	Determination of ammonium by discrete analyser.	E029
Soil	D	FOC (Fraction Organic Carbon)	Determination of fraction of organic carbon by oxidising with potassium dichromate followed by titration with iron (II) sulphate	E010
Soil	D	Loss on Ignition @ 450oC	Determination of loss on ignition in soil by gravimetrically with the sample being ignited in a muffle furnace	E019
Soil	D	Magnesium - Water Soluble	Determination of water soluble magnesium by extraction with water followed by ICP-OES	E025
Soil	D	Metals	Determination of metals by aqua-regia digestion followed by ICP-OES	E002
Soil	AR	Mineral Oil (C10 - C40)	Determination of hexane/acetone extractable hydrocarbons by GC-FID fractionating with SPE cartridge	E004
Soil	AR	Moisture Content	Moisture content; determined gravimetrically	E003
Soil	D	Nitrate - Water Soluble (2:1)	Determination of nitrate by extraction with water & analysed by ion chromatography	E009
Soil	D	Organic Matter	Determination of organic matter by oxidising with potassium dichromate followed by titration with iron (II) sulphate	E010
Soil	AR	PAH - Speciated (EPA 16)	Determination of PAH compounds by extraction in acetone and hexane followed by GC-MS with the use of surrogate and internal standards	E005
Soil	AR	PCB - 7 Congeners	Determination of PCB by extraction with acetone and hexane followed by GC-MS	E008
Soil	D	Petroleum Ether Extract (PEE)	Gravimetrically determined through extraction with petroleum ether	E011
Soil	AR	pH	Determination of pH by addition of water followed by electrometric measurement	E007
Soil	AR	Phenols - Total (monohydric)	Determination of phenols by distillation followed by colorimetry	E021
Soil	D	Phosphate - Water Soluble (2:1)	Determination of phosphate by extraction with water & analysed by ion chromatography	E009
Soil	D	Sulphate (as SO4) - Total	Determination of total sulphate by extraction with 10% HCl followed by ICP-OES	E013
Soil	D	Sulphate (as SO4) - Water Soluble (2:1)	Determination of sulphate by extraction with water & analysed by ion chromatography	E009
Soil	D	Sulphate (as SO4) - Water Soluble (2:1)	Determination of water soluble sulphate by extraction with water followed by ICP-OES	E014
Soil	AR	Sulphide	Determination of sulphide by distillation followed by colorimetry	E018
Soil	D	Sulphur - Total	Determination of total sulphur by extraction with aqua-regia followed by ICP-OES	E024
Soil	AR	SVOC	Determination of semi-volatile organic compounds by extraction in acetone and hexane followed by GC-MS	E006
Soil	AR	Thiocyanate (as SCN)	Determination of thiocyanate by extraction in caustic soda followed by acidification followed by addition of ferric nitrate followed by colorimetry	E017
Soil	D	Toluene Extractable Matter (TEM)	Gravimetrically determined through extraction with toluene	E011
Soil	D	Total Organic Carbon (TOC)	Determination of organic matter by oxidising with potassium dichromate followed by titration with iron (II) sulphate	E010
Soil	AR	TPH CWG (ali: C5- C6, C6-C8, C8-C10, C10-C12, C12-C16, C16-C21, C21-C34, aro: C5-C7, C7-C8, C8-C10, C10-C12, C12-C16, C16-C21, C21-C35)	Determination of hexane/acetone extractable hydrocarbons by GC-FID fractionating with SPE cartridge for C8 to C35. C5 to C8 by headspace GC-MS	E004
Soil	AR	TPH LQM (ali: C5-C6, C6-C8, C8-C10, C10-C12, C12-C16, C16-C35, C35-C44, aro: C5-C7, C7-C8, C8-C10, C10-C12, C12-C16, C16-C21, C21-C35, C35-C44)	Determination of hexane/acetone extractable hydrocarbons by GC-FID fractionating with SPE cartridge for C8 to C44. C5 to C8 by headspace GC-MS	E004
Soil	AR	VOCs	Determination of volatile organic compounds by headspace GC-MS	E001
Soil	AR	VPH (C6-C8 & C8-C10)	Determination of hydrocarbons C6-C8 by headspace GC-MS & C8-C10 by GC-FID	E001

D Dried
AR As Received

~Sample details provided by customer and can affect the validity of results

Normec DETS Limited
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Rose Lane
Lenham Heath
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Kent ME17 2JN
Tel : 01622 850410

List of HWOL Acronyms and Operators
DETS Report No: 24-12235
Jomas Associates Limited
~Site Reference: 26 Amyand Park Road, TW1 3HE
~Project / Job Ref: J3027
~Order No: P5802J3027.5
Reporting Date: 21/10/2024

Acronym	Description
HS	Headspace analysis
EH	Extractable Hydrocarbons - i.e. everything extracted by the solvent
CU	Clean-up - e.g. by florisil, silica gel
1D	GC - Single coil gas chromatography
2D	GC-GC - Double coil gas chromatography
Total	Aliphatics & Aromatics
AL	Aliphatics only
AR	Aromatics only
#1	EH_2D_Total but with humics mathematically subtracted
#2	EH_2D_Total but with fatty acids mathematically subtracted
_	Operator - underscore to separate acronyms (exception for +)
+	Operator to indicate cumulative eg. EH+HS_Total or EH_CU+HS_Total
~	Sample details provided by customer and can affect the validity of results

Det - Acronym

APPENDIX 5 – GROUNDWATER MONITORING RESULTS

GROUNDWATER MONITORING BOREHOLE RECORD SHEET					
Site: 26 Amyand Park Road	**Operative(s):** DJH	**Date:** 18/10/2024	**Time:** 10:00	**Round:** 1	**Page:** 1

MONITORING EQUIPMENT			
Instrument Type	**Instrument Make**	**Serial No.**	**Date Last Calibrated**
Dip Meter – Interface Probe	In-Situ	-	-

MONITORING CONDITIONS		
Weather Conditions: Overcast	**Ground Conditions:** Damp	**Temperature:** 10°C
Barometric Pressure (mbar): N/A	**Barometric Pressure Trend (24hr):** Rising	**Ambient Concentration:** N/A

MONITORING RESULTS

Monitoring Point Location	VOC (ppm)		Depth to product (mbgl)	Depth to water (mbgl)	Depth to base of well (mbgl)	Comments
	Peak	Steady				
BH1	-	-	-	6.53	8.02	

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UB8 2RZ

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N.M.N PARTNERSHIP LIMITED

Consulting Civil & Structural Engineers

9 Chamberlain Lane
Pinner
Middx HA5 2PH
Tel: 07565979671
Email: nathanmasil@aol.com

Structural Calculation for Basement and swimming pool

RC retaining walls
Basement slab
Ground floor slab
RC wals

Project 26 Amyand Park Road
Twickenham TW1 3HE

Prepared By Nathan Masil BEng, MSc, ICIOB

Date : 04.05.2024

Document: Calculation 23 227-02

Codes and standards Used:
BS8110
BS8002

Registered in England and Wales Company Number: 08789887
Registered Address: Royal Terrace, Southend-On-Sea, Essex SS1 1EA

Offices at
Southend-On-Sea Pinner

NMN Partnership Ltd

Consulting Civil & Structural Engineers

Calculation Sheet

Project:	26 Amyand Park Road TW1 3HE	Date:	01.05.24		
Element:	Basement with swimming pool	Sheet No:	1		
Ref	Existing and Proposed Drawings by Project Architect				Output

Load Assessment

Element	Dead Load (Gk)		Imposed Load (Qk)		Reference
		KN/m2		KN/m2	
Floor					
	Boards	0.1			
	Joist	0.2			
	Insulation	0.04			
	Ceiling and finishes	0.25			
	Total	0.59	Residential	1.5	
Pitch Roof					
	Tiles	0.65			
	Rafter & Battens	0.22			
	Insulation	0.04			
	Ceiling & finishes	0.25			
	Total	1.16	No Access	0.75	
Loft Floor					
	Boards	0.1			
	Joist	0.1			
	Insulation	0.04			
	Ceiling & finishes	0.25			
	Total	0.49	Accessible	0.3	
Flat roof					
	Felt	0.08			
	Joist & firrings	0.22			
	Insulation	0.04			
	Ply	0.13			
	Ceiling & finishes	0.25			
	Total	0.72	No Access	0.6	
Wall					
	0.1m single skin	2.2			
	render & finishes	0.25			
	Total	2.45			
	0.225 m solid wall	4.95			
	render & finishes	0.25			
	Total	5.2			

NMN Partnership Ltd

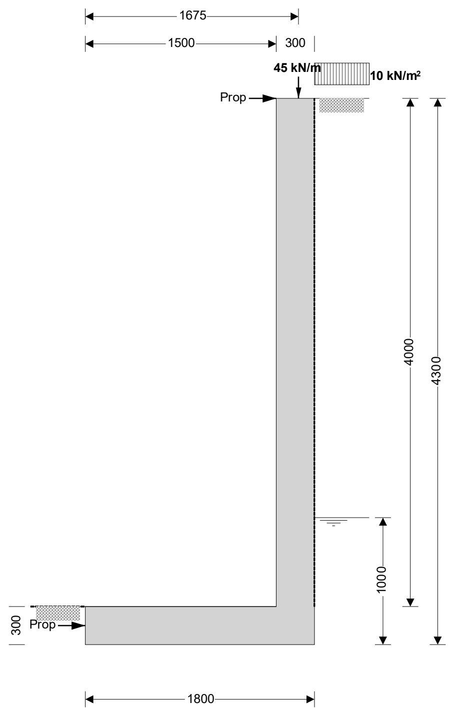
Consulting Civil & Structural Engineers Engineers

Calculation Sheet

Project:	26 Amyand Park Road TW1 3HE					Date:	01.05.24	
Element:	Basement with swimming pool					Sheet No:	2	
Ref	Existing and Proposed Drawings by Project Architect							Output
Member	Elements	Gk	KN/m	Qk		KN/m	Comments	
RW1	Lateral eth pressure only							
depth 4.6m	reaction B10		62.2			9.3		
RW2	lateral earth pressure							
Depth	Wall	5.2x9	47					
RW3	Lateral earth pressure							
Depth 1.6m	basement slab	7.2x2/2	7.2		1.5x2/2	1.5		
RW4	Wall	5.2x9	47					
Depth 3m	Lateral earth pressure							
	basement slab	7.2x2/2	7.2		1.5x2/2	1.5		
	Floors	2x0.59x5/2	2.95		2x1.5x5/2	6		
RW5	Wall	5.2x9	47					
Depth 4.6m	Floors	2x0.59x5/2	2.95		2x1.5x5/2	6		
	reaction B11		65.3			14.3		
RW6	Floors	2x0.59x5/2	2.95		2x1.5x5/2	6		
Depth 3m	Wall	5.2x9	47					
	basement floor	7.2x2/2	7.2		1.5x2/2	1.5		
RCW7	reaction B11		31.4			6		
Depth 3m								
RCW8	Wall	5.2x7	26		ddn windows 30%			
Depth 3m	Floors	2x0.59x4/2	2.36		2x15x4/2	6		
	Roof	1.16x4/2cos30	2.68		0.75x4/2Cos30	1.73		
	Front ground floor slab	6x4/2	12		5x4/2	10		
B12	Concrete ground floor	24x0.2x7.8/2	18.72		1.5x7.8/2	5.85		
Span 5m	Wall	5.2x2.4	12.48				PUDL	
	Reaction B10		62.2			9.3		
	Reaction B10		30.8			1.5		
B14	Concrete floor	24.0.2x9.6/2	23		1.5x9.6/2	7.2		
Span 3.7m								
B13	Concrete ground floor	24x0.2x7.5/2	18		1.5x7.5/2	5.62		
Span 5m	Partiiton	05x7.5/2	1.87					
B14								
Span 5m								

RETAINING WALL ANALYSIS (BS 8002:1994)

TEDDS calculation version 1.2.01.08



Wall details

Retaining wall type	**Cantilever propped at both**
Height of retaining wall stem	h_{stem} = **4000** mm
Thickness of wall stem	t_{wall} = **300** mm
Length of toe	l_{toe} = **1500** mm
Length of heel	l_{heel} = **0** mm
Overall length of base	$l_{base} = l_{toe} + l_{heel} + t_{wall}$ = **1800** mm
Thickness of base	t_{base} = **300** mm
Depth of downstand	d_{ds} = **0** mm
Position of downstand	l_{ds} = **200** mm
Thickness of downstand	t_{ds} = **300** mm
Height of retaining wall	$h_{wall} = h_{stem} + t_{base} + d_{ds}$ = **4300** mm
Depth of cover in front of wall	d_{cover} = **0** mm
Depth of unplanned excavation	d_{exc} = **0** mm
Height of ground water behind wall	h_{water} = **1000** mm
Height of saturated fill above base	$h_{sat} = \max(h_{water} - t_{base} - d_{ds}, 0 \text{ mm})$ = **700** mm
Density of wall construction	γ_{wall} = **23.6** kN/m^3
Density of base construction	γ_{base} = **23.6** kN/m^3
Angle of rear face of wall	α = **90.0** deg
Angle of soil surface behind wall	β = **0.0** deg
Effective height at virtual back of wall	$h_{eff} = h_{wall} + l_{heel} \times \tan(\beta)$ = **4300** mm

Retained material details

Mobilisation factor	M = **1.5**
Moist density of retained material	γ_m = **18.0** kN/m^3

Saturated density of retained material γ_s = **21.0** kN/m^3

Design shear strength ϕ' = **24.2** deg

Angle of wall friction δ = **0.0** deg

Base material details

Moist density γ_{mb} = **18.0** kN/m^3

Design shear strength ϕ'_b = **24.2** deg

Design base friction δ_b = **18.6** deg

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

Using Coulomb theory

Active pressure coefficient for retained material

$$K_a = \sin(\alpha + \phi')^2 / (\sin(\alpha)^2 \times \sin(\alpha - \delta) \times [1 + \sqrt{(\sin(\phi' + \delta) \times \sin(\phi' - \beta) / (\sin(\alpha - \delta) \times \sin(\alpha + \beta)))}]^2) = \mathbf{0.419}$$

Passive pressure coefficient for base material

$$K_p = \sin(90 - \phi'_b)^2 / (\sin(90 - \delta_b) \times [1 - \sqrt{(\sin(\phi'_b + \delta_b) \times \sin(\phi'_b) / (\sin(90 + \delta_b)))}]^2) = \mathbf{4.187}$$

At-rest pressure

At-rest pressure for retained material $K_0 = 1 - \sin(\phi')$ = **0.590**

Loading details

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

Applied vertical dead load on wall W_{dead} = **40.0** kN/m

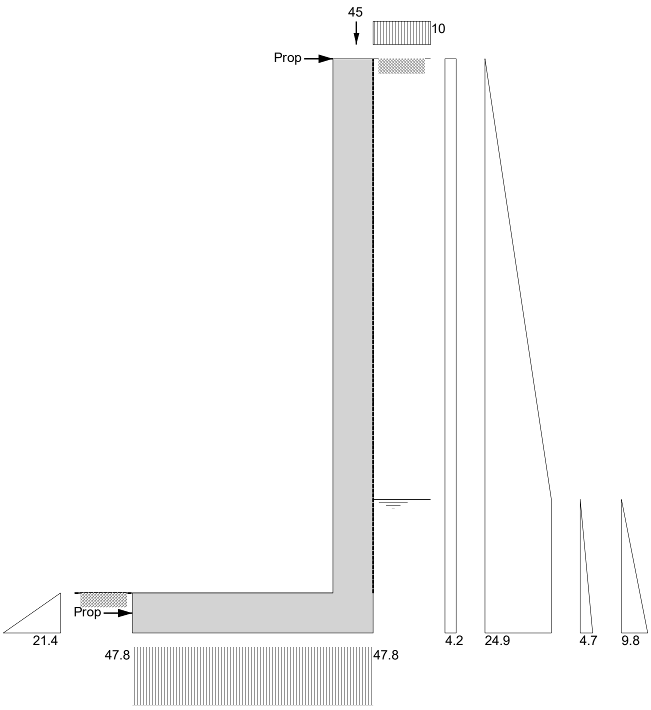
Applied vertical live load on wall W_{live} = **5.0** kN/m

Position of applied vertical load on wall l_{load} = **1675** mm

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

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

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



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

Vertical forces on wall

Wall stem $w_{wall} = h_{stem} \times t_{wall} \times \gamma_{wall}$ = **28.3** kN/m

Wall base $w_{base} = l_{base} \times t_{base} \times \gamma_{base}$ = **12.7** kN/m

Applied vertical load $W_v = W_{dead} + W_{live}$ = **45** kN/m

Total vertical load $W_{total} = w_{wall} + w_{base} + W_v$ = **86.1** kN/m

Horizontal forces on wall

Surcharge $F_{sur} = K_a \times \text{Surcharge} \times h_{eff}$ = **18** kN/m

Moist backfill above water table $F_{m\_a} = 0.5 \times K_a \times \gamma_m \times (h_{eff} - h_{water})^2$ = **41** kN/m

Moist backfill below water table $F_{m\_b} = K_a \times \gamma_m \times (h_{eff} - h_{water}) \times h_{water}$ = **24.9** kN/m

Saturated backfill $F_s = 0.5 \times K_a \times (\gamma_s - \gamma_{water}) \times h_{water}^2$ = **2.3** kN/m

Water $F_{water} = 0.5 \times h_{water}^2 \times \gamma_{water}$ = **4.9** kN/m

Total horizontal load $F_{total} = F_{sur} + F_{m\_a} + F_{m\_b} + F_s + F_{water}$ = **91.1** kN/m

Calculate total propping force

Passive resistance of soil in front of wall $F_p = 0.5 \times K_p \times \cos(\delta_b) \times (d_{cover} + t_{base} + d_{ds} - d_{exc})^2 \times \gamma_{mb}$ = **3.2** kN/m

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

F_{prop} = **60.6** kN/m

Overturning moments

Surcharge $M_{sur} = F_{sur} \times (h_{eff} - 2 \times d_{ds}) / 2$ = **38.7** kNm/m

Moist backfill above water table $M_{m\_a} = F_{m\_a} \times (h_{eff} + 2 \times h_{water} - 3 \times d_{ds}) / 3$ = **86.1** kNm/m

Moist backfill below water table $M_{m\_b} = F_{m\_b} \times (h_{water} - 2 \times d_{ds}) / 2$ = **12.4** kNm/m

Saturated backfill $M_s = F_s \times (h_{water} - 3 \times d_{ds}) / 3$ = **0.8** kNm/m

Water $M_{water} = F_{water} \times (h_{water} - 3 \times d_{ds}) / 3$ = **1.6** kNm/m

Total overturning moment $M_{ot} = M_{sur} + M_{m\_a} + M_{m\_b} + M_s + M_{water}$ = **139.7** kNm/m

Restoring moments

Wall stem $M_{wall} = w_{wall} \times (l_{toe} + t_{wall} / 2)$ = **46.7** kNm/m

Wall base $M_{base} = w_{base} \times l_{base} / 2$ = **11.5** kNm/m

Design vertical dead load $M_{dead} = W_{dead} \times l_{load}$ = **67** kNm/m

Total restoring moment $M_{rest} = M_{wall} + M_{base} + M_{dead}$ = **125.2** kNm/m

Check bearing pressure

Total vertical reaction $R = W_{total}$ = **86.1** kN/m

Distance to reaction $x_{bar} = l_{base} / 2$ = **900** mm

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

Reaction acts within middle third of base

Bearing pressure at toe $p_{toe} = (R / l_{base}) - (6 \times R \times e / l_{base}^2)$ = **47.8** kN/m^2

Bearing pressure at heel $p_{heel} = (R / l_{base}) + (6 \times R \times e / l_{base}^2)$ = **47.8** kN/m^2

PASS - Maximum bearing pressure is less than allowable bearing pressure

Calculate propping forces to top and base of wall

Propping force to top of wall

$F_{prop\_top} = (M_{ot} - M_{rest} + R \times l_{base} / 2 - F_{prop} \times t_{base} / 2) / (h_{stem} + t_{base} / 2)$ = **19.962** kN/m

Propping force to base of wall $F_{prop\_base} = F_{prop} - F_{prop\_top}$ = **40.664** kN/m

RETAINING WALL DESIGN (BS 8002:1994)

TEDDS calculation version 1.2.01.08

Ultimate limit state load factors

Dead load factor γ_{f_d} = **1.4**

Live load factor γ_{f_l} = **1.6**

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

Factored vertical forces on wall

Wall stem $w_{wall\_f} = \gamma_{f\_d} \times h_{stem} \times t_{wall} \times \gamma_{wall}$ = **39.6** kN/m

Wall base $w_{base\_f} = \gamma_{f\_d} \times l_{base} \times t_{base} \times \gamma_{base}$ = **17.8** kN/m

Applied vertical load $W_{v\_f} = \gamma_{f\_d} \times W_{dead} + \gamma_{f\_l} \times W_{live}$ = **64** kN/m

Total vertical load $W_{total\_f} = w_{wall\_f} + w_{base\_f} + W_{v\_f}$ = **121.5** kN/m

Factored horizontal active forces on wall

Surcharge $F_{sur\_f} = \gamma_{f\_l} \times K_a \times Surcharge \times h_{eff}$ = **28.8** kN/m

Moist backfill above water table $F_{m\_a\_f} = \gamma_{f\_e} \times 0.5 \times K_a \times \gamma_m \times (h_{eff} - h_{water})^2$ = **57.4** kN/m

Moist backfill below water table $F_{m\_b\_f} = \gamma_{f\_e} \times K_a \times \gamma_m \times (h_{eff} - h_{water}) \times h_{water}$ = **34.8** kN/m

Saturated backfill $F_{s\_f} = \gamma_{f\_e} \times 0.5 \times K_a \times (\gamma_s - \gamma_{water}) \times h_{water}^2$ = **3.3** kN/m

Water $F_{water\_f} = \gamma_{f\_e} \times 0.5 \times h_{water}^2 \times \gamma_{water}$ = **6.9** kN/m

Total horizontal load $F_{total\_f} = F_{sur\_f} + F_{m\_a\_f} + F_{m\_b\_f} + F_{s\_f} + F_{water\_f}$ = **131.2** kN/m

Calculate total propping force

Passive resistance of soil in front of wall $F_{p\_f} = \gamma_{f\_e} \times 0.5 \times K_p \times \cos(\delta_b) \times (d_{cover} + t_{base} + d_{ds} - d_{exc})^2 \times \gamma_{mb}$ = **4.5** kN/m

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

F_{prop_f} = **88.5** kN/m

Factored overturning moments

Surcharge $M_{sur\_f} = F_{sur\_f} \times (h_{eff} - 2 \times d_{ds}) / 2$ = **61.9** kNm/m

Moist backfill above water table $M_{m\_a\_f} = F_{m\_a\_f} \times (h_{eff} + 2 \times h_{water} - 3 \times d_{ds}) / 3$ = **120.6** kNm/m

Moist backfill below water table $M_{m\_b\_f} = F_{m\_b\_f} \times (h_{water} - 2 \times d_{ds}) / 2$ = **17.4** kNm/m

Saturated backfill $M_{s\_f} = F_{s\_f} \times (h_{water} - 3 \times d_{ds}) / 3$ = **1.1** kNm/m

Water $M_{water\_f} = F_{water\_f} \times (h_{water} - 3 \times d_{ds}) / 3$ = **2.3** kNm/m

Total overturning moment $M_{ot\_f} = M_{sur\_f} + M_{m\_a\_f} + M_{m\_b\_f} + M_{s\_f} + M_{water\_f}$ = **203.3** kNm/m

Restoring moments

Wall stem $M_{wall\_f} = w_{wall\_f} \times (l_{toe} + t_{wall} / 2)$ = **65.4** kNm/m

Wall base $M_{base\_f} = w_{base\_f} \times l_{base} / 2$ = **16.1** kNm/m

Design vertical load $M_{v\_f} = W_{v\_f} \times l_{load}$ = **107.2** kNm/m

Total restoring moment $M_{rest\_f} = M_{wall\_f} + M_{base\_f} + M_{v\_f}$ = **188.7** kNm/m

Factored bearing pressure

Total vertical reaction $R_f = W_{total\_f}$ = **121.5** kN/m

Distance to reaction $x_{bar\_f} = l_{base} / 2$ = **900** mm

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

Reaction acts within middle third of base

Bearing pressure at toe $p_{toe\_f} = (R_f / l_{base}) - (6 \times R_f \times e_f / l_{base}^2)$ = **67.5** kN/m²

Bearing pressure at heel $p_{heel\_f} = (R_f / l_{base}) + (6 \times R_f \times e_f / l_{base}^2)$ = **67.5** kN/m²

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

Bearing pressure at stem / toe $p_{stem\_toe\_f} = \max(p_{toe\_f} - (rate \times l_{toe}), 0 \text{ kN/m}^2)$ = **67.5** kN/m²

Bearing pressure at mid stem $p_{stem\_mid\_f} = \max(p_{toe\_f} - (rate \times (l_{toe} + t_{wall} / 2)), 0 \text{ kN/m}^2) = \mathbf{67.5} \text{ kN/m}^2$

Bearing pressure at stem / heel $p_{stem\_heel\_f} = \max(p_{toe\_f} - (rate \times (l_{toe} + t_{wall})), 0 \text{ kN/m}^2) = \mathbf{67.5} \text{ kN/m}^2$

Calculate propping forces to top and base of wall

Propping force to top of wall

$F_{prop\_top\_f} = (M_{ot\_f} - M_{rest\_f} + R_f \times l_{base} / 2 - F_{prop\_f} \times t_{base} / 2) / (h_{stem} + t_{base} / 2) = \mathbf{26.670} \text{ kN/m}$

Propping force to base of wall $F_{prop\_base\_f} = F_{prop\_f} - F_{prop\_top\_f} = \mathbf{61.807} \text{ kN/m}$

Design of reinforced concrete retaining wall toe (BS 8002:1994)

Material properties

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

Characteristic strength of reinforcement $f_y = \mathbf{500} \text{ N/mm}^2$

Base details

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

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

Calculate shear for toe design

Shear from bearing pressure $V_{toe\_bear} = (p_{toe\_f} + p_{stem\_toe\_f}) \times l_{toe} / 2 = \mathbf{101.2} \text{ kN/m}$

Shear from weight of base $V_{toe\_wt\_base} = \gamma_{f\_d} \times \gamma_{base} \times l_{toe} \times t_{base} = \mathbf{14.9} \text{ kN/m}$

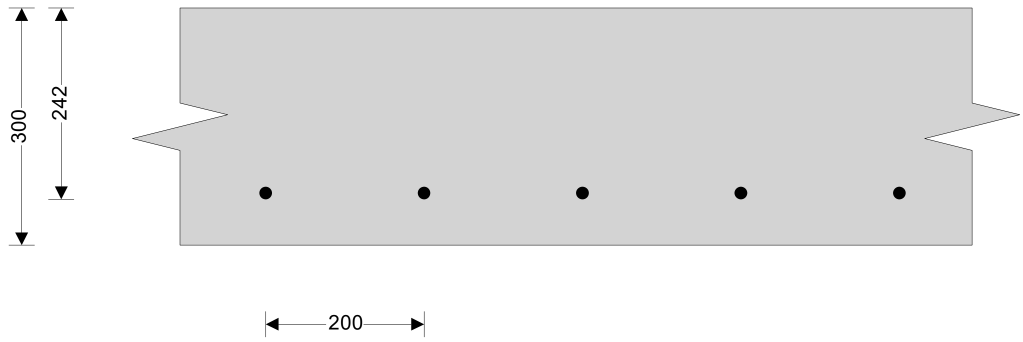
Total shear for toe design $V_{toe} = V_{toe\_bear} - V_{toe\_wt\_base} = \mathbf{86.4} \text{ kN/m}$

Calculate moment for toe design

Moment from bearing pressure $M_{toe\_bear} = (2 \times p_{toe\_f} + p_{stem\_mid\_f}) \times (l_{toe} + t_{wall} / 2)^2 / 6 = \mathbf{91.9} \text{ kNm/m}$

Moment from weight of base $M_{toe\_wt\_base} = (\gamma_{f\_d} \times \gamma_{base} \times t_{base} \times (l_{toe} + t_{wall} / 2)^2 / 2) = \mathbf{13.5} \text{ kNm/m}$

Total moment for toe design $M_{toe} = M_{toe\_bear} - M_{toe\_wt\_base} = \mathbf{78.4} \text{ kNm/m}$



Check toe in bending

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

Depth of reinforcement $d_{toe} = t_{base} - c_{toe} - (\phi_{toe} / 2) = \mathbf{242.0}$ mm

Constant $K_{toe} = M_{toe} / (b \times d_{toe}^2 \times f_{cu}) = \mathbf{0.033}$

Compression reinforcement is not required

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

$z_{toe} = \mathbf{230}$ mm

Area of tension reinforcement required $A_{s\_toe\_des} = M_{toe} / (0.87 \times f_y \times z_{toe}) = \mathbf{784} \text{ mm}^2\text{/m}$

Minimum area of tension reinforcement $A_{s\_toe\_min} = k \times b \times t_{base} = \mathbf{390} \text{ mm}^2\text{/m}$

Area of tension reinforcement required $A_{s\_toe\_req} = \text{Max}(A_{s\_toe\_des}, A_{s\_toe\_min}) = \mathbf{784} \text{ mm}^2\text{/m}$

Reinforcement provided **16 mm dia.bars @ 200 mm centres**

Area of reinforcement provided $A_{s\_toe\_prov} = \mathbf{1005} \text{ mm}^2\text{/m}$

PASS - Reinforcement provided at the retaining wall toe is adequate

Check shear resistance at toe

Design shear stress $v_{toe} = V_{toe} / (b \times d_{toe}) = \mathbf{0.357}$ N/mm^2

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

PASS - Design shear stress is less than maximum shear stress

From BS8110:Part 1:1997 – Table 3.8

Design concrete shear stress $v_{c\_toe} = \mathbf{0.625}$ N/mm^2

v_{toe} < v_{c_toe} - No shear reinforcement required

<u>Design of reinforced concrete retaining wall stem (BS 8002:1994)</u>

Material properties

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

Characteristic strength of reinforcement $f_y = \mathbf{500}$ N/mm^2

Wall details

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

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

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

Factored horizontal active forces on stem

Surcharge $F_{s\_sur\_f} = \gamma_{f\_l} \times K_a \times \text{Surcharge} \times (h_{eff} - t_{base} - d_{ds}) = \mathbf{26.8}$ kN/m

Moist backfill above water table $F_{s\_m\_a\_f} = 0.5 \times \gamma_{f\_e} \times K_a \times \gamma_m \times (h_{eff} - t_{base} - d_{ds} - h_{sat})^2 = \mathbf{57.4}$ kN/m

Moist backfill below water table $F_{s\_m\_b\_f} = \gamma_{f\_e} \times K_a \times \gamma_m \times (h_{eff} - t_{base} - d_{ds} - h_{sat}) \times h_{sat} = \mathbf{24.4}$ kN/m

Saturated backfill $F_{s\_s\_f} = 0.5 \times \gamma_{f\_e} \times K_a \times (\gamma_s - \gamma_{water}) \times h_{sat}^2 = \mathbf{1.6}$ kN/m

Water $F_{s\_water\_f} = 0.5 \times \gamma_{f\_e} \times \gamma_{water} \times h_{sat}^2 = \mathbf{3.4}$ kN/m

Calculate shear for stem design

Surcharge $V_{s\_sur\_f} = 5 \times F_{s\_sur\_f} / 8 = \mathbf{16.7}$ kN/m

Moist backfill above water table $V_{s\_m\_a\_f} = F_{s\_m\_a\_f} \times b_l \times ((5 \times L^2) - b_l^2) / (5 \times L^3) = \mathbf{39.9}$ kN/m

Moist backfill below water table $V_{s\_m\_b\_f} = F_{s\_m\_b\_f} \times (8 - (n^2 \times (4 - n))) / 8 = \mathbf{23.9}$ kN/m

Saturated backfill $V_{s\_s\_f} = F_{s\_s\_f} \times (1 - (a_l^2 \times ((5 \times L) - a_l) / (20 \times L^3))) = \mathbf{1.6}$ kN/m

Water $V_{s\_water\_f} = F_{s\_water\_f} \times (1 - (a_l^2 \times ((5 \times L) - a_l) / (20 \times L^3))) = \mathbf{3.3}$ kN/m

Total shear for stem design $V_{stem} = V_{s\_sur\_f} + V_{s\_m\_a\_f} + V_{s\_m\_b\_f} + V_{s\_s\_f} + V_{s\_water\_f} = \mathbf{85.4}$ kN/m

Calculate moment for stem design

Surcharge $M_{s\_sur} = F_{s\_sur\_f} \times L / 8 = \mathbf{13.9}$ kNm/m

Moist backfill above water table $M_{s\_m\_a} = F_{s\_m\_a\_f} \times b_l \times ((5 \times L^2) - (3 \times b_l^2)) / (15 \times L^2) = \mathbf{39.2}$ kNm/m

Moist backfill below water table $M_{s\_m\_b} = F_{s\_m\_b\_f} \times a_l \times (2 - n)^2 / 8 = \mathbf{8.3}$ kNm/m

Saturated backfill $M_{s\_s} = F_{s\_s\_f} \times a_l \times ((3 \times a_l^2) - (15 \times a_l \times L) + (20 \times L^2)) / (60 \times L^2) = \mathbf{0.4}$ kNm/m

Water $M_{s\_water} = F_{s\_water\_f} \times a_l \times ((3 \times a_l^2) - (15 \times a_l \times L) + (20 \times L^2)) / (60 \times L^2) = \mathbf{0.8}$ kNm/m

Total moment for stem design $M_{stem} = M_{s\_sur} + M_{s\_m\_a} + M_{s\_m\_b} + M_{s\_s} + M_{s\_water} = \mathbf{62.6}$ kNm/m

Calculate moment for wall design

Surcharge $M_{w\_sur} = 9 \times F_{s\_sur\_f} \times L / 128 = \mathbf{7.8}$ kNm/m

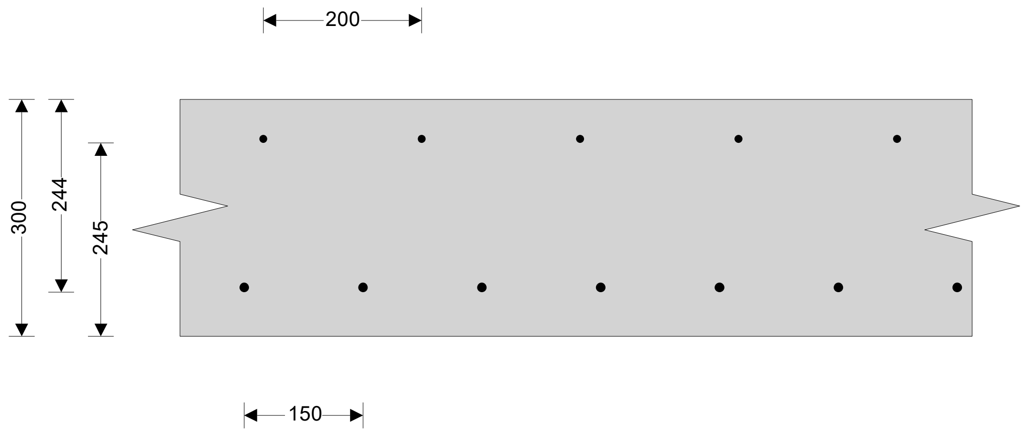
Moist backfill above water table $M_{w\_m\_a} = F_{s\_m\_a\_f} \times 0.577 \times b_l [(b_l^3 + 5 \times a_l \times L^2) / (5 \times L^3) - 0.577^2 / 3] = \mathbf{21.3}$
kNm/m

Moist backfill below water table $M_{w\_m\_b} = F_{s\_m\_b\_f} \times a_l \times [((8 - n^2 \times (4 - n))^2 / 16) - 4 + n \times (4 - n)] / 8 = \mathbf{1.6}$ kNm/m

Saturated backfill $M_{w\_s} = F_{s\_s\_f} \times [a_l^2 \times x \times ((5 \times L) - a_l) / (20 \times L^3) - (x - b_l)^3 / (3 \times a_l^2)] = \mathbf{0.1}$ kNm/m

Water $M_{w\_water} = F_{s\_water\_f} \times [a_l^2 \times x \times ((5 \times L) - a_l) / (20 \times L^3) - (x - b_l)^3 / (3 \times a_l^2)] = \mathbf{0.1}$
kNm/m

Total moment for wall design $M_{wall} = M_{w\_sur} + M_{w\_m\_a} + M_{w\_m\_b} + M_{w\_s} + M_{w\_water} = \mathbf{30.8}$ kNm/m



Check wall stem in bending

Width of wall stem $b = \mathbf{1000}$ mm/m

Depth of reinforcement $d_{stem} = t_{wall} - c_{stem} - (\phi_{stem} / 2) = \mathbf{244.0}$ mm

Constant $K_{stem} = M_{stem} / (b \times d_{stem}^2 \times f_{cu}) = \mathbf{0.026}$

Compression reinforcement is not required

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

$z_{stem} = \mathbf{232}$ mm

Area of tension reinforcement required $A_{s\_stem\_des} = M_{stem} / (0.87 \times f_y \times z_{stem}) = \mathbf{621}$ mm^2/m

Minimum area of tension reinforcement $A_{s\_stem\_min} = k \times b \times t_{wall} = \mathbf{390}$ mm^2/m

Area of tension reinforcement required $A_{s\_stem\_req} = \mathrm{Max}(A_{s\_stem\_des}, A_{s\_stem\_min}) = \mathbf{621}$ mm^2/m

Reinforcement provided **12 mm dia.bars @ 150 mm centres**

Area of reinforcement provided $A_{s\_stem\_prov} = \mathbf{754}$ mm^2/m

PASS - Reinforcement provided at the retaining wall stem is adequate

Check shear resistance at wall stem

Design shear stress $v_{stem} = V_{stem} / (b \times d_{stem}) = \mathbf{0.350}$ N/mm^2

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

PASS - Design shear stress is less than maximum shear stress

From BS8110:Part 1:1997 – Table 3.8

Design concrete shear stress $v_{c\_stem} = \mathbf{0.565}$ N/mm^2

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

Check mid height of wall in bending

Depth of reinforcement $d_{wall} = t_{wall} - c_{wall} - (\phi_{wall} / 2) = \mathbf{245.0}$ mm

Constant $K_{wall} = M_{wall} / (b \times d_{wall}^2 \times f_{cu}) = \mathbf{0.013}$

Compression reinforcement is not required

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

$z_{wall} = \mathbf{233}$ mm

Area of tension reinforcement required $A_{s\_wall\_des} = M_{wall} / (0.87 \times f_y \times z_{wall}) = \mathbf{305}$ mm^2/m

Minimum area of tension reinforcement $A_{s\_wall\_min} = k \times b \times t_{wall} = \mathbf{390}$ mm^2/m

Area of tension reinforcement required $A_{s\_wall\_req} = \mathrm{Max}(A_{s\_wall\_des}, A_{s\_wall\_min}) = \mathbf{390}$ mm^2/m

Reinforcement provided **10 mm dia.bars @ 200 mm centres**

Area of reinforcement provided $A_{s\_wall\_prov} = \mathbf{393}$ mm^2/m

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

Check retaining wall deflection

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

Design service stress $f_s = 2 \times f_y \times A_{s\_stem\_req} / (3 \times A_{s\_stem\_prov})$ = **274.6** N/mm^2

Modification factor $factor_{tens} = min(0.55 + (477 \text{ N/mm}^2 - f_s)/(120 \times (0.9 \text{ N/mm}^2 + (M_{stem}/(b \times d_{stem}^2)))),2)$ = **1.41**

Maximum span/effective depth ratio $ratio_{max} = ratio_{bas} \times factor_{tens}$ = **28.28**

Actual span/effective depth ratio $ratio_{act} = h_{stem} / d_{stem}$ = **16.39**

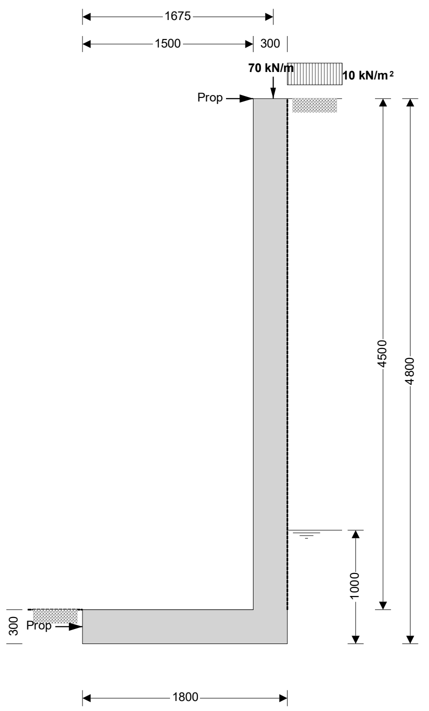
PASS - Span to depth ratio is acceptable

Indicative retaining wall reinforcement diagram

Toe bars - 16 mm dia.@ 200 mm centres - (1005 mm^2/m)
Wall bars - 10 mm dia.@ 200 mm centres - (393 mm^2/m)
Stem bars - 12 mm dia.@ 150 mm centres - (754 mm^2/m)

RETAINING WALL ANALYSIS (BS 8002:1994)

TEDDS calculation version 1.2.01.08



Wall details

Retaining wall type	**Cantilever propped at both**
Height of retaining wall stem	h_{stem} = **4500** mm
Thickness of wall stem	t_{wall} = **300** mm
Length of toe	l_{toe} = **1500** mm
Length of heel	l_{heel} = **0** mm
Overall length of base	$l_{base} = l_{toe} + l_{heel} + t_{wall}$ = **1800** mm
Thickness of base	t_{base} = **300** mm
Depth of downstand	d_{ds} = **0** mm
Position of downstand	l_{ds} = **200** mm
Thickness of downstand	t_{ds} = **300** mm
Height of retaining wall	$h_{wall} = h_{stem} + t_{base} + d_{ds}$ = **4800** mm
Depth of cover in front of wall	d_{cover} = **0** mm
Depth of unplanned excavation	d_{exc} = **0** mm
Height of ground water behind wall	h_{water} = **1000** mm
Height of saturated fill above base	$h_{sat} = \max(h_{water} - t_{base} - d_{ds}, 0 \text{ mm})$ = **700** mm
Density of wall construction	γ_{wall} = **23.6** kN/m^3
Density of base construction	γ_{base} = **23.6** kN/m^3
Angle of rear face of wall	α = **90.0** deg
Angle of soil surface behind wall	β = **0.0** deg
Effective height at virtual back of wall	$h_{eff} = h_{wall} + l_{heel} \times \tan(\beta)$ = **4800** mm

Retained material details

Mobilisation factor	M = **1.5**
Moist density of retained material	γ_m = **18.0** kN/m^3

Tekla Tedds NMN Partnership	Project 26 Amyand Park Road TW1 3HE				Job no. 23 227	
	Calcs for RC wall 2 to extension of basement and wall above				Start page no./Revision RW2 13	
	Calcs by NM	Calcs date 02/05/2024	Checked by	Checked date	Approved by	Approved date

Saturated density of retained material γ_s = **21.0** kN/m^3

Design shear strength ϕ' = **24.2** deg

Angle of wall friction δ = **0.0** deg

Base material details

Moist density γ_{mb} = **18.0** kN/m^3

Design shear strength ϕ'_b = **24.2** deg

Design base friction δ_b = **18.6** deg

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

Using Coulomb theory

Active pressure coefficient for retained material

$$K_a = \sin(\alpha + \phi')^2 / (\sin(\alpha)^2 \times \sin(\alpha - \delta) \times [1 + \sqrt{(\sin(\phi' + \delta) \times \sin(\phi' - \beta) / (\sin(\alpha - \delta) \times \sin(\alpha + \beta)))}]^2) = \mathbf{0.419}$$

Passive pressure coefficient for base material

$$K_p = \sin(90 - \phi'_b)^2 / (\sin(90 - \delta_b) \times [1 - \sqrt{(\sin(\phi'_b + \delta_b) \times \sin(\phi'_b) / (\sin(90 + \delta_b)))}]^2) = \mathbf{4.187}$$

At-rest pressure

At-rest pressure for retained material $K_0 = 1 - \sin(\phi')$ = **0.590**

Loading details

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

Applied vertical dead load on wall W_{dead} = **65.0** kN/m

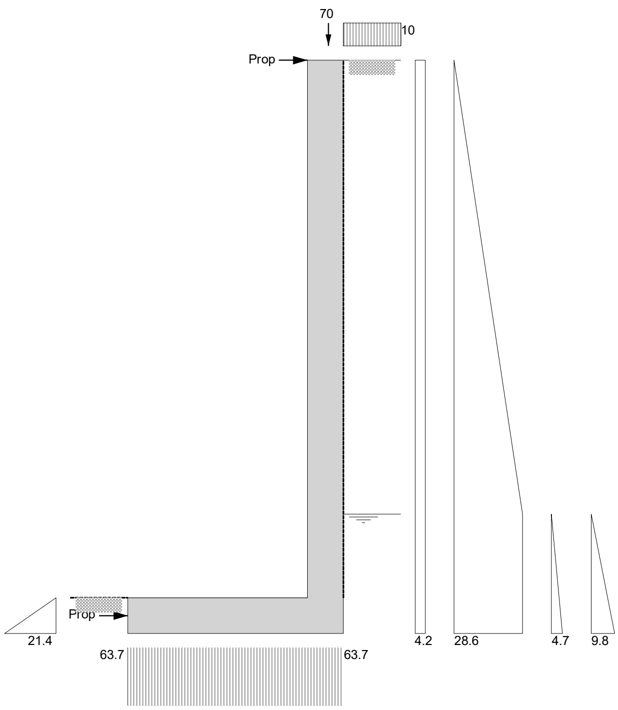
Applied vertical live load on wall W_{live} = **5.0** kN/m

Position of applied vertical load on wall l_{load} = **1675** mm

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

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

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



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

Vertical forces on wall

Wall stem $w_{wall} = h_{stem} \times t_{wall} \times \gamma_{wall}$ = **31.9** kN/m

Wall base $w_{base} = l_{base} \times t_{base} \times \gamma_{base}$ = **12.7** kN/m

Applied vertical load $W_v = W_{dead} + W_{live}$ = **70** kN/m

Total vertical load $W_{total} = w_{wall} + w_{base} + W_v$ = **114.6** kN/m

Horizontal forces on wall

Surcharge $F_{sur} = K_a \times \text{Surcharge} \times h_{eff}$ = **20.1** kN/m

Moist backfill above water table $F_{m\_a} = 0.5 \times K_a \times \gamma_m \times (h_{eff} - h_{water})^2$ = **54.4** kN/m

Moist backfill below water table $F_{m\_b} = K_a \times \gamma_m \times (h_{eff} - h_{water}) \times h_{water}$ = **28.6** kN/m

Saturated backfill $F_s = 0.5 \times K_a \times (\gamma_s - \gamma_{water}) \times h_{water}^2$ = **2.3** kN/m

Water $F_{water} = 0.5 \times h_{water}^2 \times \gamma_{water}$ = **4.9** kN/m

Total horizontal load $F_{total} = F_{sur} + F_{m\_a} + F_{m\_b} + F_s + F_{water}$ = **110.4** kN/m

Calculate total propping force

Passive resistance of soil in front of wall $F_p = 0.5 \times K_p \times \cos(\delta_b) \times (d_{cover} + t_{base} + d_{ds} - d_{exc})^2 \times \gamma_{mb}$ = **3.2** kN/m

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

F_{prop} = **70.3** kN/m

Overturning moments

Surcharge $M_{sur} = F_{sur} \times (h_{eff} - 2 \times d_{ds}) / 2$ = **48.2** kNm/m

Moist backfill above water table $M_{m\_a} = F_{m\_a} \times (h_{eff} + 2 \times h_{water} - 3 \times d_{ds}) / 3$ = **123.3** kNm/m

Moist backfill below water table $M_{m\_b} = F_{m\_b} \times (h_{water} - 2 \times d_{ds}) / 2$ = **14.3** kNm/m

Saturated backfill $M_s = F_s \times (h_{water} - 3 \times d_{ds}) / 3$ = **0.8** kNm/m

Water $M_{water} = F_{water} \times (h_{water} - 3 \times d_{ds}) / 3$ = **1.6** kNm/m

Total overturning moment $M_{ot} = M_{sur} + M_{m\_a} + M_{m\_b} + M_s + M_{water}$ = **188.2** kNm/m

Restoring moments

Wall stem $M_{wall} = w_{wall} \times (l_{toe} + t_{wall} / 2)$ = **52.6** kNm/m

Wall base $M_{base} = w_{base} \times l_{base} / 2$ = **11.5** kNm/m

Design vertical dead load $M_{dead} = W_{dead} \times l_{load}$ = **108.9** kNm/m

Total restoring moment $M_{rest} = M_{wall} + M_{base} + M_{dead}$ = **172.9** kNm/m

Check bearing pressure

Total vertical reaction $R = W_{total}$ = **114.6** kN/m

Distance to reaction $x_{bar} = l_{base} / 2$ = **900** mm

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

Reaction acts within middle third of base

Bearing pressure at toe $p_{toe} = (R / l_{base}) - (6 \times R \times e / l_{base}^2)$ = **63.7** kN/m^2

Bearing pressure at heel $p_{heel} = (R / l_{base}) + (6 \times R \times e / l_{base}^2)$ = **63.7** kN/m^2

PASS - Maximum bearing pressure is less than allowable bearing pressure

Calculate propping forces to top and base of wall

Propping force to top of wall

$F_{prop\_top} = (M_{ot} - M_{rest} + R \times l_{base} / 2 - F_{prop} \times t_{base} / 2) / (h_{stem} + t_{base} / 2)$ = **23.208** kN/m

Propping force to base of wall $F_{prop\_base} = F_{prop} - F_{prop\_top}$ = **47.044** kN/m

RETAINING WALL DESIGN (BS 8002:1994)

TEDDS calculation version 1.2.01.08

Ultimate limit state load factors

Dead load factor γ_{f_d} = **1.4**

Live load factor γ_{f_l} = **1.6**

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

Factored vertical forces on wall

Wall stem $w_{wall\_f} = \gamma_{f\_d} \times h_{stem} \times t_{wall} \times \gamma_{wall}$ = **44.6** kN/m

Wall base $w_{base\_f} = \gamma_{f\_d} \times l_{base} \times t_{base} \times \gamma_{base}$ = **17.8** kN/m

Applied vertical load $W_{v\_f} = \gamma_{f\_d} \times W_{dead} + \gamma_{f\_l} \times W_{live}$ = **99** kN/m

Total vertical load $W_{total\_f} = w_{wall\_f} + w_{base\_f} + W_{v\_f}$ = **161.4** kN/m

Factored horizontal active forces on wall

Surcharge $F_{sur\_f} = \gamma_{f\_l} \times K_a \times \text{Surcharge} \times h_{eff}$ = **32.1** kN/m

Moist backfill above water table $F_{m\_a\_f} = \gamma_{f\_e} \times 0.5 \times K_a \times \gamma_m \times (h_{eff} - h_{water})^2$ = **76.1** kN/m

Moist backfill below water table $F_{m\_b\_f} = \gamma_{f\_e} \times K_a \times \gamma_m \times (h_{eff} - h_{water}) \times h_{water}$ = **40.1** kN/m

Saturated backfill $F_{s\_f} = \gamma_{f\_e} \times 0.5 \times K_a \times (\gamma_s - \gamma_{water}) \times h_{water}^2$ = **3.3** kN/m

Water $F_{water\_f} = \gamma_{f\_e} \times 0.5 \times h_{water}^2 \times \gamma_{water}$ = **6.9** kN/m

Total horizontal load $F_{total\_f} = F_{sur\_f} + F_{m\_a\_f} + F_{m\_b\_f} + F_{s\_f} + F_{water\_f}$ = **158.5** kN/m

Calculate total propping force

Passive resistance of soil in front of wall $F_{p\_f} = \gamma_{f\_e} \times 0.5 \times K_p \times \cos(\delta_b) \times (d_{cover} + t_{base} + d_{ds} - d_{exc})^2 \times \gamma_{mb}$ = **4.5** kN/m

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

F_{prop_f} = **102.4** kN/m

Factored overturning moments

Surcharge $M_{sur\_f} = F_{sur\_f} \times (h_{eff} - 2 \times d_{ds}) / 2$ = **77.1** kNm/m

Moist backfill above water table $M_{m\_a\_f} = F_{m\_a\_f} \times (h_{eff} + 2 \times h_{water} - 3 \times d_{ds}) / 3$ = **172.6** kNm/m

Moist backfill below water table $M_{m\_b\_f} = F_{m\_b\_f} \times (h_{water} - 2 \times d_{ds}) / 2$ = **20** kNm/m

Saturated backfill $M_{s\_f} = F_{s\_f} \times (h_{water} - 3 \times d_{ds}) / 3$ = **1.1** kNm/m

Water $M_{water\_f} = F_{water\_f} \times (h_{water} - 3 \times d_{ds}) / 3$ = **2.3** kNm/m

Total overturning moment $M_{ot\_f} = M_{sur\_f} + M_{m\_a\_f} + M_{m\_b\_f} + M_{s\_f} + M_{water\_f}$ = **273.2** kNm/m

Restoring moments

Wall stem $M_{wall\_f} = w_{wall\_f} \times (l_{toe} + t_{wall} / 2)$ = **73.6** kNm/m

Wall base $M_{base\_f} = w_{base\_f} \times l_{base} / 2$ = **16.1** kNm/m

Design vertical load $M_{v\_f} = W_{v\_f} \times l_{load}$ = **165.8** kNm/m

Total restoring moment $M_{rest\_f} = M_{wall\_f} + M_{base\_f} + M_{v\_f}$ = **255.5** kNm/m

Factored bearing pressure

Total vertical reaction $R_f = W_{total\_f}$ = **161.4** kN/m

Distance to reaction $x_{bar\_f} = l_{base} / 2$ = **900** mm

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

Reaction acts within middle third of base

Bearing pressure at toe $p_{toe\_f} = (R_f / l_{base}) - (6 \times R_f \times e_f / l_{base}^2)$ = **89.7** kN/m²

Bearing pressure at heel $p_{heel\_f} = (R_f / l_{base}) + (6 \times R_f \times e_f / l_{base}^2)$ = **89.7** kN/m²

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

Bearing pressure at stem / toe $p_{stem\_toe\_f} = \max(p_{toe\_f} - (\text{rate} \times l_{toe}), 0 \text{ kN/m}^2)$ = **89.7** kN/m²

Bearing pressure at mid stem $p_{stem\_mid\_f} = \max(p_{toe\_f} - (rate \times (l_{toe} + t_{wall} / 2)), 0\ kN/m^2) =$ **89.7** kN/m²

Bearing pressure at stem / heel $p_{stem\_heel\_f} = \max(p_{toe\_f} - (rate \times (l_{toe} + t_{wall})), 0\ kN/m^2) =$ **89.7** kN/m²

Calculate propping forces to top and base of wall

Propping force to top of wall

$F_{prop\_top\_f} = (M_{ot\_f} - M_{rest\_f} + R_f \times l_{base} / 2 - F_{prop\_f} \times t_{base} / 2) / (h_{stem} + t_{base} / 2) =$ **31.748** kN/m

Propping force to base of wall $F_{prop\_base\_f} = F_{prop\_f} - F_{prop\_top\_f} =$ **70.624** kN/m

Design of reinforced concrete retaining wall toe (BS 8002:1994)

Material properties

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

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

Base details

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

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

Calculate shear for toe design

Shear from bearing pressure $V_{toe\_bear} = (p_{toe\_f} + p_{stem\_toe\_f}) \times l_{toe} / 2 =$ **134.5** kN/m

Shear from weight of base $V_{toe\_wt\_base} = \gamma_{f\_d} \times \gamma_{base} \times l_{toe} \times t_{base} =$ **14.9** kN/m

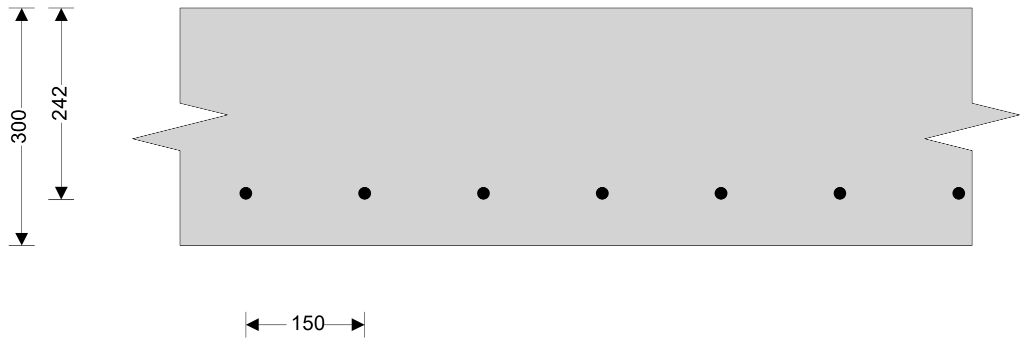
Total shear for toe design $V_{toe} = V_{toe\_bear} - V_{toe\_wt\_base} =$ **119.7** kN/m

Calculate moment for toe design

Moment from bearing pressure $M_{toe\_bear} = (2 \times p_{toe\_f} + p_{stem\_mid\_f}) \times (l_{toe} + t_{wall} / 2)^2 / 6 =$ **122.1** kNm/m

Moment from weight of base $M_{toe\_wt\_base} = (\gamma_{f\_d} \times \gamma_{base} \times t_{base} \times (l_{toe} + t_{wall} / 2)^2 / 2) =$ **13.5** kNm/m

Total moment for toe design $M_{toe} = M_{toe\_bear} - M_{toe\_wt\_base} =$ **108.6** kNm/m



Check toe in bending

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

Depth of reinforcement $d_{toe} = t_{base} - c_{toe} - (\phi_{toe} / 2) =$ **242.0** mm

Constant $K_{toe} = M_{toe} / (b \times d_{toe}^2 \times f_{cu}) =$ **0.046**

Compression reinforcement is not required

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

$z_{toe} =$ **229** mm

Area of tension reinforcement required $As_{toe\_des} = M_{toe} / (0.87 \times f_y \times z_{toe}) =$ **1091** mm²/m

Minimum area of tension reinforcement $As_{toe\_min} = k \times b \times t_{base} =$ **390** mm²/m

Area of tension reinforcement required $As_{toe\_req} = \mathrm{Max}(As_{toe\_des}, As_{toe\_min}) =$ **1091** mm²/m

Reinforcement provided **16 mm dia.bars @ 150 mm centres**

Area of reinforcement provided $As_{toe\_prov} =$ **1340** mm²/m

PASS - Reinforcement provided at the retaining wall toe is adequate

Check shear resistance at toe

Design shear stress — $v_{toe} = V_{toe} / (b \times d_{toe})$ = **0.495** N/mm^2

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

PASS - Design shear stress is less than maximum shear stress

From BS8110:Part 1:1997 – Table 3.8

Design concrete shear stress — v_{c_toe} = **0.688** N/mm^2

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

Design of reinforced concrete retaining wall stem (BS 8002:1994)

Material properties

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

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

Wall details

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

Cover to reinforcement in stem — c_{stem} = **50** mm

Cover to reinforcement in wall — c_{wall} = **50** mm

Factored horizontal active forces on stem

Surcharge — $F_{s\_sur\_f} = \gamma_{f\_l} \times K_a \times \text{Surcharge} \times (h_{eff} - t_{base} - d_{ds})$ = **30.1** kN/m

Moist backfill above water table — $F_{s\_m\_a\_f} = 0.5 \times \gamma_{f\_e} \times K_a \times \gamma_m \times (h_{eff} - t_{base} - d_{ds} - h_{sat})^2$ = **76.1** kN/m

Moist backfill below water table — $F_{s\_m\_b\_f} = \gamma_{f\_e} \times K_a \times \gamma_m \times (h_{eff} - t_{base} - d_{ds} - h_{sat}) \times h_{sat}$ = **28.1** kN/m

Saturated backfill — $F_{s\_s\_f} = 0.5 \times \gamma_{f\_e} \times K_a \times (\gamma_s - \gamma_{water}) \times h_{sat}^2$ = **1.6** kN/m

Water — $F_{s\_water\_f} = 0.5 \times \gamma_{f\_e} \times \gamma_{water} \times h_{sat}^2$ = **3.4** kN/m

Calculate shear for stem design

Surcharge — $V_{s\_sur\_f} = 5 \times F_{s\_sur\_f} / 8$ = **18.8** kN/m

Moist backfill above water table — $V_{s\_m\_a\_f} = F_{s\_m\_a\_f} \times b_l \times ((5 \times L^2) - b_l^2) / (5 \times L^3)$ = **53.9** kN/m

Moist backfill below water table — $V_{s\_m\_b\_f} = F_{s\_m\_b\_f} \times (8 - (n^2 \times (4 - n))) / 8$ = **27.6** kN/m

Saturated backfill — $V_{s\_s\_f} = F_{s\_s\_f} \times (1 - (a_l^2 \times ((5 \times L) - a_l) / (20 \times L^3)))$ = **1.6** kN/m

Water — $V_{s\_water\_f} = F_{s\_water\_f} \times (1 - (a_l^2 \times ((5 \times L) - a_l) / (20 \times L^3)))$ = **3.3** kN/m

Total shear for stem design — $V_{stem} = V_{s\_sur\_f} + V_{s\_m\_a\_f} + V_{s\_m\_b\_f} + V_{s\_s\_f} + V_{s\_water\_f}$ = **105.3** kN/m

Calculate moment for stem design

Surcharge — $M_{s\_sur} = F_{s\_sur\_f} \times L / 8$ = **17.5** kNm/m

Moist backfill above water table — $M_{s\_m\_a} = F_{s\_m\_a\_f} \times b_l \times ((5 \times L^2) - (3 \times b_l^2)) / (15 \times L^2)$ = **57.8** kNm/m

Moist backfill below water table — $M_{s\_m\_b} = F_{s\_m\_b\_f} \times a_l \times (2 - n)^2 / 8$ = **9.8** kNm/m

Saturated backfill — $M_{s\_s} = F_{s\_s\_f} \times a_l \times ((3 \times a_l^2) - (15 \times a_l \times L) + (20 \times L^2)) / (60 \times L^2)$ = **0.4** kNm/m

Water — $M_{s\_water} = F_{s\_water\_f} \times a_l \times ((3 \times a_l^2) - (15 \times a_l \times L) + (20 \times L^2)) / (60 \times L^2)$ = **0.8** kNm/m

Total moment for stem design — $M_{stem} = M_{s\_sur} + M_{s\_m\_a} + M_{s\_m\_b} + M_{s\_s} + M_{s\_water}$ = **86.4** kNm/m

Calculate moment for wall design

Surcharge — $M_{w\_sur} = 9 \times F_{s\_sur\_f} \times L / 128$ = **9.9** kNm/m

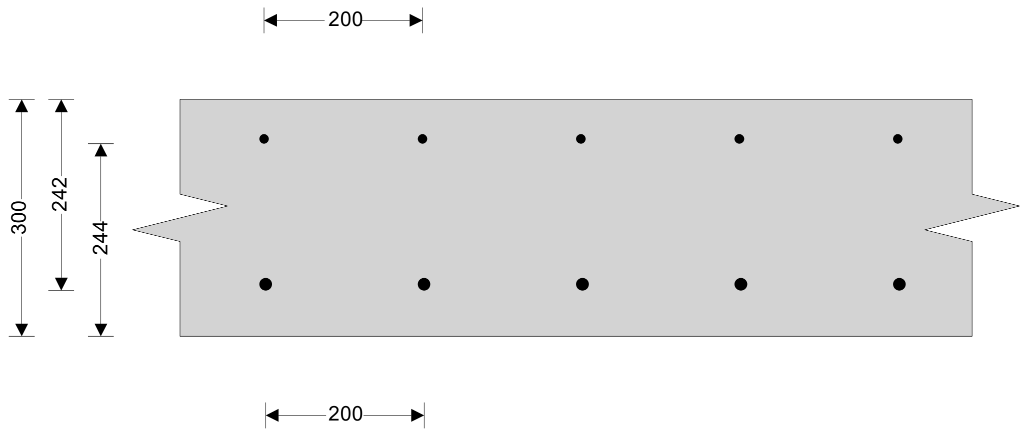
Moist backfill above water table — $M_{w\_m\_a} = F_{s\_m\_a\_f} \times 0.577 \times b_l \times [(b_l^3 + 5 \times a_l \times L^2) / (5 \times L^3) - 0.577^2 / 3]$ = **30.2** kNm/m

Moist backfill below water table — $M_{w\_m\_b} = F_{s\_m\_b\_f} \times a_l \times [((8 - n^2 \times (4 - n))^2 / 16) - 4 + n \times (4 - n)] / 8$ = **1.7** kNm/m

Saturated backfill — $M_{w\_s} = F_{s\_s\_f} \times [a_l^2 \times x \times ((5 \times L) - a_l) / (20 \times L^3) - (x - b_l)^3 / (3 \times a_l^2)]$ = **0** kNm/m

Water — $M_{w\_water} = F_{s\_water\_f} \times [a_l^2 \times x \times ((5 \times L) - a_l) / (20 \times L^3) - (x - b_l)^3 / (3 \times a_l^2)]$ = **0.1** kNm/m

Total moment for wall design — $M_{wall} = M_{w\_sur} + M_{w\_m\_a} + M_{w\_m\_b} + M_{w\_s} + M_{w\_water}$ = **41.9** kNm/m



Check wall stem in bending

Width of wall stem — $b = \mathbf{1000}$ mm/m

Depth of reinforcement — $d_{stem} = t_{wall} - c_{stem} - (\phi_{stem} / 2) = \mathbf{242.0}$ mm

Constant — $K_{stem} = M_{stem} / (b \times d_{stem}^2 \times f_{cu}) = \mathbf{0.037}$

Compression reinforcement is not required

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

$z_{stem} = \mathbf{230}$ mm

Area of tension reinforcement required — $A_{s\_stem\_des} = M_{stem} / (0.87 \times f_y \times z_{stem}) = \mathbf{864}$ mm²/m

Minimum area of tension reinforcement — $A_{s\_stem\_min} = k \times b \times t_{wall} = \mathbf{390}$ mm²/m

Area of tension reinforcement required — $A_{s\_stem\_req} = \mathrm{Max}(A_{s\_stem\_des}, A_{s\_stem\_min}) = \mathbf{864}$ mm²/m

Reinforcement provided — **16 mm dia.bars @ 200 mm centres**

Area of reinforcement provided — $A_{s\_stem\_prov} = \mathbf{1005}$ mm²/m

PASS - Reinforcement provided at the retaining wall stem is adequate

Check shear resistance at wall stem

Design shear stress — $v_{stem} = V_{stem} / (b \times d_{stem}) = \mathbf{0.435}$ N/mm²

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

PASS - Design shear stress is less than maximum shear stress

From BS8110:Part 1:1997 – Table 3.8

Design concrete shear stress — $v_{c\_stem} = \mathbf{0.625}$ N/mm²

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

Check mid height of wall in bending

Depth of reinforcement — $d_{wall} = t_{wall} - c_{wall} - (\phi_{wall} / 2) = \mathbf{244.0}$ mm

Constant — $K_{wall} = M_{wall} / (b \times d_{wall}^2 \times f_{cu}) = \mathbf{0.018}$

Compression reinforcement is not required

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

$z_{wall} = \mathbf{232}$ mm

Area of tension reinforcement required — $A_{s\_wall\_des} = M_{wall} / (0.87 \times f_y \times z_{wall}) = \mathbf{416}$ mm²/m

Minimum area of tension reinforcement — $A_{s\_wall\_min} = k \times b \times t_{wall} = \mathbf{390}$ mm²/m

Area of tension reinforcement required — $A_{s\_wall\_req} = \mathrm{Max}(A_{s\_wall\_des}, A_{s\_wall\_min}) = \mathbf{416}$ mm²/m

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

Area of reinforcement provided — $A_{s\_wall\_prov} = \mathbf{565}$ mm²/m

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

Check retaining wall deflection

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

Design service stress $f_s = 2 \times f_y \times A_{s\_stem\_req} / (3 \times A_{s\_stem\_prov})$ = **286.4** N/mm^2

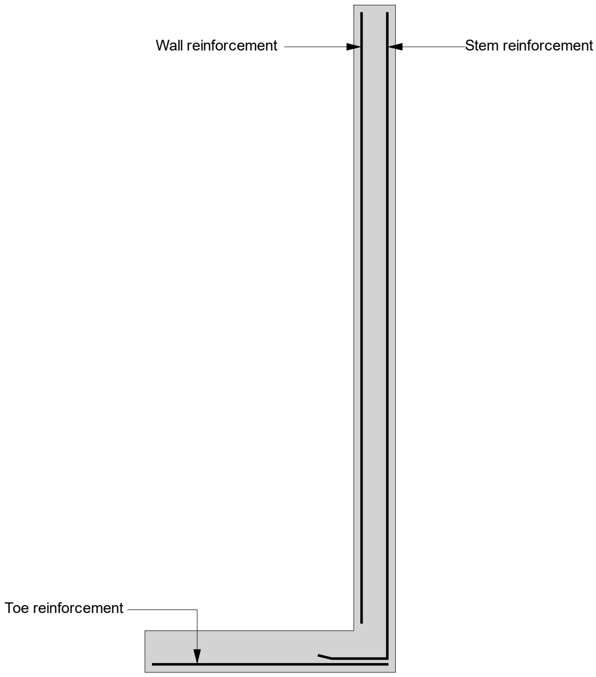
Modification factor $factor_{tens} = \min(0.55 + (477\ N/mm^2 - f_s)/(120 \times (0.9\ N/mm^2 + (M_{stem}/(b \times d_{stem}^2)))),2)$ = **1.22**

Maximum span/effective depth ratio $ratio_{max} = ratio_{bas} \times factor_{tens}$ = **24.37**

Actual span/effective depth ratio $ratio_{act} = h_{stem} / d_{stem}$ = **18.60**

PASS - Span to depth ratio is acceptable

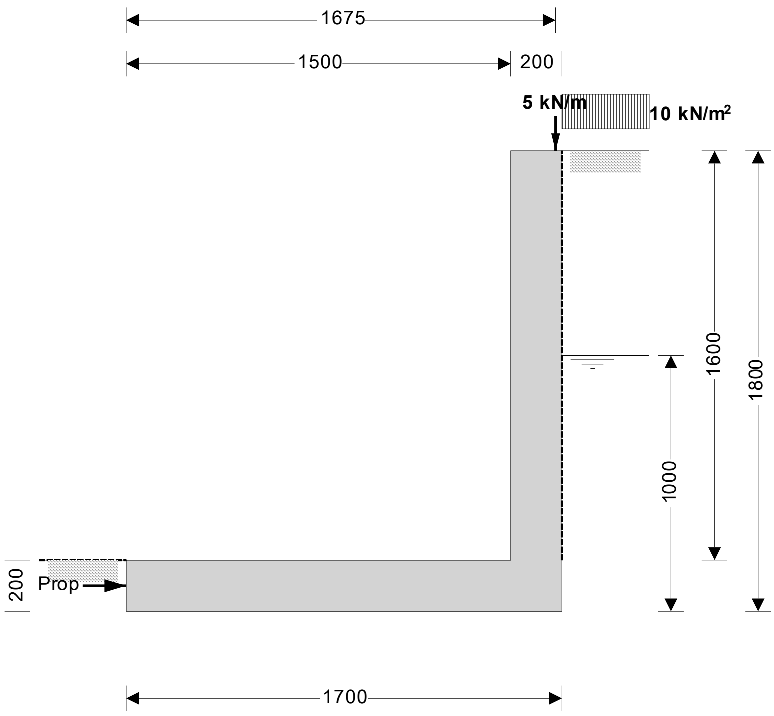
Indicative retaining wall reinforcement diagram



Toe bars - 16 mm dia.@ 150 mm centres - (1340 mm^2/m)
Wall bars - 12 mm dia.@ 200 mm centres - (565 mm^2/m)
Stem bars - 16 mm dia.@ 200 mm centres - (1005 mm^2/m)

RETAINING WALL ANALYSIS (BS 8002:1994)

TEDDS calculation version 1.2.01.08



Wall details

Retaining wall type	**Cantilever propped at base**
Height of retaining wall stem	h_{stem} = **1600** mm
Thickness of wall stem	t_{wall} = **200** mm
Length of toe	l_{toe} = **1500** mm
Length of heel	l_{heel} = **0** mm
Overall length of base	$l_{base} = l_{toe} + l_{heel} + t_{wall}$ = **1700** mm
Thickness of base	t_{base} = **200** mm
Depth of downstand	d_{ds} = **0** mm
Position of downstand	l_{ds} = **200** mm
Thickness of downstand	t_{ds} = **200** mm
Height of retaining wall	$h_{wall} = h_{stem} + t_{base} + d_{ds}$ = **1800** mm
Depth of cover in front of wall	d_{cover} = **0** mm
Depth of unplanned excavation	d_{exc} = **0** mm
Height of ground water behind wall	h_{water} = **1000** mm
Height of saturated fill above base	$h_{sat} = \max(h_{water} - t_{base} - d_{ds}, 0 \text{ mm})$ = **800** mm
Density of wall construction	γ_{wall} = **23.6** kN/m^3
Density of base construction	γ_{base} = **23.6** kN/m^3
Angle of rear face of wall	α = **90.0** deg
Angle of soil surface behind wall	β = **0.0** deg
Effective height at virtual back of wall	$h_{eff} = h_{wall} + l_{heel} \times \tan(\beta)$ = **1800** mm

Retained material details

Mobilisation factor	M = **1.5**
Moist density of retained material	γ_m = **18.0** kN/m^3

Saturated density of retained material γ_s = **21.0** kN/m^3

Design shear strength ϕ' = **24.2** deg

Angle of wall friction δ = **0.0** deg

Base material details

Moist density γ_{mb} = **18.0** kN/m^3

Design shear strength ϕ'_b = **24.2** deg

Design base friction δ_b = **18.6** deg

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

Using Coulomb theory

Active pressure coefficient for retained material

$$K_a = \sin(\alpha + \phi')^2 / (\sin(\alpha)^2 \times \sin(\alpha - \delta) \times [1 + \sqrt{(\sin(\phi' + \delta) \times \sin(\phi' - \beta) / (\sin(\alpha - \delta) \times \sin(\alpha + \beta)))}]^2) = \mathbf{0.419}$$

Passive pressure coefficient for base material

$$K_p = \sin(90 - \phi'_b)^2 / (\sin(90 - \delta_b) \times [1 - \sqrt{(\sin(\phi'_b + \delta_b) \times \sin(\phi'_b) / (\sin(90 + \delta_b)))}]^2) = \mathbf{4.187}$$

At-rest pressure

At-rest pressure for retained material $K_0 = 1 – \sin(\phi') =$ **0.590**

Loading details

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

Applied vertical dead load on wall W_{dead} = **0.0** kN/m

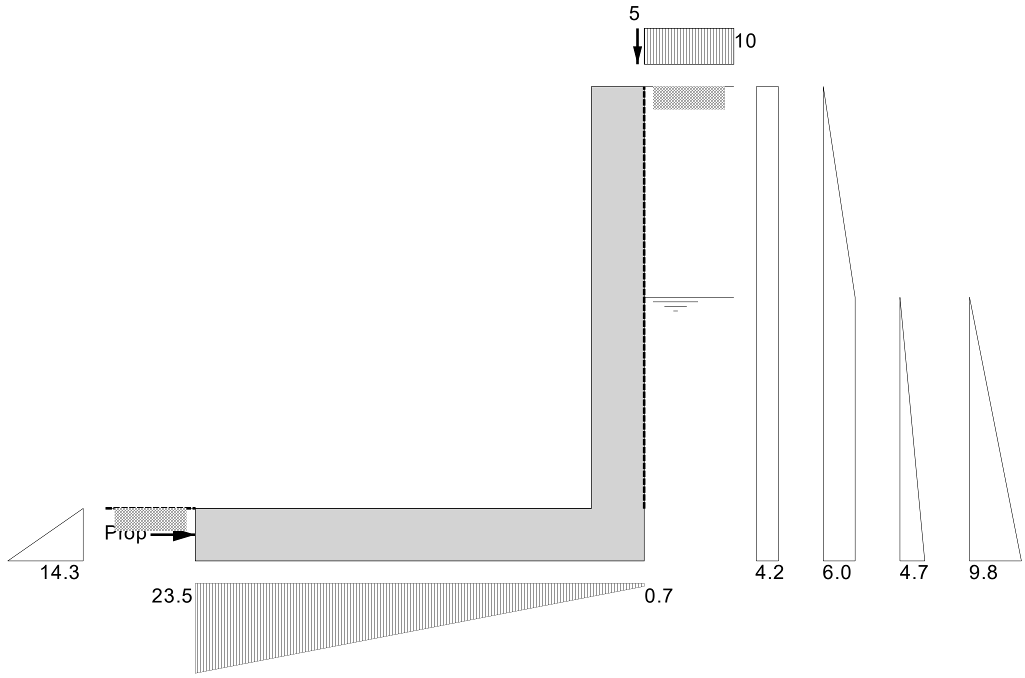
Applied vertical live load on wall W_{live} = **5.0** kN/m

Position of applied vertical load on wall l_{load} = **1675** mm

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

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

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



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

Vertical forces on wall

Wall stem $w_{wall} = h_{stem} \times t_{wall} \times \gamma_{wall}$ = **7.6** kN/m

Wall base	$w_{base} = l_{base} \times t_{base} \times \gamma_{base}$ = **8** kN/m
Applied vertical load	$W_v = W_{dead} + W_{live}$ = **5** kN/m
Total vertical load	$W_{total} = w_{wall} + w_{base} + W_v$ = **20.6** kN/m

Horizontal forces on wall

Surcharge	$F_{sur} = K_a \times \text{Surcharge} \times h_{eff}$ = **7.5** kN/m
Moist backfill above water table	$F_{m\_a} = 0.5 \times K_a \times \gamma_m \times (h_{eff} - h_{water})^2$ = **2.4** kN/m
Moist backfill below water table	$F_{m\_b} = K_a \times \gamma_m \times (h_{eff} - h_{water}) \times h_{water}$ = **6** kN/m
Saturated backfill	$F_s = 0.5 \times K_a \times (\gamma_s - \gamma_{water}) \times h_{water}^2$ = **2.3** kN/m
Water	$F_{water} = 0.5 \times h_{water}^2 \times \gamma_{water}$ = **4.9** kN/m
Total horizontal load	$F_{total} = F_{sur} + F_{m\_a} + F_{m\_b} + F_s + F_{water}$ = **23.2** kN/m

Calculate propping force

Passive resistance of soil in front of wall	$F_p = 0.5 \times K_p \times \cos(\delta_b) \times (d_{cover} + t_{base} + d_{ds} - d_{exc})^2 \times \gamma_{mb}$ = **1.4** kN/m
Propping force	$F_{prop} = \max(F_{total} - F_p - (W_{total} - W_{live}) \times \tan(\delta_b), 0 \text{ kN/m})$
	F_{prop} = **16.5** kN/m

Overturning moments

Surcharge	$M_{sur} = F_{sur} \times (h_{eff} - 2 \times d_{ds}) / 2$ = **6.8** kNm/m
Moist backfill above water table	$M_{m\_a} = F_{m\_a} \times (h_{eff} + 2 \times h_{water} - 3 \times d_{ds}) / 3$ = **3.1** kNm/m
Moist backfill below water table	$M_{m\_b} = F_{m\_b} \times (h_{water} - 2 \times d_{ds}) / 2$ = **3** kNm/m
Saturated backfill	$M_s = F_s \times (h_{water} - 3 \times d_{ds}) / 3$ = **0.8** kNm/m
Water	$M_{water} = F_{water} \times (h_{water} - 3 \times d_{ds}) / 3$ = **1.6** kNm/m
Total overturning moment	$M_{ot} = M_{sur} + M_{m\_a} + M_{m\_b} + M_s + M_{water}$ = **15.3** kNm/m

Restoring moments

Wall stem	$M_{wall} = w_{wall} \times (l_{toe} + t_{wall} / 2)$ = **12.1** kNm/m
Wall base	$M_{base} = w_{base} \times l_{base} / 2$ = **6.8** kNm/m
Total restoring moment	$M_{rest} = M_{wall} + M_{base}$ = **18.9** kNm/m

Check bearing pressure

Design vertical live load	$M_{live} = W_{live} \times l_{load}$ = **8.4** kNm/m
Total moment for bearing	$M_{total} = M_{rest} - M_{ot} + M_{live}$ = **12** kNm/m
Total vertical reaction	$R = W_{total}$ = **20.6** kN/m
Distance to reaction	$x_{bar} = M_{total} / R$ = **584** mm
Eccentricity of reaction	$e = \text{abs}((l_{base} / 2) - x_{bar})$ = **266** mm

Reaction acts within middle third of base

Bearing pressure at toe	$p_{toe} = (R / l_{base}) + (6 \times R \times e / l_{base}^2)$ = **23.5** kN/m^2
Bearing pressure at heel	$p_{heel} = (R / l_{base}) - (6 \times R \times e / l_{base}^2)$ = **0.7** kN/m^2

PASS - Maximum bearing pressure is less than allowable bearing pressure

RETAINING WALL DESIGN (BS 8002:1994)

TEDDS calculation version 1.2.01.08

Ultimate limit state load factors

Dead load factor γ_{f_d} = **1.4**

Live load factor γ_{f_l} = **1.6**

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

Factored vertical forces on wall

Wall stem $w_{wall\_f} = \gamma_{f\_d} \times h_{stem} \times t_{wall} \times \gamma_{wall}$ = **10.6** kN/m

Wall base $w_{base\_f} = \gamma_{f\_d} \times l_{base} \times t_{base} \times \gamma_{base}$ = **11.2** kN/m

Applied vertical load $W_{v\_f} = \gamma_{f\_d} \times W_{dead} + \gamma_{f\_l} \times W_{live}$ = **8** kN/m

Total vertical load $W_{total\_f} = w_{wall\_f} + w_{base\_f} + W_{v\_f}$ = **29.8** kN/m

Factored horizontal active forces on wall

Surcharge $F_{sur\_f} = \gamma_{f\_l} \times K_a \times Surcharge \times h_{eff}$ = **12.1** kN/m

Moist backfill above water table $F_{m\_a\_f} = \gamma_{f\_e} \times 0.5 \times K_a \times \gamma_m \times (h_{eff} - h_{water})^2$ = **3.4** kN/m

Moist backfill below water table $F_{m\_b\_f} = \gamma_{f\_e} \times K_a \times \gamma_m \times (h_{eff} - h_{water}) \times h_{water}$ = **8.4** kN/m

Saturated backfill $F_{s\_f} = \gamma_{f\_e} \times 0.5 \times K_a \times (\gamma_s - \gamma_{water}) \times h_{water}^2$ = **3.3** kN/m

Water $F_{water\_f} = \gamma_{f\_e} \times 0.5 \times h_{water}^2 \times \gamma_{water}$ = **6.9** kN/m

Total horizontal load $F_{total\_f} = F_{sur\_f} + F_{m\_a\_f} + F_{m\_b\_f} + F_{s\_f} + F_{water\_f}$ = **34** kN/m

Calculate propping force

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

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

F_{prop_f} = **24.7** kN/m

Factored overturning moments

Surcharge $M_{sur\_f} = F_{sur\_f} \times (h_{eff} - 2 \times d_{ds}) / 2$ = **10.8** kNm/m

Moist backfill above water table $M_{m\_a\_f} = F_{m\_a\_f} \times (h_{eff} + 2 \times h_{water} - 3 \times d_{ds}) / 3$ = **4.3** kNm/m

Moist backfill below water table $M_{m\_b\_f} = F_{m\_b\_f} \times (h_{water} - 2 \times d_{ds}) / 2$ = **4.2** kNm/m

Saturated backfill $M_{s\_f} = F_{s\_f} \times (h_{water} - 3 \times d_{ds}) / 3$ = **1.1** kNm/m

Water $M_{water\_f} = F_{water\_f} \times (h_{water} - 3 \times d_{ds}) / 3$ = **2.3** kNm/m

Total overturning moment $M_{ot\_f} = M_{sur\_f} + M_{m\_a\_f} + M_{m\_b\_f} + M_{s\_f} + M_{water\_f}$ = **22.7** kNm/m

Restoring moments

Wall stem $M_{wall\_f} = w_{wall\_f} \times (l_{toe} + t_{wall} / 2)$ = **16.9** kNm/m

Wall base $M_{base\_f} = w_{base\_f} \times l_{base} / 2$ = **9.5** kNm/m

Design vertical load $M_{v\_f} = W_{v\_f} \times l_{load}$ = **13.4** kNm/m

Total restoring moment $M_{rest\_f} = M_{wall\_f} + M_{base\_f} + M_{v\_f}$ = **39.9** kNm/m

Factored bearing pressure

Total moment for bearing $M_{total\_f} = M_{rest\_f} - M_{ot\_f}$ = **17.1** kNm/m

Total vertical reaction $R_f = W_{total\_f}$ = **29.8** kN/m

Distance to reaction $x_{bar\_f} = M_{total\_f} / R_f$ = **575** mm

Eccentricity of reaction $e_f = abs((l_{base} / 2) - x_{bar\_f})$ = **275** mm

Reaction acts within middle third of base

Bearing pressure at toe $p_{toe\_f} = (R_f / l_{base}) + (6 \times R_f \times e_f / l_{base}^2)$ = **34.5** kN/m^2

Bearing pressure at heel $p_{heel\_f} = (R_f / l_{base}) - (6 \times R_f \times e_f / l_{base}^2)$ = **0.5** kN/m^2

Rate of change of base reaction $rate = (p_{toe\_f} - p_{heel\_f}) / l_{base}$ = **20.01** kN/m^2/m

Bearing pressure at stem / toe $p_{stem\_toe\_f} = \max(p_{toe\_f} - (rate \times l_{toe}), 0 \text{ kN/m}^2)$ = **4.5** kN/m^2

Bearing pressure at mid stem $p_{stem\_mid\_f}$ = max(p_{toe_f} - (rate × (l_{toe} + t_{wall} / 2)), 0 kN/m^2) = **2.5** kN/m^2

Bearing pressure at stem / heel $p_{stem\_heel\_f}$ = max(p_{toe_f} - (rate × (l_{toe} + t_{wall})), 0 kN/m^2) = **0.5** kN/m^2

Design of reinforced concrete retaining wall toe (BS 8002:1994)

Material properties

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

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

Base details

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

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

Calculate shear for toe design

Shear from bearing pressure V_{toe_bear} = (p_{toe_f} + $p_{stem\_toe\_f}$) × l_{toe} / 2 = **29.3** kN/m

Shear from weight of base $V_{toe\_wt\_base}$ = γ_{f_d} × γ_{base} × l_{toe} × t_{base} = **9.9** kN/m

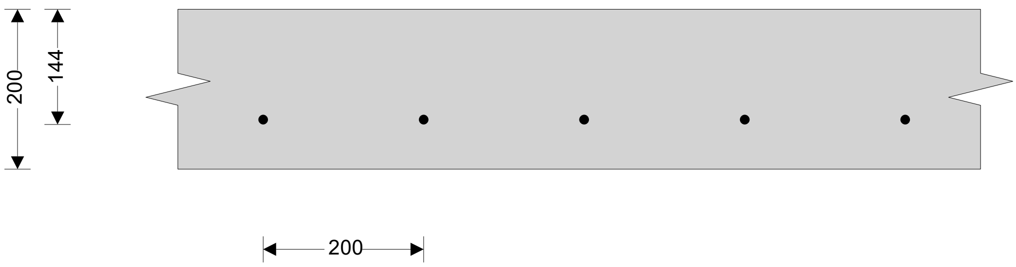
Total shear for toe design V_{toe} = V_{toe_bear} - $V_{toe\_wt\_base}$ = **19.4** kN/m

Calculate moment for toe design

Moment from bearing pressure M_{toe_bear} = (2 × p_{toe_f} + $p_{stem\_mid\_f}$) × (l_{toe} + t_{wall} / 2)^2 / 6 = **30.6** kNm/m

Moment from weight of base $M_{toe\_wt\_base}$ = (γ_{f_d} × γ_{base} × t_{base} × (l_{toe} + t_{wall} / 2)^2 / 2) = **8.5** kNm/m

Total moment for toe design M_{toe} = M_{toe_bear} - $M_{toe\_wt\_base}$ = **22.1** kNm/m



Check toe in bending

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

Depth of reinforcement d_{toe} = t_{base} – c_{toe} – (ϕ_{toe} / 2) = **144.0** mm

Constant K_{toe} = M_{toe} / (b × d_{toe}^2 × f_{cu}) = **0.027**

Compression reinforcement is not required

Lever arm z_{toe} = min(0.5 + √(0.25 - (min(K_{toe}, 0.225) / 0.9)),0.95) × d_{toe}

z_{toe} = **137** mm

Area of tension reinforcement required As_{toe_des} = M_{toe} / (0.87 × f_y × z_{toe}) = **371** mm^2/m

Minimum area of tension reinforcement As_{toe_min} = k × b × t_{base} = **260** mm^2/m

Area of tension reinforcement required As_{toe_req} = Max(As_{toe_des}, As_{toe_min}) = **371** mm^2/m

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

Area of reinforcement provided As_{toe_prov} = **565** mm^2/m

PASS - Reinforcement provided at the retaining wall toe is adequate

Check shear resistance at toe

Design shear stress v_{toe} = V_{toe} / (b × d_{toe}) = **0.135** N/mm^2

Allowable shear stress v_{adm} = min(0.8 × √(f_{cu} / 1 N/mm^2), 5) × 1 N/mm^2 = **5.000** N/mm^2

PASS - Design shear stress is less than maximum shear stress

From BS8110:Part 1:1997 – Table 3.8

Design concrete shear stress v_{c_toe} = **0.699** N/mm^2

v_{toe} < v_{c_toe} - No shear reinforcement required

Design of reinforced concrete retaining wall stem (BS 8002:1994)

Material properties

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

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

Wall details

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

Cover to reinforcement in stem c_{stem} = **50** mm

Cover to reinforcement in wall c_{wall} = **50** mm

Factored horizontal active forces on stem

Surcharge $F_{s\_sur\_f} = \gamma_{f\_l} \times K_a \times Surcharge \times (h_{eff} - t_{base} - d_{ds})$ = **10.7** kN/m

Moist backfill above water table $F_{s\_m\_a\_f} = 0.5 \times \gamma_{f\_e} \times K_a \times \gamma_m \times (h_{eff} - t_{base} - d_{ds} - h_{sat})^2$ = **3.4** kN/m

Moist backfill below water table $F_{s\_m\_b\_f} = \gamma_{f\_e} \times K_a \times \gamma_m \times (h_{eff} - t_{base} - d_{ds} - h_{sat}) \times h_{sat}$ = **6.7** kN/m

Saturated backfill $F_{s\_s\_f} = 0.5 \times \gamma_{f\_e} \times K_a \times (\gamma_s - \gamma_{water}) \times h_{sat}^2$ = **2.1** kN/m

Water $F_{s\_water\_f} = 0.5 \times \gamma_{f\_e} \times \gamma_{water} \times h_{sat}^2$ = **4.4** kN/m

Calculate shear for stem design

Shear at base of stem $V_{stem} = F_{s\_sur\_f} + F_{s\_m\_a\_f} + F_{s\_m\_b\_f} + F_{s\_s\_f} + F_{s\_water\_f} - F_{prop\_f}$ = **2.7** kN/m

Calculate moment for stem design

Surcharge $M_{s\_sur} = F_{s\_sur\_f} \times (h_{stem} + t_{base}) / 2$ = **9.6** kNm/m

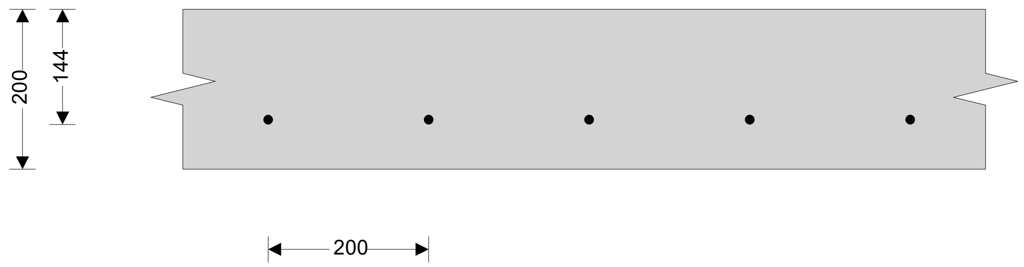
Moist backfill above water table $M_{s\_m\_a} = F_{s\_m\_a\_f} \times (2 \times h_{sat} + h_{eff} - d_{ds} + t_{base} / 2) / 3$ = **3.9** kNm/m

Moist backfill below water table $M_{s\_m\_b} = F_{s\_m\_b\_f} \times h_{sat} / 2$ = **2.7** kNm/m

Saturated backfill $M_{s\_s} = F_{s\_s\_f} \times h_{sat} / 3$ = **0.6** kNm/m

Water $M_{s\_water} = F_{s\_water\_f} \times h_{sat} / 3$ = **1.2** kNm/m

Total moment for stem design $M_{stem} = M_{s\_sur} + M_{s\_m\_a} + M_{s\_m\_b} + M_{s\_s} + M_{s\_water}$ = **18** kNm/m



Check wall stem in bending

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

Depth of reinforcement $d_{stem} = t_{wall} - c_{stem} - (\phi_{stem} / 2)$ = **144.0** mm

Constant $K_{stem} = M_{stem} / (b \times d_{stem}^2 \times f_{cu})$ = **0.022**

Compression reinforcement is not required

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

z_{stem} = **137** mm

Area of tension reinforcement required $A_{s\_stem\_des} = M_{stem} / (0.87 \times f_y \times z_{stem})$ = **303** mm^2/m

Minimum area of tension reinforcement $A_{s\_stem\_min} = k \times b \times t_{wall}$ = **260** mm^2/m

Area of tension reinforcement required $A_{s\_stem\_req} = \max(A_{s\_stem\_des}, A_{s\_stem\_min})$ = **303** mm^2/m

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

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

PASS - Reinforcement provided at the retaining wall stem is adequate

Check shear resistance at wall stem

Design shear stress $v_{stem} = V_{stem} / (b \times d_{stem})$ = **0.018** N/mm²

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

PASS - Design shear stress is less than maximum shear stress

From BS8110:Part 1:1997 – Table 3.8

Design concrete shear stress v_{c_stem} = **0.699** N/mm²

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

Check retaining wall deflection

Basic span/effective depth ratio $ratio_{bas}$ = **7**

Design service stress $f_s = 2 \times f_y \times A_{s\_stem\_req} / (3 \times A_{s\_stem\_prov})$ = **178.4** N/mm²

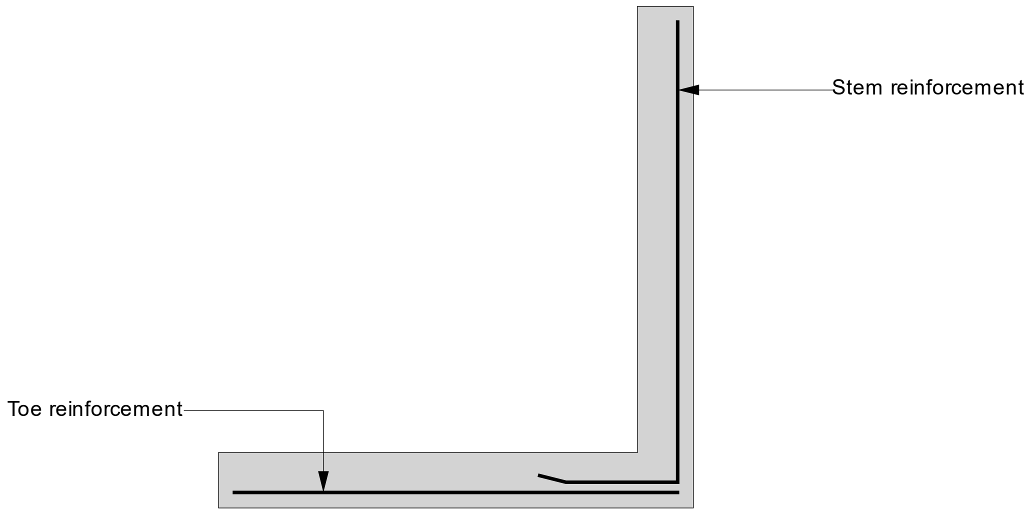
Modification factor $factor_{tens} = \min(0.55 + (477 \text{ N/mm}^2 - f_s)/(120 \times (0.9 \text{ N/mm}^2 + (M_{stem}/(b \times d_{stem}^2)))),2)$ = **1.96**

Maximum span/effective depth ratio $ratio_{max} = ratio_{bas} \times factor_{tens}$ = **13.70**

Actual span/effective depth ratio $ratio_{act} = h_{stem} / d_{stem}$ = **11.11**

PASS - Span to depth ratio is acceptable

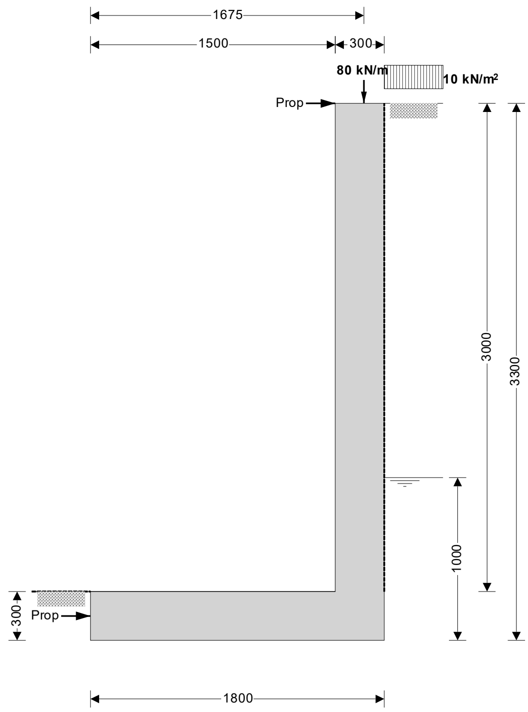
Indicative retaining wall reinforcement diagram



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

RETAINING WALL ANALYSIS (BS 8002:1994)

TEDDS calculation version 1.2.01.08



Wall details

Retaining wall type	**Cantilever propped at both**
Height of retaining wall stem	h_{stem} = **3000** mm
Thickness of wall stem	t_{wall} = **300** mm
Length of toe	l_{toe} = **1500** mm
Length of heel	l_{heel} = **0** mm
Overall length of base	$l_{base} = l_{toe} + l_{heel} + t_{wall}$ = **1800** mm
Thickness of base	t_{base} = **300** mm
Depth of downstand	d_{ds} = **0** mm
Position of downstand	l_{ds} = **200** mm
Thickness of downstand	t_{ds} = **300** mm
Height of retaining wall	$h_{wall} = h_{stem} + t_{base} + d_{ds}$ = **3300** mm
Depth of cover in front of wall	d_{cover} = **0** mm
Depth of unplanned excavation	d_{exc} = **0** mm
Height of ground water behind wall	h_{water} = **1000** mm
Height of saturated fill above base	$h_{sat} = \max(h_{water} - t_{base} - d_{ds}, 0 \text{ mm})$ = **700** mm
Density of wall construction	γ_{wall} = **23.6** kN/m^3
Density of base construction	γ_{base} = **23.6** kN/m^3
Angle of rear face of wall	α = **90.0** deg
Angle of soil surface behind wall	β = **0.0** deg
Effective height at virtual back of wall	$h_{eff} = h_{wall} + l_{heel} \times \tan(\beta)$ = **3300** mm

Retained material details

Mobilisation factor	M = **1.5**
Moist density of retained material	γ_m = **18.0** kN/m^3

Saturated density of retained material γ_s = **21.0** kN/m^3

Design shear strength ϕ' = **24.2** deg

Angle of wall friction δ = **0.0** deg

Base material details

Moist density γ_{mb} = **18.0** kN/m^3

Design shear strength ϕ'_b = **24.2** deg

Design base friction δ_b = **18.6** deg

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

Using Coulomb theory

Active pressure coefficient for retained material

$$K_a = \sin(\alpha + \phi')^2 / (\sin(\alpha)^2 \times \sin(\alpha - \delta) \times [1 + \sqrt{(\sin(\phi' + \delta) \times \sin(\phi' - \beta) / (\sin(\alpha - \delta) \times \sin(\alpha + \beta)))}]^2) = \mathbf{0.419}$$

Passive pressure coefficient for base material

$$K_p = \sin(90 - \phi'_b)^2 / (\sin(90 - \delta_b) \times [1 - \sqrt{(\sin(\phi'_b + \delta_b) \times \sin(\phi'_b) / (\sin(90 + \delta_b)))}]^2) = \mathbf{4.187}$$

At-rest pressure

At-rest pressure for retained material $K_0 = 1 – \sin(\phi')$ = **0.590**

Loading details

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

Applied vertical dead load on wall W_{dead} = **75.0** kN/m

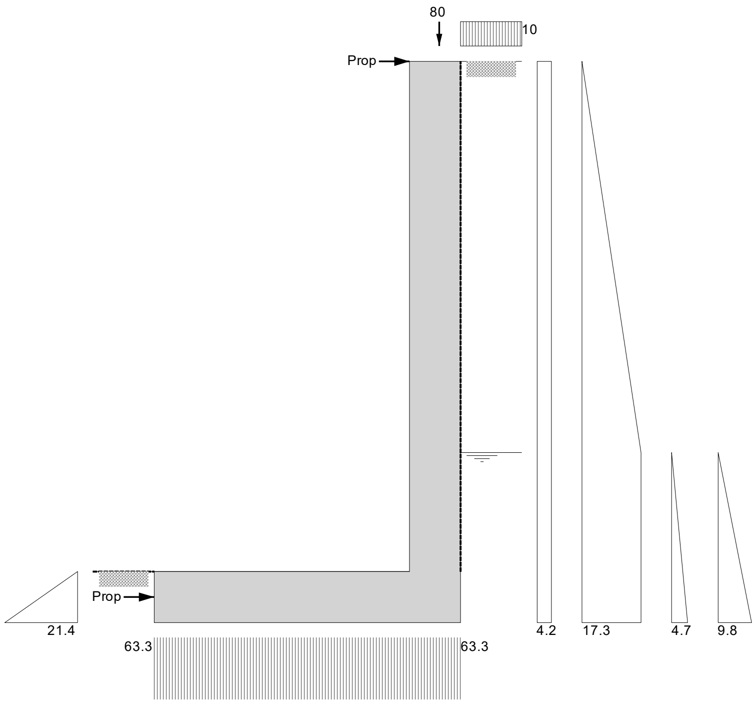
Applied vertical live load on wall W_{live} = **5.0** kN/m

Position of applied vertical load on wall l_{load} = **1675** mm

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

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

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



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

Vertical forces on wall

Wall stem $w_{wall} = h_{stem} \times t_{wall} \times \gamma_{wall}$ = **21.2** kN/m

Wall base $w_{base} = l_{base} \times t_{base} \times \gamma_{base}$ = **12.7** kN/m

Applied vertical load $W_v = W_{dead} + W_{live}$ = **80** kN/m

Total vertical load $W_{total} = w_{wall} + w_{base} + W_v$ = **114** kN/m

Horizontal forces on wall

Surcharge $F_{sur} = K_a \times \text{Surcharge} \times h_{eff}$ = **13.8** kN/m

Moist backfill above water table $F_{m\_a} = 0.5 \times K_a \times \gamma_m \times (h_{eff} - h_{water})^2$ = **19.9** kN/m

Moist backfill below water table $F_{m\_b} = K_a \times \gamma_m \times (h_{eff} - h_{water}) \times h_{water}$ = **17.3** kN/m

Saturated backfill $F_s = 0.5 \times K_a \times (\gamma_s - \gamma_{water}) \times h_{water}^2$ = **2.3** kN/m

Water $F_{water} = 0.5 \times h_{water}^2 \times \gamma_{water}$ = **4.9** kN/m

Total horizontal load $F_{total} = F_{sur} + F_{m\_a} + F_{m\_b} + F_s + F_{water}$ = **58.3** kN/m

Calculate total propping force

Passive resistance of soil in front of wall $F_p = 0.5 \times K_p \times \cos(\delta_b) \times (d_{cover} + t_{base} + d_{ds} - d_{exc})^2 \times \gamma_{mb}$ = **3.2** kN/m

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

F_{prop} = **18.4** kN/m

Overturning moments

Surcharge $M_{sur} = F_{sur} \times (h_{eff} - 2 \times d_{ds}) / 2$ = **22.8** kNm/m

Moist backfill above water table $M_{m\_a} = F_{m\_a} \times (h_{eff} + 2 \times h_{water} - 3 \times d_{ds}) / 3$ = **35.2** kNm/m

Moist backfill below water table $M_{m\_b} = F_{m\_b} \times (h_{water} - 2 \times d_{ds}) / 2$ = **8.7** kNm/m

Saturated backfill $M_s = F_s \times (h_{water} - 3 \times d_{ds}) / 3$ = **0.8** kNm/m

Water $M_{water} = F_{water} \times (h_{water} - 3 \times d_{ds}) / 3$ = **1.6** kNm/m

Total overturning moment $M_{ot} = M_{sur} + M_{m\_a} + M_{m\_b} + M_s + M_{water}$ = **69.1** kNm/m

Restoring moments

Wall stem $M_{wall} = w_{wall} \times (l_{toe} + t_{wall} / 2)$ = **35** kNm/m

Wall base $M_{base} = w_{base} \times l_{base} / 2$ = **11.5** kNm/m

Design vertical load $M_v = W_v \times l_{load}$ = **134** kNm/m

Total restoring moment $M_{rest} = M_{wall} + M_{base} + M_v$ = **180.5** kNm/m

Check bearing pressure

Total vertical reaction $R = W_{total}$ = **114.0** kN/m

Distance to reaction $x_{bar} = l_{base} / 2$ = **900** mm

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

Reaction acts within middle third of base

Bearing pressure at toe $p_{toe} = (R / l_{base}) - (6 \times R \times e / l_{base}^2)$ = **63.3** kN/m^2

Bearing pressure at heel $p_{heel} = (R / l_{base}) + (6 \times R \times e / l_{base}^2)$ = **63.3** kN/m^2

PASS - Maximum bearing pressure is less than allowable bearing pressure

Calculate propping forces to top and base of wall

Propping force to top of wall

$F_{prop\_top} = (M_{ot} - M_{rest} + R \times l_{base} / 2 - F_{prop} \times t_{base} / 2) / (h_{stem} + t_{base} / 2)$ = **-3.690** kN/m

Propping force to base of wall $F_{prop\_base} = F_{prop} - F_{prop\_top}$ = **22.109** kN/m

RETAINING WALL DESIGN (BS 8002:1994)

TEDDS calculation version 1.2.01.08

Ultimate limit state load factors

Dead load factor γ_{f_d} = **1.4**

Live load factor γ_{f_l} = **1.6**

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

Factored vertical forces on wall

Wall stem $w_{wall\_f} = \gamma_{f\_d} \times h_{stem} \times t_{wall} \times \gamma_{wall}$ = **29.7** kN/m

Wall base $w_{base\_f} = \gamma_{f\_d} \times l_{base} \times t_{base} \times \gamma_{base}$ = **17.8** kN/m

Applied vertical load $W_{v\_f} = \gamma_{f\_d} \times W_{dead} + \gamma_{f\_l} \times W_{live}$ = **113** kN/m

Total vertical load $W_{total\_f} = w_{wall\_f} + w_{base\_f} + W_{v\_f}$ = **160.6** kN/m

Factored horizontal active forces on wall

Surcharge $F_{sur\_f} = \gamma_{f\_l} \times K_a \times Surcharge \times h_{eff}$ = **22.1** kN/m

Moist backfill above water table $F_{m\_a\_f} = \gamma_{f\_e} \times 0.5 \times K_a \times \gamma_m \times (h_{eff} - h_{water})^2$ = **27.9** kN/m

Moist backfill below water table $F_{m\_b\_f} = \gamma_{f\_e} \times K_a \times \gamma_m \times (h_{eff} - h_{water}) \times h_{water}$ = **24.3** kN/m

Saturated backfill $F_{s\_f} = \gamma_{f\_e} \times 0.5 \times K_a \times (\gamma_s - \gamma_{water}) \times h_{water}^2$ = **3.3** kN/m

Water $F_{water\_f} = \gamma_{f\_e} \times 0.5 \times h_{water}^2 \times \gamma_{water}$ = **6.9** kN/m

Total horizontal load $F_{total\_f} = F_{sur\_f} + F_{m\_a\_f} + F_{m\_b\_f} + F_{s\_f} + F_{water\_f}$ = **84.4** kN/m

Calculate total propping force

Passive resistance of soil in front of wall $F_{p\_f} = \gamma_{f\_e} \times 0.5 \times K_p \times \cos(\delta_b) \times (d_{cover} + t_{base} + d_{ds} - d_{exc})^2 \times \gamma_{mb}$ = **4.5** kN/m

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

F_{prop_f} = **28.5** kN/m

Factored overturning moments

Surcharge $M_{sur\_f} = F_{sur\_f} \times (h_{eff} - 2 \times d_{ds}) / 2$ = **36.5** kNm/m

Moist backfill above water table $M_{m\_a\_f} = F_{m\_a\_f} \times (h_{eff} + 2 \times h_{water} - 3 \times d_{ds}) / 3$ = **49.3** kNm/m

Moist backfill below water table $M_{m\_b\_f} = F_{m\_b\_f} \times (h_{water} - 2 \times d_{ds}) / 2$ = **12.1** kNm/m

Saturated backfill $M_{s\_f} = F_{s\_f} \times (h_{water} - 3 \times d_{ds}) / 3$ = **1.1** kNm/m

Water $M_{water\_f} = F_{water\_f} \times (h_{water} - 3 \times d_{ds}) / 3$ = **2.3** kNm/m

Total overturning moment $M_{ot\_f} = M_{sur\_f} + M_{m\_a\_f} + M_{m\_b\_f} + M_{s\_f} + M_{water\_f}$ = **101.3** kNm/m

Restoring moments

Wall stem $M_{wall\_f} = w_{wall\_f} \times (l_{toe} + t_{wall} / 2)$ = **49.1** kNm/m

Wall base $M_{base\_f} = w_{base\_f} \times l_{base} / 2$ = **16.1** kNm/m

Design vertical load $M_{v\_f} = W_{v\_f} \times l_{load}$ = **189.3** kNm/m

Total restoring moment $M_{rest\_f} = M_{wall\_f} + M_{base\_f} + M_{v\_f}$ = **254.4** kNm/m

Factored bearing pressure

Total vertical reaction $R_f = W_{total\_f}$ = **160.6** kN/m

Distance to reaction $x_{bar\_f} = l_{base} / 2$ = **900** mm

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

Reaction acts within middle third of base

Bearing pressure at toe $p_{toe\_f} = (R_f / l_{base}) - (6 \times R_f \times e_f / l_{base}^2)$ = **89.2** kN/m^2

Bearing pressure at heel $p_{heel\_f} = (R_f / l_{base}) + (6 \times R_f \times e_f / l_{base}^2)$ = **89.2** kN/m^2

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

Bearing pressure at stem / toe $p_{stem\_toe\_f} = \max(p_{toe\_f} - (rate \times l_{toe}), 0\ kN/m^2)$ = **89.2** kN/m^2

Bearing pressure at mid stem $p_{stem\_mid\_f} = \max(p_{toe\_f} - (rate \times (l_{toe} + t_{wall} / 2)), 0 \text{ kN/m}^2) =$ **89.2** kN/m^2

Bearing pressure at stem / heel $p_{stem\_heel\_f} = \max(p_{toe\_f} - (rate \times (l_{toe} + t_{wall})), 0 \text{ kN/m}^2) =$ **89.2** kN/m^2

Calculate propping forces to top and base of wall

Propping force to top of wall

$F_{prop\_top\_f} = (M_{ot\_f} - M_{rest\_f} + R_f \times l_{base} / 2 - F_{prop\_f} \times t_{base} / 2) / (h_{stem} + t_{base} / 2) =$ **-4.097** kN/m

Propping force to base of wall $F_{prop\_base\_f} = F_{prop\_f} - F_{prop\_top\_f} =$ **32.645** kN/m

Design of reinforced concrete retaining wall toe (BS 8002:1994)

Material properties

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

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

Base details

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

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

Calculate shear for toe design

Shear from bearing pressure $V_{toe\_bear} = (p_{toe\_f} + p_{stem\_toe\_f}) \times l_{toe} / 2 =$ **133.8** kN/m

Shear from weight of base $V_{toe\_wt\_base} = \gamma_{f\_d} \times \gamma_{base} \times l_{toe} \times t_{base} =$ **14.9** kN/m

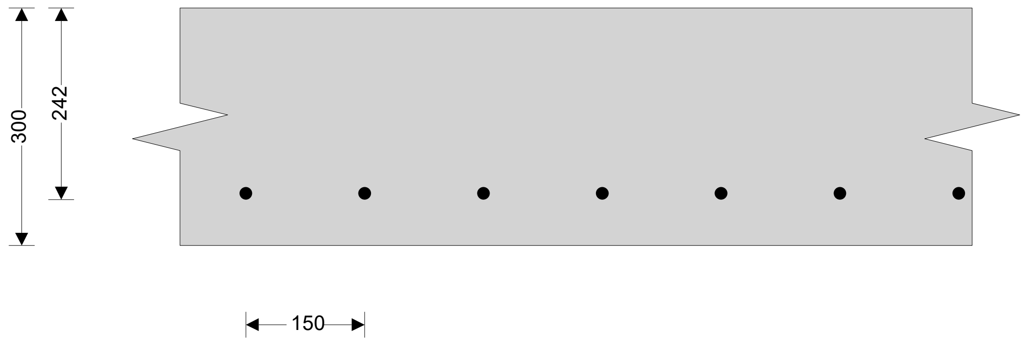
Total shear for toe design $V_{toe} = V_{toe\_bear} - V_{toe\_wt\_base} =$ **118.9** kN/m

Calculate moment for toe design

Moment from bearing pressure $M_{toe\_bear} = (2 \times p_{toe\_f} + p_{stem\_mid\_f}) \times (l_{toe} + t_{wall} / 2)^2 / 6 =$ **121.4** kNm/m

Moment from weight of base $M_{toe\_wt\_base} = (\gamma_{f\_d} \times \gamma_{base} \times t_{base} \times (l_{toe} + t_{wall} / 2)^2 / 2) =$ **13.5** kNm/m

Total moment for toe design $M_{toe} = M_{toe\_bear} - M_{toe\_wt\_base} =$ **107.9** kNm/m



Check toe in bending

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

Depth of reinforcement $d_{toe} = t_{base} - c_{toe} - (\phi_{toe} / 2) =$ **242.0** mm

Constant $K_{toe} = M_{toe} / (b \times d_{toe}^2 \times f_{cu}) =$ **0.046**

Compression reinforcement is not required

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

$z_{toe} =$ **229** mm

Area of tension reinforcement required $A_{s\_toe\_des} = M_{toe} / (0.87 \times f_y \times z_{toe}) =$ **1084** mm^2/m

Minimum area of tension reinforcement $A_{s\_toe\_min} = k \times b \times t_{base} =$ **390** mm^2/m

Area of tension reinforcement required $A_{s\_toe\_req} = \text{Max}(A_{s\_toe\_des}, A_{s\_toe\_min}) =$ **1084** mm^2/m

Reinforcement provided **16 mm dia.bars @ 150 mm centres**

Area of reinforcement provided $A_{s\_toe\_prov} =$ **1340** mm^2/m

PASS - Reinforcement provided at the retaining wall toe is adequate

Check shear resistance at toe

Design shear stress $v_{toe} = V_{toe} / (b \times d_{toe})$ = **0.492** N/mm^2

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

PASS - Design shear stress is less than maximum shear stress

From BS8110:Part 1:1997 – Table 3.8

Design concrete shear stress v_{c_toe} = **0.688** N/mm^2

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

Design of reinforced concrete retaining wall stem (BS 8002:1994)

Material properties

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

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

Wall details

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

Cover to reinforcement in stem c_{stem} = **50** mm

Cover to reinforcement in wall c_{wall} = **50** mm

Factored horizontal active forces on stem

Surcharge $F_{s\_sur\_f} = \gamma_{f\_l} \times K_a \times Surcharge \times (h_{eff} - t_{base} - d_{ds})$ = **20.1** kN/m

Moist backfill above water table $F_{s\_m\_a\_f} = 0.5 \times \gamma_{f\_e} \times K_a \times \gamma_m \times (h_{eff} - t_{base} - d_{ds} - h_{sat})^2$ = **27.9** kN/m

Moist backfill below water table $F_{s\_m\_b\_f} = \gamma_{f\_e} \times K_a \times \gamma_m \times (h_{eff} - t_{base} - d_{ds} - h_{sat}) \times h_{sat}$ = **17** kN/m

Saturated backfill $F_{s\_s\_f} = 0.5 \times \gamma_{f\_e} \times K_a \times (\gamma_s - \gamma_{water}) \times h_{sat}^2$ = **1.6** kN/m

Water $F_{s\_water\_f} = 0.5 \times \gamma_{f\_e} \times \gamma_{water} \times h_{sat}^2$ = **3.4** kN/m

Calculate shear for stem design

Surcharge $V_{s\_sur\_f} = 5 \times F_{s\_sur\_f} / 8$ = **12.6** kN/m

Moist backfill above water table $V_{s\_m\_a\_f} = F_{s\_m\_a\_f} \times b_l \times ((5 \times L^2) - b_l^2) / (5 \times L^3)$ = **18.2** kN/m

Moist backfill below water table $V_{s\_m\_b\_f} = F_{s\_m\_b\_f} \times (8 - (n^2 \times (4 - n))) / 8$ = **16.4** kN/m

Saturated backfill $V_{s\_s\_f} = F_{s\_s\_f} \times (1 - (a_l^2 \times ((5 \times L) - a_l) / (20 \times L^3)))$ = **1.6** kN/m

Water $V_{s\_water\_f} = F_{s\_water\_f} \times (1 - (a_l^2 \times ((5 \times L) - a_l) / (20 \times L^3)))$ = **3.3** kN/m

Total shear for stem design $V_{stem} = V_{s\_sur\_f} + V_{s\_m\_a\_f} + V_{s\_m\_b\_f} + V_{s\_s\_f} + V_{s\_water\_f}$ = **52** kN/m

Calculate moment for stem design

Surcharge $M_{s\_sur} = F_{s\_sur\_f} \times L / 8$ = **7.9** kNm/m

Moist backfill above water table $M_{s\_m\_a} = F_{s\_m\_a\_f} \times b_l \times ((5 \times L^2) - (3 \times b_l^2)) / (15 \times L^2)$ = **14.5** kNm/m

Moist backfill below water table $M_{s\_m\_b} = F_{s\_m\_b\_f} \times a_l \times (2 - n)^2 / 8$ = **5.4** kNm/m

Saturated backfill $M_{s\_s} = F_{s\_s\_f} \times a_l \times ((3 \times a_l^2) - (15 \times a_l \times L) + (20 \times L^2)) / (60 \times L^2)$ = **0.4** kNm/m

Water $M_{s\_water} = F_{s\_water\_f} \times a_l \times ((3 \times a_l^2) - (15 \times a_l \times L) + (20 \times L^2)) / (60 \times L^2)$ = **0.8** kNm/m

Total moment for stem design $M_{stem} = M_{s\_sur} + M_{s\_m\_a} + M_{s\_m\_b} + M_{s\_s} + M_{s\_water}$ = **29** kNm/m

Calculate moment for wall design

Surcharge $M_{w\_sur} = 9 \times F_{s\_sur\_f} \times L / 128$ = **4.4** kNm/m

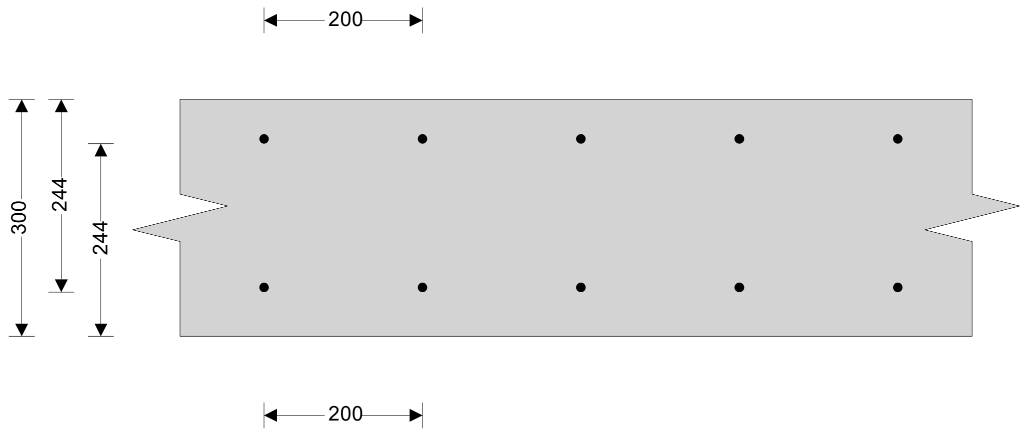
Moist backfill above water table $M_{w\_m\_a} = F_{s\_m\_a\_f} \times 0.577 \times b_l \times [(b_l^3 + 5 \times a_l \times L^2)/(5 \times L^3) - 0.577^2/3]$ = **8.8** kNm/m

Moist backfill below water table $M_{w\_m\_b} = F_{s\_m\_b\_f} \times a_l \times [((8 - n^2 \times (4-n))^2 / 16) - 4 + n \times (4-n)]/8$ = **1.3** kNm/m

Saturated backfill $M_{w\_s} = F_{s\_s\_f} \times [a_l^2 \times x \times ((5 \times L) - a_l)/(20 \times L^3) - (x - b_l)^3 / (3 \times a_l^2)]$ = **0.1** kNm/m

Water $M_{w\_water} = F_{s\_water\_f} \times [a_l^2 \times x \times ((5 \times L) - a_l)/(20 \times L^3) - (x - b_l)^3 / (3 \times a_l^2)]$ = **0.1** kNm/m

Total moment for wall design $M_{wall} = M_{w\_sur} + M_{w\_m\_a} + M_{w\_m\_b} + M_{w\_s} + M_{w\_water}$ = **14.8** kNm/m



Check wall stem in bending

Width of wall stem — $b = \mathbf{1000}$ mm/m

Depth of reinforcement — $d_{stem} = t_{wall} - c_{stem} - (\phi_{stem} / 2) = \mathbf{244.0}$ mm

Constant — $K_{stem} = M_{stem} / (b \times d_{stem}^2 \times f_{cu}) = \mathbf{0.012}$

Compression reinforcement is not required

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

$z_{stem} = \mathbf{232}$ mm

Area of tension reinforcement required — $As_{stem\_des} = M_{stem} / (0.87 \times f_y \times z_{stem}) = \mathbf{288}$ mm^2/m

Minimum area of tension reinforcement — $As_{stem\_min} = k \times b \times t_{wall} = \mathbf{390}$ mm^2/m

Area of tension reinforcement required — $As_{stem\_req} = Max(As_{stem\_des}, As_{stem\_min}) = \mathbf{390}$ mm^2/m

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

Area of reinforcement provided — $As_{stem\_prov} = \mathbf{565}$ mm^2/m

PASS - Reinforcement provided at the retaining wall stem is adequate

Check shear resistance at wall stem

Design shear stress — $v_{stem} = V_{stem} / (b \times d_{stem}) = \mathbf{0.213}$ N/mm^2

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

PASS - Design shear stress is less than maximum shear stress

From BS8110:Part 1:1997 – Table 3.8

Design concrete shear stress — $v_{c\_stem} = \mathbf{0.514}$ N/mm^2

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

Check mid height of wall in bending

Depth of reinforcement — $d_{wall} = t_{wall} - c_{wall} - (\phi_{wall} / 2) = \mathbf{244.0}$ mm

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

Compression reinforcement is not required

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

$z_{wall} = \mathbf{232}$ mm

Area of tension reinforcement required — $As_{wall\_des} = M_{wall} / (0.87 \times f_y \times z_{wall}) = \mathbf{146}$ mm^2/m

Minimum area of tension reinforcement — $As_{wall\_min} = k \times b \times t_{wall} = \mathbf{390}$ mm^2/m

Area of tension reinforcement required — $As_{wall\_req} = Max(As_{wall\_des}, As_{wall\_min}) = \mathbf{390}$ mm^2/m

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

Area of reinforcement provided — $As_{wall\_prov} = \mathbf{565}$ mm^2/m

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

Check retaining wall deflection

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

Design service stress $f_s = 2 \times f_y \times A_{s\_stem\_req} / (3 \times A_{s\_stem\_prov})$ = **229.9** N/mm^2

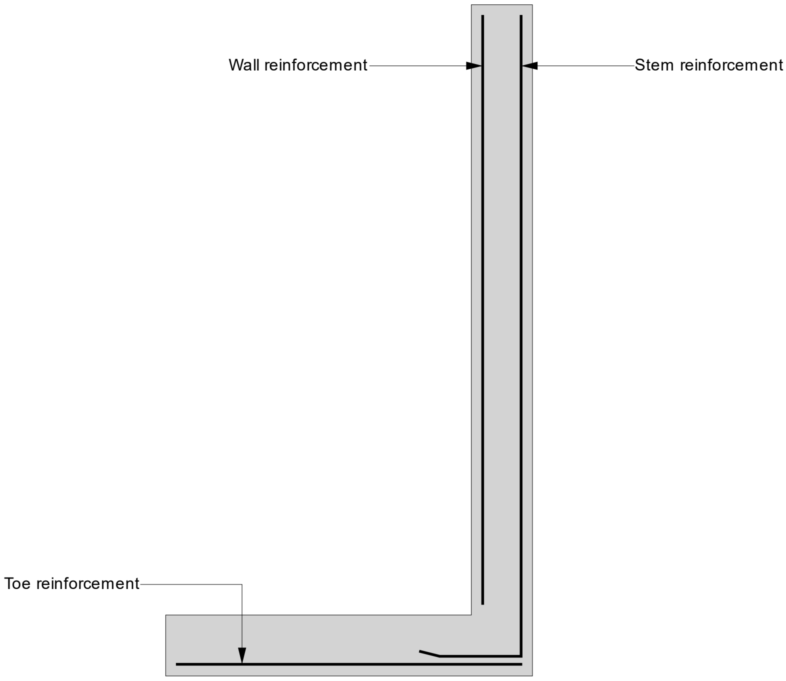
Modification factor $factor_{tens} = \min(0.55 + (477\text{ N/mm}^2 - f_s)/(120 \times (0.9\text{ N/mm}^2 + (M_{stem}/(b \times d_{stem}^2)))),2)$ = **2.00**

Maximum span/effective depth ratio $ratio_{max} = ratio_{bas} \times factor_{tens}$ = **40.00**

Actual span/effective depth ratio $ratio_{act} = h_{stem} / d_{stem}$ = **12.30**

PASS - Span to depth ratio is acceptable

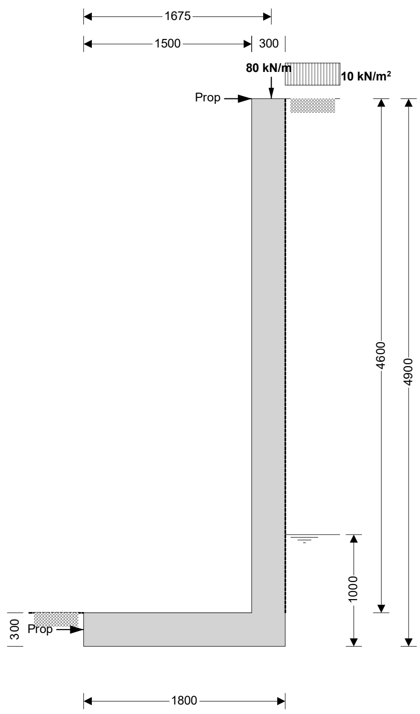
Indicative retaining wall reinforcement diagram



Toe bars - 16 mm dia.@ 150 mm centres - (1340 mm^2/m)
Wall bars - 12 mm dia.@ 200 mm centres - (565 mm^2/m)
Stem bars - 12 mm dia.@ 200 mm centres - (565 mm^2/m)

RETAINING WALL ANALYSIS (BS 8002:1994)

TEDDS calculation version 1.2.01.08



Wall details

Retaining wall type	**Cantilever propped at both**
Height of retaining wall stem	h_{stem} = **4600** mm
Thickness of wall stem	t_{wall} = **300** mm
Length of toe	l_{toe} = **1500** mm
Length of heel	l_{heel} = **0** mm
Overall length of base	$l_{base} = l_{toe} + l_{heel} + t_{wall}$ = **1800** mm
Thickness of base	t_{base} = **300** mm
Depth of downstand	d_{ds} = **0** mm
Position of downstand	l_{ds} = **200** mm
Thickness of downstand	t_{ds} = **300** mm
Height of retaining wall	$h_{wall} = h_{stem} + t_{base} + d_{ds}$ = **4900** mm
Depth of cover in front of wall	d_{cover} = **0** mm
Depth of unplanned excavation	d_{exc} = **0** mm
Height of ground water behind wall	h_{water} = **1000** mm
Height of saturated fill above base	$h_{sat} = \max(h_{water} - t_{base} - d_{ds}, 0\text{ mm})$ = **700** mm
Density of wall construction	γ_{wall} = **23.6** kN/m^3
Density of base construction	γ_{base} = **23.6** kN/m^3
Angle of rear face of wall	α = **90.0** deg
Angle of soil surface behind wall	β = **0.0** deg
Effective height at virtual back of wall	$h_{eff} = h_{wall} + l_{heel} \times \tan(\beta)$ = **4900** mm

Retained material details

Mobilisation factor	M = **1.5**
Moist density of retained material	γ_m = **18.0** kN/m^3

Saturated density of retained material γ_s = **21.0** kN/m^3

Design shear strength ϕ' = **24.2** deg

Angle of wall friction δ = **0.0** deg

Base material details

Moist density γ_{mb} = **18.0** kN/m^3

Design shear strength ϕ'_b = **24.2** deg

Design base friction δ_b = **18.6** deg

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

Using Coulomb theory

Active pressure coefficient for retained material

$$K_a = \sin(\alpha + \phi')^2 / (\sin(\alpha)^2 \times \sin(\alpha - \delta) \times [1 + \sqrt{(\sin(\phi' + \delta) \times \sin(\phi' - \beta) / (\sin(\alpha - \delta) \times \sin(\alpha + \beta)))}]^2) = \mathbf{0.419}$$

Passive pressure coefficient for base material

$$K_p = \sin(90 - \phi'_b)^2 / (\sin(90 - \delta_b) \times [1 - \sqrt{(\sin(\phi'_b + \delta_b) \times \sin(\phi'_b) / (\sin(90 + \delta_b)))}]^2) = \mathbf{4.187}$$

At-rest pressure

At-rest pressure for retained material $K_0 = 1 - \sin(\phi') = \mathbf{0.590}$

Loading details

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

Applied vertical dead load on wall W_{dead} = **75.0** kN/m

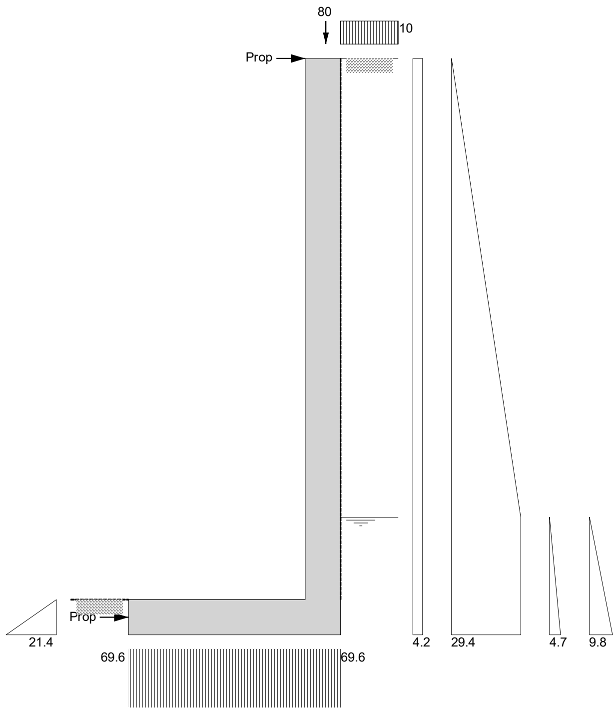
Applied vertical live load on wall W_{live} = **5.0** kN/m

Position of applied vertical load on wall l_{load} = **1675** mm

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

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

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



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

Vertical forces on wall

Wall stem $w_{wall} = h_{stem} \times t_{wall} \times \gamma_{wall}$ = **32.6** kN/m

Wall base $w_{base} = l_{base} \times t_{base} \times \gamma_{base}$ = **12.7** kN/m

Applied vertical load $W_v = W_{dead} + W_{live}$ = **80** kN/m

Total vertical load $W_{total} = w_{wall} + w_{base} + W_v$ = **125.3** kN/m

Horizontal forces on wall

Surcharge $F_{sur} = K_a \times \text{Surcharge} \times h_{eff}$ = **20.5** kN/m

Moist backfill above water table $F_{m\_a} = 0.5 \times K_a \times \gamma_m \times (h_{eff} - h_{water})^2$ = **57.3** kN/m

Moist backfill below water table $F_{m\_b} = K_a \times \gamma_m \times (h_{eff} - h_{water}) \times h_{water}$ = **29.4** kN/m

Saturated backfill $F_s = 0.5 \times K_a \times (\gamma_s - \gamma_{water}) \times h_{water}^2$ = **2.3** kN/m

Water $F_{water} = 0.5 \times h_{water}^2 \times \gamma_{water}$ = **4.9** kN/m

Total horizontal load $F_{total} = F_{sur} + F_{m\_a} + F_{m\_b} + F_s + F_{water}$ = **114.4** kN/m

Calculate total propping force

Passive resistance of soil in front of wall $F_p = 0.5 \times K_p \times \cos(\delta_b) \times (d_{cover} + t_{base} + d_{ds} - d_{exc})^2 \times \gamma_{mb}$ = **3.2** kN/m

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

F_{prop} = **70.7** kN/m

Overturning moments

Surcharge $M_{sur} = F_{sur} \times (h_{eff} - 2 \times d_{ds}) / 2$ = **50.2** kNm/m

Moist backfill above water table $M_{m\_a} = F_{m\_a} \times (h_{eff} + 2 \times h_{water} - 3 \times d_{ds}) / 3$ = **131.8** kNm/m

Moist backfill below water table $M_{m\_b} = F_{m\_b} \times (h_{water} - 2 \times d_{ds}) / 2$ = **14.7** kNm/m

Saturated backfill $M_s = F_s \times (h_{water} - 3 \times d_{ds}) / 3$ = **0.8** kNm/m

Water $M_{water} = F_{water} \times (h_{water} - 3 \times d_{ds}) / 3$ = **1.6** kNm/m

Total overturning moment $M_{ot} = M_{sur} + M_{m\_a} + M_{m\_b} + M_s + M_{water}$ = **199.1** kNm/m

Restoring moments

Wall stem $M_{wall} = w_{wall} \times (l_{toe} + t_{wall} / 2)$ = **53.7** kNm/m

Wall base $M_{base} = w_{base} \times l_{base} / 2$ = **11.5** kNm/m

Design vertical load $M_v = W_v \times l_{load}$ = **134** kNm/m

Total restoring moment $M_{rest} = M_{wall} + M_{base} + M_v$ = **199.2** kNm/m

Check bearing pressure

Total vertical reaction $R = W_{total}$ = **125.3** kN/m

Distance to reaction $x_{bar} = l_{base} / 2$ = **900** mm

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

Reaction acts within middle third of base

Bearing pressure at toe $p_{toe} = (R / l_{base}) - (6 \times R \times e / l_{base}^2)$ = **69.6** kN/m^2

Bearing pressure at heel $p_{heel} = (R / l_{base}) + (6 \times R \times e / l_{base}^2)$ = **69.6** kN/m^2

PASS - Maximum bearing pressure is less than allowable bearing pressure

Calculate propping forces to top and base of wall

Propping force to top of wall

$F_{prop\_top} = (M_{ot} - M_{rest} + R \times l_{base} / 2 - F_{prop} \times t_{base} / 2) / (h_{stem} + t_{base} / 2)$ = **21.491** kN/m

Propping force to base of wall $F_{prop\_base} = F_{prop} - F_{prop\_top}$ = **49.230** kN/m

RETAINING WALL DESIGN (BS 8002:1994)

TEDDS calculation version 1.2.01.08

Ultimate limit state load factors

Dead load factor γ_{f_d} = **1.4**

Live load factor γ_{f_l} = **1.6**

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

Factored vertical forces on wall

Wall stem $w_{wall\_f} = \gamma_{f\_d} \times h_{stem} \times t_{wall} \times \gamma_{wall}$ = **45.6** kN/m

Wall base $w_{base\_f} = \gamma_{f\_d} \times l_{base} \times t_{base} \times \gamma_{base}$ = **17.8** kN/m

Applied vertical load $W_{v\_f} = \gamma_{f\_d} \times W_{dead} + \gamma_{f\_l} \times W_{live}$ = **113** kN/m

Total vertical load $W_{total\_f} = w_{wall\_f} + w_{base\_f} + W_{v\_f}$ = **176.4** kN/m

Factored horizontal active forces on wall

Surcharge $F_{sur\_f} = \gamma_{f\_l} \times K_a \times \text{Surcharge} \times h_{eff}$ = **32.8** kN/m

Moist backfill above water table $F_{m\_a\_f} = \gamma_{f\_e} \times 0.5 \times K_a \times \gamma_m \times (h_{eff} - h_{water})^2$ = **80.2** kN/m

Moist backfill below water table $F_{m\_b\_f} = \gamma_{f\_e} \times K_a \times \gamma_m \times (h_{eff} - h_{water}) \times h_{water}$ = **41.1** kN/m

Saturated backfill $F_{s\_f} = \gamma_{f\_e} \times 0.5 \times K_a \times (\gamma_s - \gamma_{water}) \times h_{water}^2$ = **3.3** kN/m

Water $F_{water\_f} = \gamma_{f\_e} \times 0.5 \times h_{water}^2 \times \gamma_{water}$ = **6.9** kN/m

Total horizontal load $F_{total\_f} = F_{sur\_f} + F_{m\_a\_f} + F_{m\_b\_f} + F_{s\_f} + F_{water\_f}$ = **164.3** kN/m

Calculate total propping force

Passive resistance of soil in front of wall $F_{p\_f} = \gamma_{f\_e} \times 0.5 \times K_p \times \cos(\delta_b) \times (d_{cover} + t_{base} + d_{ds} - d_{exc})^2 \times \gamma_{mb}$ = **4.5** kN/m

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

F_{prop_f} = **103.1** kN/m

Factored overturning moments

Surcharge $M_{sur\_f} = F_{sur\_f} \times (h_{eff} - 2 \times d_{ds}) / 2$ = **80.4** kNm/m

Moist backfill above water table $M_{m\_a\_f} = F_{m\_a\_f} \times (h_{eff} + 2 \times h_{water} - 3 \times d_{ds}) / 3$ = **184.5** kNm/m

Moist backfill below water table $M_{m\_b\_f} = F_{m\_b\_f} \times (h_{water} - 2 \times d_{ds}) / 2$ = **20.6** kNm/m

Saturated backfill $M_{s\_f} = F_{s\_f} \times (h_{water} - 3 \times d_{ds}) / 3$ = **1.1** kNm/m

Water $M_{water\_f} = F_{water\_f} \times (h_{water} - 3 \times d_{ds}) / 3$ = **2.3** kNm/m

Total overturning moment $M_{ot\_f} = M_{sur\_f} + M_{m\_a\_f} + M_{m\_b\_f} + M_{s\_f} + M_{water\_f}$ = **288.8** kNm/m

Restoring moments

Wall stem $M_{wall\_f} = w_{wall\_f} \times (l_{toe} + t_{wall} / 2)$ = **75.2** kNm/m

Wall base $M_{base\_f} = w_{base\_f} \times l_{base} / 2$ = **16.1** kNm/m

Design vertical load $M_{v\_f} = W_{v\_f} \times l_{load}$ = **189.3** kNm/m

Total restoring moment $M_{rest\_f} = M_{wall\_f} + M_{base\_f} + M_{v\_f}$ = **280.6** kNm/m

Factored bearing pressure

Total vertical reaction $R_f = W_{total\_f}$ = **176.4** kN/m

Distance to reaction $x_{bar\_f} = l_{base} / 2$ = **900** mm

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

Reaction acts within middle third of base

Bearing pressure at toe $p_{toe\_f} = (R_f / l_{base}) - (6 \times R_f \times e_f / l_{base}^2)$ = **98** kN/m^2

Bearing pressure at heel $p_{heel\_f} = (R_f / l_{base}) + (6 \times R_f \times e_f / l_{base}^2)$ = **98** kN/m^2

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

Bearing pressure at stem / toe $p_{stem\_toe\_f} = \max(p_{toe\_f} - (\text{rate} \times l_{toe}), 0 \text{ kN/m}^2)$ = **98** kN/m^2

Bearing pressure at mid stem $p_{stem\_mid\_f} = \max(p_{toe\_f} - (rate \times (l_{toe} + t_{wall} / 2)), 0 \text{ kN/m}^2) =$ **98** kN/m²

Bearing pressure at stem / heel $p_{stem\_heel\_f} = \max(p_{toe\_f} - (rate \times (l_{toe} + t_{wall})), 0 \text{ kN/m}^2) =$ **98** kN/m²

Calculate propping forces to top and base of wall

Propping force to top of wall

$F_{prop\_top\_f} = (M_{ot\_f} - M_{rest\_f} + R_f \times l_{base} / 2 - F_{prop\_f} \times t_{base} / 2) / (h_{stem} + t_{base} / 2) =$ **31.910** kN/m

Propping force to base of wall $F_{prop\_base\_f} = F_{prop\_f} - F_{prop\_top\_f} =$ **71.201** kN/m

Design of reinforced concrete retaining wall toe (BS 8002:1994)

Material properties

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

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

Base details

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

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

Calculate shear for toe design

Shear from bearing pressure $V_{toe\_bear} = (p_{toe\_f} + p_{stem\_toe\_f}) \times l_{toe} / 2 =$ **147** kN/m

Shear from weight of base $V_{toe\_wt\_base} = \gamma_{f\_d} \times \gamma_{base} \times l_{toe} \times t_{base} =$ **14.9** kN/m

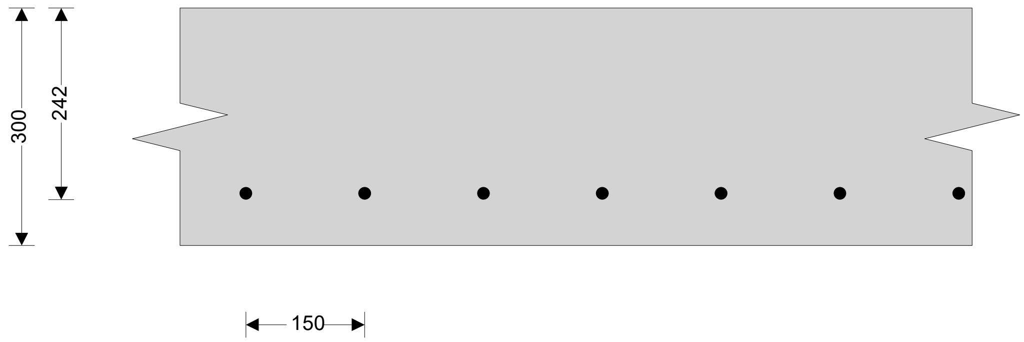
Total shear for toe design $V_{toe} = V_{toe\_bear} - V_{toe\_wt\_base} =$ **132.2** kN/m

Calculate moment for toe design

Moment from bearing pressure $M_{toe\_bear} = (2 \times p_{toe\_f} + p_{stem\_mid\_f}) \times (l_{toe} + t_{wall} / 2)^2 / 6 =$ **133.4** kNm/m

Moment from weight of base $M_{toe\_wt\_base} = (\gamma_{f\_d} \times \gamma_{base} \times t_{base} \times (l_{toe} + t_{wall} / 2)^2 / 2) =$ **13.5** kNm/m

Total moment for toe design $M_{toe} = M_{toe\_bear} - M_{toe\_wt\_base} =$ **119.9** kNm/m



Check toe in bending

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

Depth of reinforcement $d_{toe} = t_{base} - c_{toe} - (\phi_{toe} / 2) =$ **242.0** mm

Constant $K_{toe} = M_{toe} / (b \times d_{toe}^2 \times f_{cu}) =$ **0.051**

Compression reinforcement is not required

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

$z_{toe} =$ **227** mm

Area of tension reinforcement required $As_{toe\_des} = M_{toe} / (0.87 \times f_y \times z_{toe}) =$ **1213** mm²/m

Minimum area of tension reinforcement $As_{toe\_min} = k \times b \times t_{base} =$ **390** mm²/m

Area of tension reinforcement required $As_{toe\_req} = \max(As_{toe\_des}, As_{toe\_min}) =$ **1213** mm²/m

Reinforcement provided **16 mm dia.bars @ 150 mm centres**

Area of reinforcement provided $As_{toe\_prov} =$ **1340** mm²/m

PASS - Reinforcement provided at the retaining wall toe is adequate

Check shear resistance at toe

Design shear stress $v_{toe} = V_{toe} / (b \times d_{toe})$ = **0.546** N/mm^2

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

PASS - Design shear stress is less than maximum shear stress

From BS8110:Part 1:1997 – Table 3.8

Design concrete shear stress v_{c_toe} = **0.688** N/mm^2

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

<u>Design of reinforced concrete retaining wall stem (BS 8002:1994)</u>

Material properties

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

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

Wall details

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

Cover to reinforcement in stem c_{stem} = **50** mm

Cover to reinforcement in wall c_{wall} = **50** mm

Factored horizontal active forces on stem

Surcharge $F_{s\_sur\_f} = \gamma_{f\_l} \times K_a \times \text{Surcharge} \times (h_{eff} - t_{base} - d_{ds})$ = **30.8** kN/m

Moist backfill above water table $F_{s\_m\_a\_f} = 0.5 \times \gamma_{f\_e} \times K_a \times \gamma_m \times (h_{eff} - t_{base} - d_{ds} - h_{sat})^2$ = **80.2** kN/m

Moist backfill below water table $F_{s\_m\_b\_f} = \gamma_{f\_e} \times K_a \times \gamma_m \times (h_{eff} - t_{base} - d_{ds} - h_{sat}) \times h_{sat}$ = **28.8** kN/m

Saturated backfill $F_{s\_s\_f} = 0.5 \times \gamma_{f\_e} \times K_a \times (\gamma_s - \gamma_{water}) \times h_{sat}^2$ = **1.6** kN/m

Water $F_{s\_water\_f} = 0.5 \times \gamma_{f\_e} \times \gamma_{water} \times h_{sat}^2$ = **3.4** kN/m

Calculate shear for stem design

Surcharge $V_{s\_sur\_f} = 5 \times F_{s\_sur\_f} / 8$ = **19.3** kN/m

Moist backfill above water table $V_{s\_m\_a\_f} = F_{s\_m\_a\_f} \times b_l \times ((5 \times L^2) - b_l^2) / (5 \times L^3)$ = **57** kN/m

Moist backfill below water table $V_{s\_m\_b\_f} = F_{s\_m\_b\_f} \times (8 - (n^2 \times (4 - n))) / 8$ = **28.4** kN/m

Saturated backfill $V_{s\_s\_f} = F_{s\_s\_f} \times (1 - (a_l^2 \times ((5 \times L) - a_l) / (20 \times L^3)))$ = **1.6** kN/m

Water $V_{s\_water\_f} = F_{s\_water\_f} \times (1 - (a_l^2 \times ((5 \times L) - a_l) / (20 \times L^3)))$ = **3.3** kN/m

Total shear for stem design $V_{stem} = V_{s\_sur\_f} + V_{s\_m\_a\_f} + V_{s\_m\_b\_f} + V_{s\_s\_f} + V_{s\_water\_f}$ = **109.5** kN/m

Calculate moment for stem design

Surcharge $M_{s\_sur} = F_{s\_sur\_f} \times L / 8$ = **18.3** kNm/m

Moist backfill above water table $M_{s\_m\_a} = F_{s\_m\_a\_f} \times b_l \times ((5 \times L^2) - (3 \times b_l^2)) / (15 \times L^2)$ = **62.1** kNm/m

Moist backfill below water table $M_{s\_m\_b} = F_{s\_m\_b\_f} \times a_l \times (2 - n)^2 / 8$ = **10.1** kNm/m

Saturated backfill $M_{s\_s} = F_{s\_s\_f} \times a_l \times ((3 \times a_l^2) - (15 \times a_l \times L) + (20 \times L^2)) / (60 \times L^2)$ = **0.4** kNm/m

Water $M_{s\_water} = F_{s\_water\_f} \times a_l \times ((3 \times a_l^2) - (15 \times a_l \times L) + (20 \times L^2)) / (60 \times L^2)$ = **0.8** kNm/m

Total moment for stem design $M_{stem} = M_{s\_sur} + M_{s\_m\_a} + M_{s\_m\_b} + M_{s\_s} + M_{s\_water}$ = **91.8** kNm/m

Calculate moment for wall design

Surcharge $M_{w\_sur} = 9 \times F_{s\_sur\_f} \times L / 128$ = **10.3** kNm/m

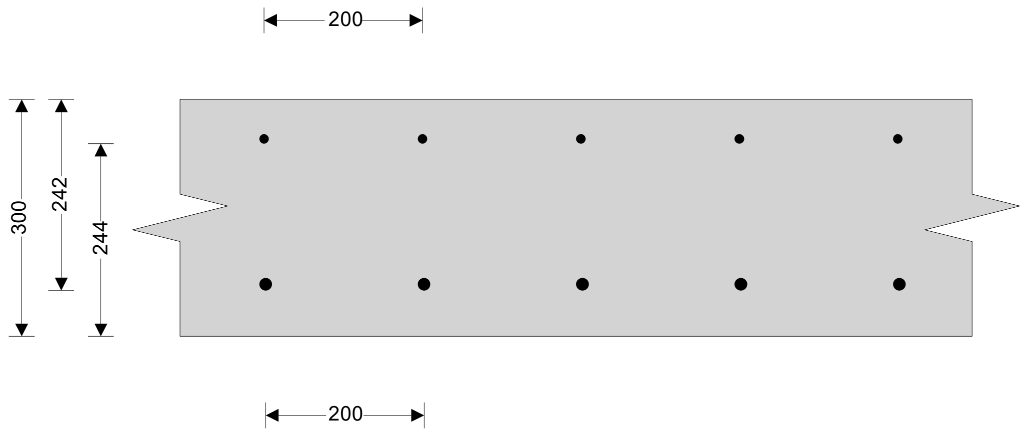
Moist backfill above water table $M_{w\_m\_a} = F_{s\_m\_a\_f} \times 0.577 \times b_l \times [(b_l^3 + 5 \times a_l \times L^2)/(5 \times L^3) - 0.577^2/3]$ = **32.2**
kNm/m

Moist backfill below water table $M_{w\_m\_b} = F_{s\_m\_b\_f} \times a_l \times [((8 - n^2 \times (4-n))^2 / 16) - 4 + n \times (4-n)] / 8$ = **1.7** kNm/m

Saturated backfill $M_{w\_s} = F_{s\_s\_f} \times [a_l^2 \times x \times ((5 \times L) - a_l) / (20 \times L^3) - (x - b_l)^3 / (3 \times a_l^2)]$ = **0** kNm/m

Water $M_{w\_water} = F_{s\_water\_f} \times [a_l^2 \times x \times ((5 \times L) - a_l) / (20 \times L^3) - (x - b_l)^3 / (3 \times a_l^2)]$ = **0.1**
kNm/m

Total moment for wall design $M_{wall} = M_{w\_sur} + M_{w\_m\_a} + M_{w\_m\_b} + M_{w\_s} + M_{w\_water}$ = **44.4** kNm/m



Check wall stem in bending

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

Depth of reinforcement — $d_{stem} = t_{wall} - c_{stem} - (\phi_{stem} / 2) =$ **242.0** mm

Constant — $K_{stem} = M_{stem} / (b \times d_{stem}^2 \times f_{cu}) =$ **0.039**

Compression reinforcement is not required

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

$z_{stem} =$ **230** mm

Area of tension reinforcement required — $A_{s\_stem\_des} = M_{stem} / (0.87 \times f_y \times z_{stem}) =$ **917** mm²/m

Minimum area of tension reinforcement — $A_{s\_stem\_min} = k \times b \times t_{wall} =$ **390** mm²/m

Area of tension reinforcement required — $A_{s\_stem\_req} = \text{Max}(A_{s\_stem\_des}, A_{s\_stem\_min}) =$ **917** mm²/m

Reinforcement provided — **16 mm dia.bars @ 200 mm centres**

Area of reinforcement provided — $A_{s\_stem\_prov} =$ **1005** mm²/m

PASS - Reinforcement provided at the retaining wall stem is adequate

Check shear resistance at wall stem

Design shear stress — $v_{stem} = V_{stem} / (b \times d_{stem}) =$ **0.453** N/mm²

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

PASS - Design shear stress is less than maximum shear stress

From BS8110:Part 1:1997 – Table 3.8

Design concrete shear stress — $v_{c\_stem} =$ **0.625** N/mm²

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

Check mid height of wall in bending

Depth of reinforcement — $d_{wall} = t_{wall} - c_{wall} - (\phi_{wall} / 2) =$ **244.0** mm

Constant — $K_{wall} = M_{wall} / (b \times d_{wall}^2 \times f_{cu}) =$ **0.019**

Compression reinforcement is not required

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

$z_{wall} =$ **232** mm

Area of tension reinforcement required — $A_{s\_wall\_des} = M_{wall} / (0.87 \times f_y \times z_{wall}) =$ **440** mm²/m

Minimum area of tension reinforcement — $A_{s\_wall\_min} = k \times b \times t_{wall} =$ **390** mm²/m

Area of tension reinforcement required — $A_{s\_wall\_req} = \text{Max}(A_{s\_wall\_des}, A_{s\_wall\_min}) =$ **440** mm²/m

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

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

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

Check retaining wall deflection

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

Design service stress $f_s = 2 \times f_y \times A_{s\_stem\_req} / (3 \times A_{s\_stem\_prov})$ = **304.2** N/mm^2

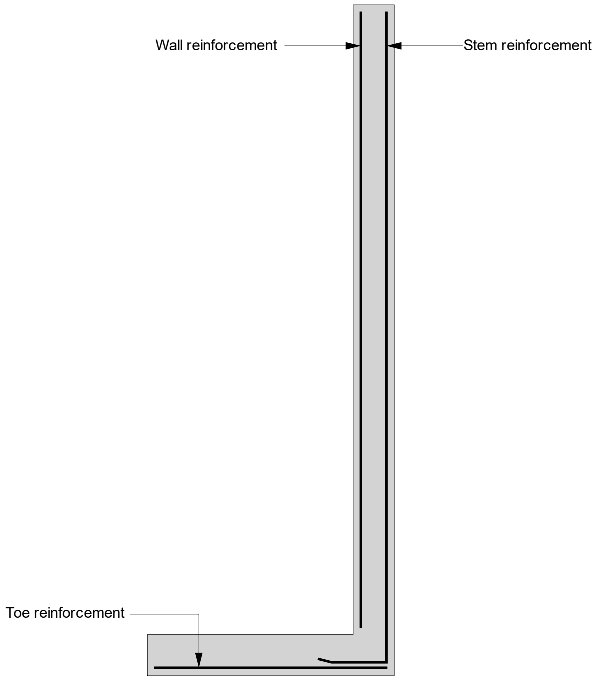
Modification factor $factor_{tens} = \min(0.55 + (477\ N/mm^2 - f_s)/(120 \times (0.9\ N/mm^2 + (M_{stem}/(b \times d_{stem}^2)))),2)$ = **1.13**

Maximum span/effective depth ratio $ratio_{max} = ratio_{bas} \times factor_{tens}$ = **22.67**

Actual span/effective depth ratio $ratio_{act} = h_{stem} / d_{stem}$ = **19.01**

PASS - Span to depth ratio is acceptable

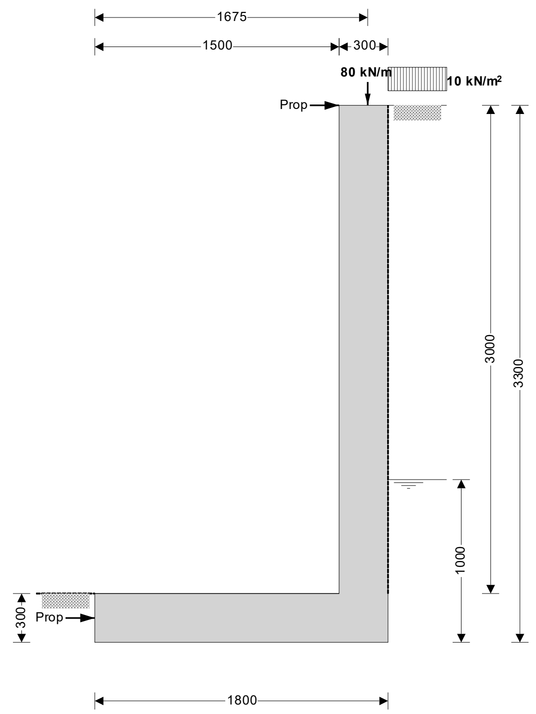
Indicative retaining wall reinforcement diagram



Toe bars - 16 mm dia.@ 150 mm centres - (1340 mm^2/m)
Wall bars - 12 mm dia.@ 200 mm centres - (565 mm^2/m)
Stem bars - 16 mm dia.@ 200 mm centres - (1005 mm^2/m)

RETAINING WALL ANALYSIS (BS 8002:1994)

TEDDS calculation version 1.2.01.08



Wall details

Retaining wall type	**Cantilever propped at both**
Height of retaining wall stem	h_{stem} = **3000** mm
Thickness of wall stem	t_{wall} = **300** mm
Length of toe	l_{toe} = **1500** mm
Length of heel	l_{heel} = **0** mm
Overall length of base	$l_{base} = l_{toe} + l_{heel} + t_{wall}$ = **1800** mm
Thickness of base	t_{base} = **300** mm
Depth of downstand	d_{ds} = **0** mm
Position of downstand	l_{ds} = **200** mm
Thickness of downstand	t_{ds} = **300** mm
Height of retaining wall	$h_{wall} = h_{stem} + t_{base} + d_{ds}$ = **3300** mm
Depth of cover in front of wall	d_{cover} = **0** mm
Depth of unplanned excavation	d_{exc} = **0** mm
Height of ground water behind wall	h_{water} = **1000** mm
Height of saturated fill above base	$h_{sat} = \max(h_{water} - t_{base} - d_{ds}, 0 \text{ mm})$ = **700** mm
Density of wall construction	γ_{wall} = **23.6** kN/m^3
Density of base construction	γ_{base} = **23.6** kN/m^3
Angle of rear face of wall	α = **90.0** deg
Angle of soil surface behind wall	β = **0.0** deg
Effective height at virtual back of wall	$h_{eff} = h_{wall} + l_{heel} \times \tan(\beta)$ = **3300** mm

Retained material details

Mobilisation factor	M = **1.5**
Moist density of retained material	γ_m = **18.0** kN/m^3

Saturated density of retained material γ_s = **21.0** kN/m^3

Design shear strength ϕ' = **24.2** deg

Angle of wall friction δ = **0.0** deg

Base material details

Moist density γ_{mb} = **18.0** kN/m^3

Design shear strength ϕ'_b = **24.2** deg

Design base friction δ_b = **18.6** deg

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

Using Coulomb theory

Active pressure coefficient for retained material

$$K_a = \sin(\alpha + \phi')^2 / (\sin(\alpha)^2 \times \sin(\alpha - \delta) \times [1 + \sqrt{(\sin(\phi' + \delta) \times \sin(\phi' - \beta) / (\sin(\alpha - \delta) \times \sin(\alpha + \beta)))}]^2) = \mathbf{0.419}$$

Passive pressure coefficient for base material

$$K_p = \sin(90 - \phi'_b)^2 / (\sin(90 - \delta_b) \times [1 - \sqrt{(\sin(\phi'_b + \delta_b) \times \sin(\phi'_b) / (\sin(90 + \delta_b)))}]^2) = \mathbf{4.187}$$

At-rest pressure

At-rest pressure for retained material $K_0 = 1 – \sin(\phi') = \mathbf{0.590}$

Loading details

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

Applied vertical dead load on wall W_{dead} = **75.0** kN/m

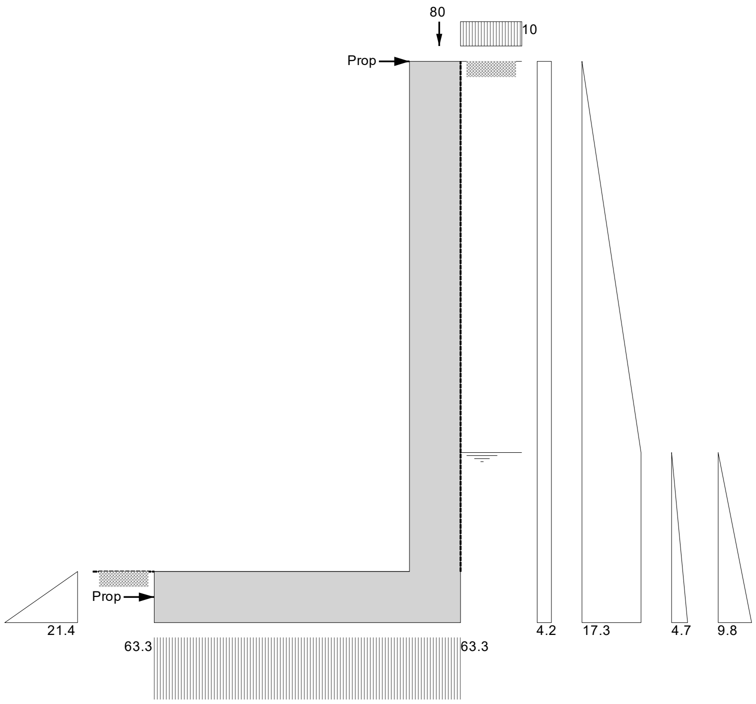
Applied vertical live load on wall W_{live} = **5.0** kN/m

Position of applied vertical load on wall l_{load} = **1675** mm

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

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

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



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

Vertical forces on wall

Wall stem $w_{wall} = h_{stem} \times t_{wall} \times \gamma_{wall}$ = **21.2** kN/m

Wall base $w_{base} = l_{base} \times t_{base} \times \gamma_{base}$ = **12.7** kN/m

Applied vertical load $W_v = W_{dead} + W_{live}$ = **80** kN/m

Total vertical load $W_{total} = w_{wall} + w_{base} + W_v$ = **114** kN/m

Horizontal forces on wall

Surcharge $F_{sur} = K_a \times \text{Surcharge} \times h_{eff}$ = **13.8** kN/m

Moist backfill above water table $F_{m\_a} = 0.5 \times K_a \times \gamma_m \times (h_{eff} - h_{water})^2$ = **19.9** kN/m

Moist backfill below water table $F_{m\_b} = K_a \times \gamma_m \times (h_{eff} - h_{water}) \times h_{water}$ = **17.3** kN/m

Saturated backfill $F_s = 0.5 \times K_a \times (\gamma_s - \gamma_{water}) \times h_{water}^2$ = **2.3** kN/m

Water $F_{water} = 0.5 \times h_{water}^2 \times \gamma_{water}$ = **4.9** kN/m

Total horizontal load $F_{total} = F_{sur} + F_{m\_a} + F_{m\_b} + F_s + F_{water}$ = **58.3** kN/m

Calculate total propping force

Passive resistance of soil in front of wall $F_p = 0.5 \times K_p \times \cos(\delta_b) \times (d_{cover} + t_{base} + d_{ds} - d_{exc})^2 \times \gamma_{mb}$ = **3.2** kN/m

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

F_{prop} = **18.4** kN/m

Overturning moments

Surcharge $M_{sur} = F_{sur} \times (h_{eff} - 2 \times d_{ds}) / 2$ = **22.8** kNm/m

Moist backfill above water table $M_{m\_a} = F_{m\_a} \times (h_{eff} + 2 \times h_{water} - 3 \times d_{ds}) / 3$ = **35.2** kNm/m

Moist backfill below water table $M_{m\_b} = F_{m\_b} \times (h_{water} - 2 \times d_{ds}) / 2$ = **8.7** kNm/m

Saturated backfill $M_s = F_s \times (h_{water} - 3 \times d_{ds}) / 3$ = **0.8** kNm/m

Water $M_{water} = F_{water} \times (h_{water} - 3 \times d_{ds}) / 3$ = **1.6** kNm/m

Total overturning moment $M_{ot} = M_{sur} + M_{m\_a} + M_{m\_b} + M_s + M_{water}$ = **69.1** kNm/m

Restoring moments

Wall stem $M_{wall} = w_{wall} \times (l_{toe} + t_{wall} / 2)$ = **35** kNm/m

Wall base $M_{base} = w_{base} \times l_{base} / 2$ = **11.5** kNm/m

Design vertical load $M_v = W_v \times l_{load}$ = **134** kNm/m

Total restoring moment $M_{rest} = M_{wall} + M_{base} + M_v$ = **180.5** kNm/m

Check bearing pressure

Total vertical reaction $R = W_{total}$ = **114.0** kN/m

Distance to reaction $x_{bar} = l_{base} / 2$ = **900** mm

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

Reaction acts within middle third of base

Bearing pressure at toe $p_{toe} = (R / l_{base}) - (6 \times R \times e / l_{base}^2)$ = **63.3** kN/m^2

Bearing pressure at heel $p_{heel} = (R / l_{base}) + (6 \times R \times e / l_{base}^2)$ = **63.3** kN/m^2

PASS - Maximum bearing pressure is less than allowable bearing pressure

Calculate propping forces to top and base of wall

Propping force to top of wall

$F_{prop\_top} = (M_{ot} - M_{rest} + R \times l_{base} / 2 - F_{prop} \times t_{base} / 2) / (h_{stem} + t_{base} / 2)$ = **-3.690** kN/m

Propping force to base of wall $F_{prop\_base} = F_{prop} - F_{prop\_top}$ = **22.109** kN/m

RETAINING WALL DESIGN (BS 8002:1994)

TEDDS calculation version 1.2.01.08

Ultimate limit state load factors

Dead load factor γ_{f_d} = **1.4**

Live load factor γ_{f_l} = **1.6**

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

Factored vertical forces on wall

Wall stem $w_{wall\_f} = \gamma_{f\_d} \times h_{stem} \times t_{wall} \times \gamma_{wall}$ = **29.7** kN/m

Wall base $w_{base\_f} = \gamma_{f\_d} \times l_{base} \times t_{base} \times \gamma_{base}$ = **17.8** kN/m

Applied vertical load $W_{v\_f} = \gamma_{f\_d} \times W_{dead} + \gamma_{f\_l} \times W_{live}$ = **113** kN/m

Total vertical load $W_{total\_f} = w_{wall\_f} + w_{base\_f} + W_{v\_f}$ = **160.6** kN/m

Factored horizontal active forces on wall

Surcharge $F_{sur\_f} = \gamma_{f\_l} \times K_a \times Surcharge \times h_{eff}$ = **22.1** kN/m

Moist backfill above water table $F_{m\_a\_f} = \gamma_{f\_e} \times 0.5 \times K_a \times \gamma_m \times (h_{eff} - h_{water})^2$ = **27.9** kN/m

Moist backfill below water table $F_{m\_b\_f} = \gamma_{f\_e} \times K_a \times \gamma_m \times (h_{eff} - h_{water}) \times h_{water}$ = **24.3** kN/m

Saturated backfill $F_{s\_f} = \gamma_{f\_e} \times 0.5 \times K_a \times (\gamma_s - \gamma_{water}) \times h_{water}^2$ = **3.3** kN/m

Water $F_{water\_f} = \gamma_{f\_e} \times 0.5 \times h_{water}^2 \times \gamma_{water}$ = **6.9** kN/m

Total horizontal load $F_{total\_f} = F_{sur\_f} + F_{m\_a\_f} + F_{m\_b\_f} + F_{s\_f} + F_{water\_f}$ = **84.4** kN/m

Calculate total propping force

Passive resistance of soil in front of wall $F_{p\_f} = \gamma_{f\_e} \times 0.5 \times K_p \times \cos(\delta_b) \times (d_{cover} + t_{base} + d_{ds} - d_{exc})^2 \times \gamma_{mb}$ = **4.5** kN/m

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

F_{prop_f} = **28.5** kN/m

Factored overturning moments

Surcharge $M_{sur\_f} = F_{sur\_f} \times (h_{eff} - 2 \times d_{ds}) / 2$ = **36.5** kNm/m

Moist backfill above water table $M_{m\_a\_f} = F_{m\_a\_f} \times (h_{eff} + 2 \times h_{water} - 3 \times d_{ds}) / 3$ = **49.3** kNm/m

Moist backfill below water table $M_{m\_b\_f} = F_{m\_b\_f} \times (h_{water} - 2 \times d_{ds}) / 2$ = **12.1** kNm/m

Saturated backfill $M_{s\_f} = F_{s\_f} \times (h_{water} - 3 \times d_{ds}) / 3$ = **1.1** kNm/m

Water $M_{water\_f} = F_{water\_f} \times (h_{water} - 3 \times d_{ds}) / 3$ = **2.3** kNm/m

Total overturning moment $M_{ot\_f} = M_{sur\_f} + M_{m\_a\_f} + M_{m\_b\_f} + M_{s\_f} + M_{water\_f}$ = **101.3** kNm/m

Restoring moments

Wall stem $M_{wall\_f} = w_{wall\_f} \times (l_{toe} + t_{wall} / 2)$ = **49.1** kNm/m

Wall base $M_{base\_f} = w_{base\_f} \times l_{base} / 2$ = **16.1** kNm/m

Design vertical load $M_{v\_f} = W_{v\_f} \times l_{load}$ = **189.3** kNm/m

Total restoring moment $M_{rest\_f} = M_{wall\_f} + M_{base\_f} + M_{v\_f}$ = **254.4** kNm/m

Factored bearing pressure

Total vertical reaction $R_f = W_{total\_f}$ = **160.6** kN/m

Distance to reaction $x_{bar\_f} = l_{base} / 2$ = **900** mm

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

Reaction acts within middle third of base

Bearing pressure at toe $p_{toe\_f} = (R_f / l_{base}) - (6 \times R_f \times e_f / l_{base}^2)$ = **89.2** kN/m^2

Bearing pressure at heel $p_{heel\_f} = (R_f / l_{base}) + (6 \times R_f \times e_f / l_{base}^2)$ = **89.2** kN/m^2

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

Bearing pressure at stem / toe $p_{stem\_toe\_f} = \max(p_{toe\_f} - (rate \times l_{toe}), 0 \text{ kN/m}^2)$ = **89.2** kN/m^2

Bearing pressure at mid stem $p_{stem\_mid\_f} = \max(p_{toe\_f} - (rate \times (l_{toe} + t_{wall} / 2)), 0 \text{ kN/m}^2) = \mathbf{89.2}$ kN/m^2

Bearing pressure at stem / heel $p_{stem\_heel\_f} = \max(p_{toe\_f} - (rate \times (l_{toe} + t_{wall})), 0 \text{ kN/m}^2) = \mathbf{89.2}$ kN/m^2

Calculate propping forces to top and base of wall

Propping force to top of wall

$F_{prop\_top\_f} = (M_{ot\_f} - M_{rest\_f} + R_f \times l_{base} / 2 - F_{prop\_f} \times t_{base} / 2) / (h_{stem} + t_{base} / 2) = \mathbf{-4.097}$ kN/m

Propping force to base of wall $F_{prop\_base\_f} = F_{prop\_f} - F_{prop\_top\_f} = \mathbf{32.645}$ kN/m

Design of reinforced concrete retaining wall toe (BS 8002:1994)

Material properties

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

Characteristic strength of reinforcement $f_y = \mathbf{500}$ N/mm^2

Base details

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

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

Calculate shear for toe design

Shear from bearing pressure $V_{toe\_bear} = (p_{toe\_f} + p_{stem\_toe\_f}) \times l_{toe} / 2 = \mathbf{133.8}$ kN/m

Shear from weight of base $V_{toe\_wt\_base} = \gamma_{f\_d} \times \gamma_{base} \times l_{toe} \times t_{base} = \mathbf{14.9}$ kN/m

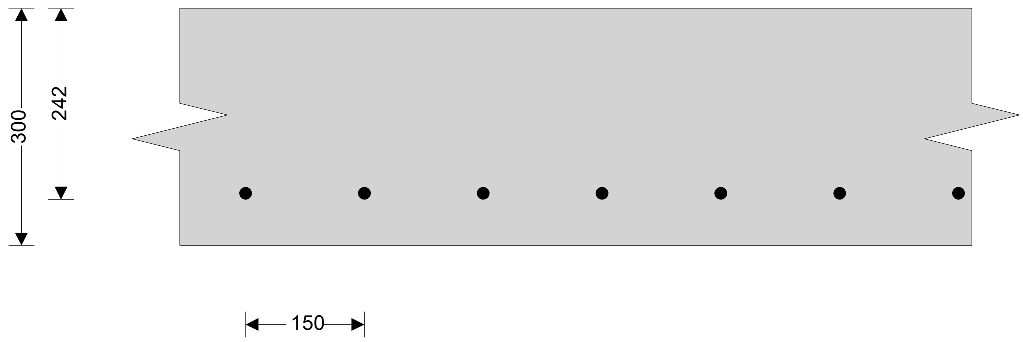
Total shear for toe design $V_{toe} = V_{toe\_bear} - V_{toe\_wt\_base} = \mathbf{118.9}$ kN/m

Calculate moment for toe design

Moment from bearing pressure $M_{toe\_bear} = (2 \times p_{toe\_f} + p_{stem\_mid\_f}) \times (l_{toe} + t_{wall} / 2)^2 / 6 = \mathbf{121.4}$ kNm/m

Moment from weight of base $M_{toe\_wt\_base} = (\gamma_{f\_d} \times \gamma_{base} \times t_{base} \times (l_{toe} + t_{wall} / 2)^2 / 2) = \mathbf{13.5}$ kNm/m

Total moment for toe design $M_{toe} = M_{toe\_bear} - M_{toe\_wt\_base} = \mathbf{107.9}$ kNm/m



Check toe in bending

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

Depth of reinforcement $d_{toe} = t_{base} - c_{toe} - (\phi_{toe} / 2) = \mathbf{242.0}$ mm

Constant $K_{toe} = M_{toe} / (b \times d_{toe}^2 \times f_{cu}) = \mathbf{0.046}$

Compression reinforcement is not required

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

$z_{toe} = \mathbf{229}$ mm

Area of tension reinforcement required $As_{toe\_des} = M_{toe} / (0.87 \times f_y \times z_{toe}) = \mathbf{1084}$ mm^2/m

Minimum area of tension reinforcement $As_{toe\_min} = k \times b \times t_{base} = \mathbf{390}$ mm^2/m

Area of tension reinforcement required $As_{toe\_req} = \mathrm{Max}(As_{toe\_des}, As_{toe\_min}) = \mathbf{1084}$ mm^2/m

Reinforcement provided **16 mm dia.bars @ 150 mm centres**

Area of reinforcement provided $As_{toe\_prov} = \mathbf{1340}$ mm^2/m

PASS - Reinforcement provided at the retaining wall toe is adequate

Check shear resistance at toe

Design shear stress $v_{toe} = V_{toe} / (b \times d_{toe}) = \mathbf{0.492}$ N/mm^2

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

PASS - Design shear stress is less than maximum shear stress

From BS8110:Part 1:1997 – Table 3.8

Design concrete shear stress $v_{c\_toe} = \mathbf{0.688}$ N/mm^2

v_{toe} < v_{c_toe} - No shear reinforcement required

Design of reinforced concrete retaining wall stem (BS 8002:1994)

Material properties

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

Characteristic strength of reinforcement $f_y = \mathbf{500}$ N/mm^2

Wall details

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

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

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

Factored horizontal active forces on stem

Surcharge $F_{s\_sur\_f} = \gamma_{f\_l} \times K_a \times \text{Surcharge} \times (h_{eff} - t_{base} - d_{ds}) = \mathbf{20.1}$ kN/m

Moist backfill above water table $F_{s\_m\_a\_f} = 0.5 \times \gamma_{f\_e} \times K_a \times \gamma_m \times (h_{eff} - t_{base} - d_{ds} - h_{sat})^2 = \mathbf{27.9}$ kN/m

Moist backfill below water table $F_{s\_m\_b\_f} = \gamma_{f\_e} \times K_a \times \gamma_m \times (h_{eff} - t_{base} - d_{ds} - h_{sat}) \times h_{sat} = \mathbf{17}$ kN/m

Saturated backfill $F_{s\_s\_f} = 0.5 \times \gamma_{f\_e} \times K_a \times (\gamma_s - \gamma_{water}) \times h_{sat}^2 = \mathbf{1.6}$ kN/m

Water $F_{s\_water\_f} = 0.5 \times \gamma_{f\_e} \times \gamma_{water} \times h_{sat}^2 = \mathbf{3.4}$ kN/m

Calculate shear for stem design

Surcharge $V_{s\_sur\_f} = 5 \times F_{s\_sur\_f} / 8 = \mathbf{12.6}$ kN/m

Moist backfill above water table $V_{s\_m\_a\_f} = F_{s\_m\_a\_f} \times b_l \times ((5 \times L^2) - b_l^2) / (5 \times L^3) = \mathbf{18.2}$ kN/m

Moist backfill below water table $V_{s\_m\_b\_f} = F_{s\_m\_b\_f} \times (8 - (n^2 \times (4 - n))) / 8 = \mathbf{16.4}$ kN/m

Saturated backfill $V_{s\_s\_f} = F_{s\_s\_f} \times (1 - (a_l^2 \times ((5 \times L) - a_l) / (20 \times L^3))) = \mathbf{1.6}$ kN/m

Water $V_{s\_water\_f} = F_{s\_water\_f} \times (1 - (a_l^2 \times ((5 \times L) - a_l) / (20 \times L^3))) = \mathbf{3.3}$ kN/m

Total shear for stem design $V_{stem} = V_{s\_sur\_f} + V_{s\_m\_a\_f} + V_{s\_m\_b\_f} + V_{s\_s\_f} + V_{s\_water\_f} = \mathbf{52}$ kN/m

Calculate moment for stem design

Surcharge $M_{s\_sur} = F_{s\_sur\_f} \times L / 8 = \mathbf{7.9}$ kNm/m

Moist backfill above water table $M_{s\_m\_a} = F_{s\_m\_a\_f} \times b_l \times ((5 \times L^2) - (3 \times b_l^2)) / (15 \times L^2) = \mathbf{14.5}$ kNm/m

Moist backfill below water table $M_{s\_m\_b} = F_{s\_m\_b\_f} \times a_l \times (2 - n)^2 / 8 = \mathbf{5.4}$ kNm/m

Saturated backfill $M_{s\_s} = F_{s\_s\_f} \times a_l \times ((3 \times a_l^2) - (15 \times a_l \times L) + (20 \times L^2)) / (60 \times L^2) = \mathbf{0.4}$ kNm/m

Water $M_{s\_water} = F_{s\_water\_f} \times a_l \times ((3 \times a_l^2) - (15 \times a_l \times L) + (20 \times L^2)) / (60 \times L^2) = \mathbf{0.8}$ kNm/m

Total moment for stem design $M_{stem} = M_{s\_sur} + M_{s\_m\_a} + M_{s\_m\_b} + M_{s\_s} + M_{s\_water} = \mathbf{29}$ kNm/m

Calculate moment for wall design

Surcharge $M_{w\_sur} = 9 \times F_{s\_sur\_f} \times L / 128 = \mathbf{4.4}$ kNm/m

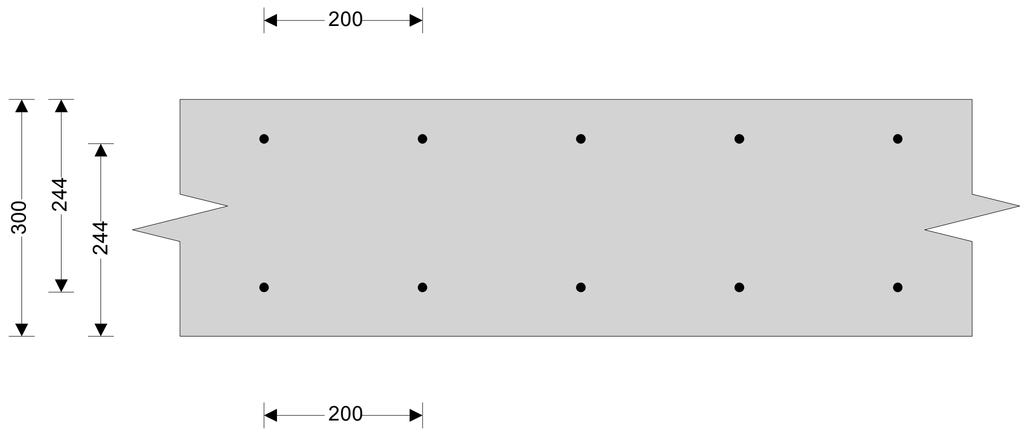
Moist backfill above water table $M_{w\_m\_a} = F_{s\_m\_a\_f} \times 0.577 \times b_l \times [(b_l^3 + 5 \times a_l \times L^2)/(5 \times L^3) - 0.577^2/3] = \mathbf{8.8}$
kNm/m

Moist backfill below water table $M_{w\_m\_b} = F_{s\_m\_b\_f} \times a_l \times [((8 - n^2 \times (4-n))^2 / 16) - 4 + n \times (4-n)]/8 = \mathbf{1.3}$ kNm/m

Saturated backfill $M_{w\_s} = F_{s\_s\_f} \times [a_l^2 \times x \times ((5 \times L) - a_l)/(20 \times L^3) - (x - b_l)^3 / (3 \times a_l^2)] = \mathbf{0.1}$ kNm/m

Water $M_{w\_water} = F_{s\_water\_f} \times [a_l^2 \times x \times ((5 \times L) - a_l)/(20 \times L^3) - (x - b_l)^3 / (3 \times a_l^2)] = \mathbf{0.1}$
kNm/m

Total moment for wall design $M_{wall} = M_{w\_sur} + M_{w\_m\_a} + M_{w\_m\_b} + M_{w\_s} + M_{w\_water} = \mathbf{14.8}$ kNm/m



Check wall stem in bending

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

Depth of reinforcement: $d_{stem} = t_{wall} – c_{stem} – (\phi_{stem} / 2)$ = **244.0** mm

Constant: $K_{stem} = M_{stem} / (b \times d_{stem}^2 \times f_{cu})$ = **0.012**

Compression reinforcement is not required

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

z_{stem} = **232** mm

Area of tension reinforcement required: $A_{s\_stem\_des} = M_{stem} / (0.87 \times f_y \times z_{stem})$ = **288** mm²/m

Minimum area of tension reinforcement: $A_{s\_stem\_min} = k \times b \times t_{wall}$ = **390** mm²/m

Area of tension reinforcement required: $A_{s\_stem\_req} = \mathrm{Max}(A_{s\_stem\_des}, A_{s\_stem\_min})$ = **390** mm²/m

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

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

PASS - Reinforcement provided at the retaining wall stem is adequate

Check shear resistance at wall stem

Design shear stress: $v_{stem} = V_{stem} / (b \times d_{stem})$ = **0.213** N/mm²

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

PASS - Design shear stress is less than maximum shear stress

From BS8110:Part 1:1997 – Table 3.8

Design concrete shear stress: v_{c_stem} = **0.514** N/mm²

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

Check mid height of wall in bending

Depth of reinforcement: $d_{wall} = t_{wall} – c_{wall} – (\phi_{wall} / 2)$ = **244.0** mm

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

Compression reinforcement is not required

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

z_{wall} = **232** mm

Area of tension reinforcement required: $A_{s\_wall\_des} = M_{wall} / (0.87 \times f_y \times z_{wall})$ = **146** mm²/m

Minimum area of tension reinforcement: $A_{s\_wall\_min} = k \times b \times t_{wall}$ = **390** mm²/m

Area of tension reinforcement required: $A_{s\_wall\_req} = \mathrm{Max}(A_{s\_wall\_des}, A_{s\_wall\_min})$ = **390** mm²/m

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

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

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

Check retaining wall deflection

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

Design service stress $f_s = 2 \times f_y \times A_{s\_stem\_req} / (3 \times A_{s\_stem\_prov})$ = **229.9** N/mm²

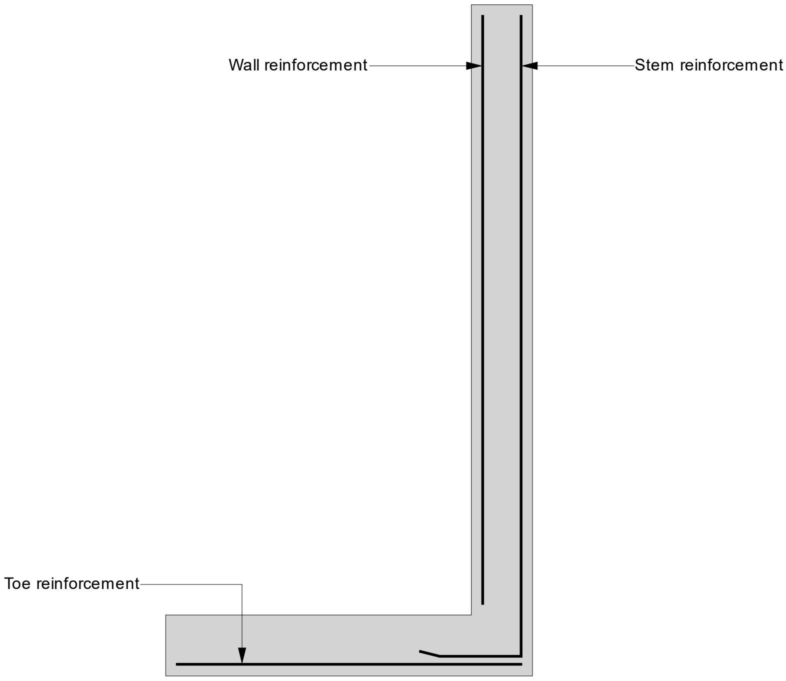
Modification factor $factor_{tens} = \min(0.55 + (477\ N/mm^2 - f_s)/(120 \times (0.9\ N/mm^2 + (M_{stem}/(b \times d_{stem}^2)))),2)$ = **2.00**

Maximum span/effective depth ratio $ratio_{max} = ratio_{bas} \times factor_{tens}$ = **40.00**

Actual span/effective depth ratio $ratio_{act} = h_{stem} / d_{stem}$ = **12.30**

PASS - Span to depth ratio is acceptable

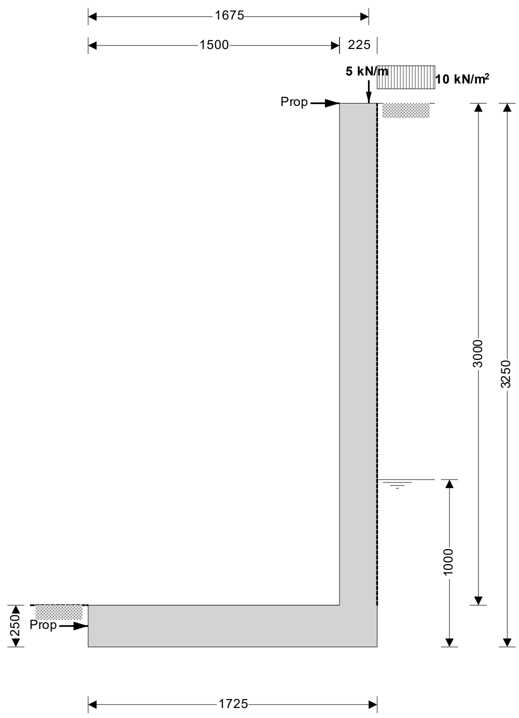
Indicative retaining wall reinforcement diagram



Toe bars - 16 mm dia.@ 150 mm centres - (1340 mm^2/m)
Wall bars - 12 mm dia.@ 200 mm centres - (565 mm^2/m)
Stem bars - 12 mm dia.@ 200 mm centres - (565 mm^2/m)

RETAINING WALL ANALYSIS (BS 8002:1994)

TEDDS calculation version 1.2.01.08



Wall details

Retaining wall type	**Cantilever propped at both**
Height of retaining wall stem	h_{stem} = **3000** mm
Thickness of wall stem	t_{wall} = **225** mm
Length of toe	l_{toe} = **1500** mm
Length of heel	l_{heel} = **0** mm
Overall length of base	$l_{base} = l_{toe} + l_{heel} + t_{wall}$ = **1725** mm
Thickness of base	t_{base} = **250** mm
Depth of downstand	d_{ds} = **0** mm
Position of downstand	l_{ds} = **200** mm
Thickness of downstand	t_{ds} = **250** mm
Height of retaining wall	$h_{wall} = h_{stem} + t_{base} + d_{ds}$ = **3250** mm
Depth of cover in front of wall	d_{cover} = **0** mm
Depth of unplanned excavation	d_{exc} = **0** mm
Height of ground water behind wall	h_{water} = **1000** mm
Height of saturated fill above base	$h_{sat} = \max(h_{water} - t_{base} - d_{ds}, 0 \text{ mm})$ = **750** mm
Density of wall construction	γ_{wall} = **23.6** kN/m^3
Density of base construction	γ_{base} = **23.6** kN/m^3
Angle of rear face of wall	α = **90.0** deg
Angle of soil surface behind wall	β = **0.0** deg
Effective height at virtual back of wall	$h_{eff} = h_{wall} + l_{heel} \times \tan(\beta)$ = **3250** mm

Retained material details

Mobilisation factor	M = **1.5**
Moist density of retained material	γ_m = **18.0** kN/m^3

Saturated density of retained material γ_s = **21.0** kN/m^3

Design shear strength ϕ' = **24.2** deg

Angle of wall friction δ = **0.0** deg

Base material details

Moist density γ_{mb} = **18.0** kN/m^3

Design shear strength ϕ'_b = **24.2** deg

Design base friction δ_b = **18.6** deg

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

Using Coulomb theory

Active pressure coefficient for retained material

$$K_a = \sin(\alpha + \phi')^2 / (\sin(\alpha)^2 \times \sin(\alpha - \delta) \times [1 + \sqrt{}(\sin(\phi' + \delta) \times \sin(\phi' - \beta) / (\sin(\alpha - \delta) \times \sin(\alpha + \beta)))]^2) = \mathbf{0.419}$$

Passive pressure coefficient for base material

$$K_p = \sin(90 - \phi'_b)^2 / (\sin(90 - \delta_b) \times [1 - \sqrt{}(\sin(\phi'_b + \delta_b) \times \sin(\phi'_b) / (\sin(90 + \delta_b)))]^2) = \mathbf{4.187}$$

At-rest pressure

At-rest pressure for retained material $K_0 = 1 - \sin(\phi')$ = **0.590**

Loading details

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

Applied vertical dead load on wall W_{dead} = **0.0** kN/m

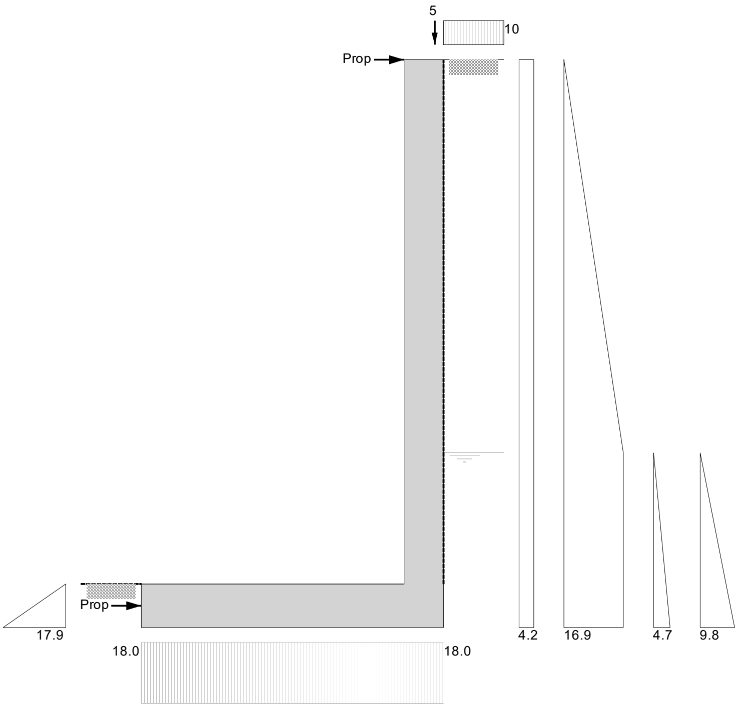
Applied vertical live load on wall W_{live} = **5.0** kN/m

Position of applied vertical load on wall l_{load} = **1675** mm

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

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

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



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

Vertical forces on wall

Wall stem $w_{wall} = h_{stem} \times t_{wall} \times \gamma_{wall}$ = **15.9** kN/m

Wall base $w_{base} = l_{base} \times t_{base} \times \gamma_{base}$ = **10.2** kN/m

Applied vertical load $W_v = W_{dead} + W_{live}$ = **5** kN/m

Total vertical load $W_{total} = w_{wall} + w_{base} + W_v$ = **31.1** kN/m

Horizontal forces on wall

Surcharge $F_{sur} = K_a \times \text{Surcharge} \times h_{eff}$ = **13.6** kN/m

Moist backfill above water table $F_{m\_a} = 0.5 \times K_a \times \gamma_m \times (h_{eff} - h_{water})^2$ = **19.1** kN/m

Moist backfill below water table $F_{m\_b} = K_a \times \gamma_m \times (h_{eff} - h_{water}) \times h_{water}$ = **16.9** kN/m

Saturated backfill $F_s = 0.5 \times K_a \times (\gamma_s - \gamma_{water}) \times h_{water}^2$ = **2.3** kN/m

Water $F_{water} = 0.5 \times h_{water}^2 \times \gamma_{water}$ = **4.9** kN/m

Total horizontal load $F_{total} = F_{sur} + F_{m\_a} + F_{m\_b} + F_s + F_{water}$ = **56.9** kN/m

Calculate total propping force

Passive resistance of soil in front of wall $F_p = 0.5 \times K_p \times \cos(\delta_b) \times (d_{cover} + t_{base} + d_{ds} - d_{exc})^2 \times \gamma_{mb}$ = **2.2** kN/m

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

F_{prop} = **45.8** kN/m

Overturning moments

Surcharge $M_{sur} = F_{sur} \times (h_{eff} - 2 \times d_{ds}) / 2$ = **22.1** kNm/m

Moist backfill above water table $M_{m\_a} = F_{m\_a} \times (h_{eff} + 2 \times h_{water} - 3 \times d_{ds}) / 3$ = **33.4** kNm/m

Moist backfill below water table $M_{m\_b} = F_{m\_b} \times (h_{water} - 2 \times d_{ds}) / 2$ = **8.5** kNm/m

Saturated backfill $M_s = F_s \times (h_{water} - 3 \times d_{ds}) / 3$ = **0.8** kNm/m

Water $M_{water} = F_{water} \times (h_{water} - 3 \times d_{ds}) / 3$ = **1.6** kNm/m

Total overturning moment $M_{ot} = M_{sur} + M_{m\_a} + M_{m\_b} + M_s + M_{water}$ = **66.4** kNm/m

Restoring moments

Wall stem $M_{wall} = w_{wall} \times (l_{toe} + t_{wall} / 2)$ = **25.7** kNm/m

Wall base $M_{base} = w_{base} \times l_{base} / 2$ = **8.8** kNm/m

Design vertical load $M_v = W_v \times l_{load}$ = **8.4** kNm/m

Total restoring moment $M_{rest} = M_{wall} + M_{base} + M_v$ = **42.8** kNm/m

Check bearing pressure

Total vertical reaction $R = W_{total}$ = **31.1** kN/m

Distance to reaction $x_{bar} = l_{base} / 2$ = **863** mm

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

Reaction acts within middle third of base

Bearing pressure at toe $p_{toe} = (R / l_{base}) - (6 \times R \times e / l_{base}^2)$ = **18** kN/m^2

Bearing pressure at heel $p_{heel} = (R / l_{base}) + (6 \times R \times e / l_{base}^2)$ = **18** kN/m^2

PASS - Maximum bearing pressure is less than allowable bearing pressure

Calculate propping forces to top and base of wall

Propping force to top of wall

$F_{prop\_top} = (M_{ot} - M_{rest} + R \times l_{base} / 2 - F_{prop} \times t_{base} / 2) / (h_{stem} + t_{base} / 2)$ = **14.279** kN/m

Propping force to base of wall $F_{prop\_base} = F_{prop} - F_{prop\_top}$ = **31.570** kN/m

RETAINING WALL DESIGN (BS 8002:1994)

TEDDS calculation version 1.2.01.08

Ultimate limit state load factors

Dead load factor γ_{f_d} = **1.4**

Live load factor γ_{f_l} = **1.6**

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

Factored vertical forces on wall

Wall stem $w_{wall\_f} = \gamma_{f\_d} \times h_{stem} \times t_{wall} \times \gamma_{wall}$ = **22.3** kN/m

Wall base $w_{base\_f} = \gamma_{f\_d} \times l_{base} \times t_{base} \times \gamma_{base}$ = **14.2** kN/m

Applied vertical load $W_{v\_f} = \gamma_{f\_d} \times W_{dead} + \gamma_{f\_l} \times W_{live}$ = **8** kN/m

Total vertical load $W_{total\_f} = w_{wall\_f} + w_{base\_f} + W_{v\_f}$ = **44.6** kN/m

Factored horizontal active forces on wall

Surcharge $F_{sur\_f} = \gamma_{f\_l} \times K_a \times \text{Surcharge} \times h_{eff}$ = **21.8** kN/m

Moist backfill above water table $F_{m\_a\_f} = \gamma_{f\_e} \times 0.5 \times K_a \times \gamma_m \times (h_{eff} - h_{water})^2$ = **26.7** kN/m

Moist backfill below water table $F_{m\_b\_f} = \gamma_{f\_e} \times K_a \times \gamma_m \times (h_{eff} - h_{water}) \times h_{water}$ = **23.7** kN/m

Saturated backfill $F_{s\_f} = \gamma_{f\_e} \times 0.5 \times K_a \times (\gamma_s - \gamma_{water}) \times h_{water}^2$ = **3.3** kN/m

Water $F_{water\_f} = \gamma_{f\_e} \times 0.5 \times h_{water}^2 \times \gamma_{water}$ = **6.9** kN/m

Total horizontal load $F_{total\_f} = F_{sur\_f} + F_{m\_a\_f} + F_{m\_b\_f} + F_{s\_f} + F_{water\_f}$ = **82.3** kN/m

Calculate total propping force

Passive resistance of soil in front of wall $F_{p\_f} = \gamma_{f\_e} \times 0.5 \times K_p \times \cos(\delta_b) \times (d_{cover} + t_{base} + d_{ds} - d_{exc})^2 \times \gamma_{mb}$ = **3.1** kN/m

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

F_{prop_f} = **66.9** kN/m

Factored overturning moments

Surcharge $M_{sur\_f} = F_{sur\_f} \times (h_{eff} - 2 \times d_{ds}) / 2$ = **35.4** kNm/m

Moist backfill above water table $M_{m\_a\_f} = F_{m\_a\_f} \times (h_{eff} + 2 \times h_{water} - 3 \times d_{ds}) / 3$ = **46.7** kNm/m

Moist backfill below water table $M_{m\_b\_f} = F_{m\_b\_f} \times (h_{water} - 2 \times d_{ds}) / 2$ = **11.9** kNm/m

Saturated backfill $M_{s\_f} = F_{s\_f} \times (h_{water} - 3 \times d_{ds}) / 3$ = **1.1** kNm/m

Water $M_{water\_f} = F_{water\_f} \times (h_{water} - 3 \times d_{ds}) / 3$ = **2.3** kNm/m

Total overturning moment $M_{ot\_f} = M_{sur\_f} + M_{m\_a\_f} + M_{m\_b\_f} + M_{s\_f} + M_{water\_f}$ = **97.3** kNm/m

Restoring moments

Wall stem $M_{wall\_f} = w_{wall\_f} \times (l_{toe} + t_{wall} / 2)$ = **36** kNm/m

Wall base $M_{base\_f} = w_{base\_f} \times l_{base} / 2$ = **12.3** kNm/m

Design vertical load $M_{v\_f} = W_{v\_f} \times l_{load}$ = **13.4** kNm/m

Total restoring moment $M_{rest\_f} = M_{wall\_f} + M_{base\_f} + M_{v\_f}$ = **61.7** kNm/m

Factored bearing pressure

Total vertical reaction $R_f = W_{total\_f}$ = **44.6** kN/m

Distance to reaction $x_{bar\_f} = l_{base} / 2$ = **863** mm

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

Reaction acts within middle third of base

Bearing pressure at toe $p_{toe\_f} = (R_f / l_{base}) - (6 \times R_f \times e_f / l_{base}^2)$ = **25.8** kN/m²

Bearing pressure at heel $p_{heel\_f} = (R_f / l_{base}) + (6 \times R_f \times e_f / l_{base}^2)$ = **25.8** kN/m²

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

Bearing pressure at stem / toe $p_{stem\_toe\_f} = \max(p_{toe\_f} - (\text{rate} \times l_{toe}), 0 \text{ kN/m}^2)$ = **25.8** kN/m²

Bearing pressure at mid stem $p_{stem\_mid\_f} = \max(p_{toe\_f} - (rate \times (l_{toe} + t_{wall} / 2)), 0\ kN/m^2) = \mathbf{25.8}\ kN/m^2$

Bearing pressure at stem / heel $p_{stem\_heel\_f} = \max(p_{toe\_f} - (rate \times (l_{toe} + t_{wall})), 0\ kN/m^2) = \mathbf{25.8}\ kN/m^2$

Calculate propping forces to top and base of wall

Propping force to top of wall

$F_{prop\_top\_f} = (M_{ot\_f} - M_{rest\_f} + R_f \times l_{base} / 2 - F_{prop\_f} \times t_{base} / 2) / (h_{stem} + t_{base} / 2) = \mathbf{21.037}\ kN/m$

Propping force to base of wall $F_{prop\_base\_f} = F_{prop\_f} - F_{prop\_top\_f} = \mathbf{45.872}\ kN/m$

Design of reinforced concrete retaining wall toe (BS 8002:1994)

Material properties

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

Characteristic strength of reinforcement $f_y = \mathbf{500}\ N/mm^2$

Base details

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

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

Calculate shear for toe design

Shear from bearing pressure $V_{toe\_bear} = (p_{toe\_f} + p_{stem\_toe\_f}) \times l_{toe} / 2 = \mathbf{38.7}\ kN/m$

Shear from weight of base $V_{toe\_wt\_base} = \gamma_{f\_d} \times \gamma_{base} \times l_{toe} \times t_{base} = \mathbf{12.4}\ kN/m$

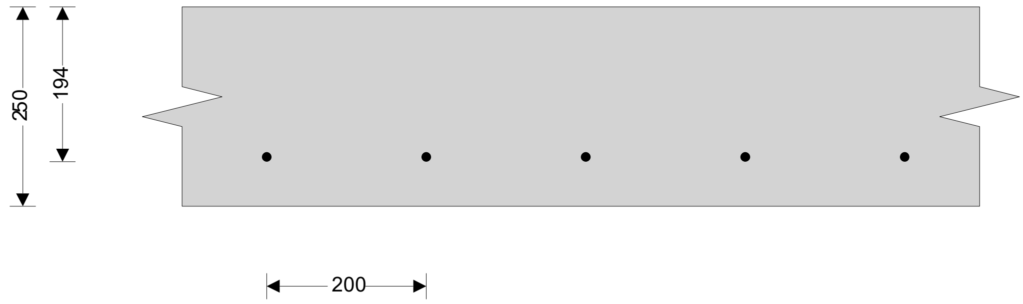
Total shear for toe design $V_{toe} = V_{toe\_bear} - V_{toe\_wt\_base} = \mathbf{26.3}\ kN/m$

Calculate moment for toe design

Moment from bearing pressure $M_{toe\_bear} = (2 \times p_{toe\_f} + p_{stem\_mid\_f}) \times (l_{toe} + t_{wall} / 2)^2 / 6 = \mathbf{33.6}\ kNm/m$

Moment from weight of base $M_{toe\_wt\_base} = (\gamma_{f\_d} \times \gamma_{base} \times t_{base} \times (l_{toe} + t_{wall} / 2)^2 / 2) = \mathbf{10.7}\ kNm/m$

Total moment for toe design $M_{toe} = M_{toe\_bear} - M_{toe\_wt\_base} = \mathbf{22.8}\ kNm/m$



Check toe in bending

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

Depth of reinforcement $d_{toe} = t_{base} - c_{toe} - (\phi_{toe} / 2) = \mathbf{194.0}\ mm$

Constant $K_{toe} = M_{toe} / (b \times d_{toe}^2 \times f_{cu}) = \mathbf{0.015}$

Compression reinforcement is not required

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

$z_{toe} = \mathbf{184}\ mm$

Area of tension reinforcement required $As_{toe\_des} = M_{toe} / (0.87 \times f_y \times z_{toe}) = \mathbf{285}\ mm^2/m$

Minimum area of tension reinforcement $As_{toe\_min} = k \times b \times t_{base} = \mathbf{325}\ mm^2/m$

Area of tension reinforcement required $As_{toe\_req} = \mathrm{Max}(As_{toe\_des}, As_{toe\_min}) = \mathbf{325}\ mm^2/m$

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

Area of reinforcement provided $As_{toe\_prov} = \mathbf{565}\ mm^2/m$

PASS - Reinforcement provided at the retaining wall toe is adequate

Check shear resistance at toe

Design shear stress $v_{toe} = V_{toe} / (b \times d_{toe}) = \mathbf{0.136}$ N/mm^2

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

PASS - Design shear stress is less than maximum shear stress

From BS8110:Part 1:1997 – Table 3.8

Design concrete shear stress $v_{c\_toe} = \mathbf{0.587}$ N/mm^2

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

Design of reinforced concrete retaining wall stem (BS 8002:1994)

Material properties

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

Characteristic strength of reinforcement $f_y = \mathbf{500}$ N/mm^2

Wall details

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

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

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

Factored horizontal active forces on stem

Surcharge $F_{s\_sur\_f} = \gamma_{f\_l} \times K_a \times \text{Surcharge} \times (h_{eff} - t_{base} - d_{ds}) = \mathbf{20.1}$ kN/m

Moist backfill above water table $F_{s\_m\_a\_f} = 0.5 \times \gamma_{f\_e} \times K_a \times \gamma_m \times (h_{eff} - t_{base} - d_{ds} - h_{sat})^2 = \mathbf{26.7}$ kN/m

Moist backfill below water table $F_{s\_m\_b\_f} = \gamma_{f\_e} \times K_a \times \gamma_m \times (h_{eff} - t_{base} - d_{ds} - h_{sat}) \times h_{sat} = \mathbf{17.8}$ kN/m

Saturated backfill $F_{s\_s\_f} = 0.5 \times \gamma_{f\_e} \times K_a \times (\gamma_s - \gamma_{water}) \times h_{sat}^2 = \mathbf{1.8}$ kN/m

Water $F_{s\_water\_f} = 0.5 \times \gamma_{f\_e} \times \gamma_{water} \times h_{sat}^2 = \mathbf{3.9}$ kN/m

Calculate shear for stem design

Surcharge $V_{s\_sur\_f} = 5 \times F_{s\_sur\_f} / 8 = \mathbf{12.6}$ kN/m

Moist backfill above water table $V_{s\_m\_a\_f} = F_{s\_m\_a\_f} \times b_l \times ((5 \times L^2) - b_l^2) / (5 \times L^3) = \mathbf{17.2}$ kN/m

Moist backfill below water table $V_{s\_m\_b\_f} = F_{s\_m\_b\_f} \times (8 - (n^2 \times (4 - n))) / 8 = \mathbf{17.1}$ kN/m

Saturated backfill $V_{s\_s\_f} = F_{s\_s\_f} \times (1 - (a_l^2 \times ((5 \times L) - a_l) / (20 \times L^3))) = \mathbf{1.8}$ kN/m

Water $V_{s\_water\_f} = F_{s\_water\_f} \times (1 - (a_l^2 \times ((5 \times L) - a_l) / (20 \times L^3))) = \mathbf{3.8}$ kN/m

Total shear for stem design $V_{stem} = V_{s\_sur\_f} + V_{s\_m\_a\_f} + V_{s\_m\_b\_f} + V_{s\_s\_f} + V_{s\_water\_f} = \mathbf{52.5}$ kN/m

Calculate moment for stem design

Surcharge $M_{s\_sur} = F_{s\_sur\_f} \times L / 8 = \mathbf{7.8}$ kNm/m

Moist backfill above water table $M_{s\_m\_a} = F_{s\_m\_a\_f} \times b_l \times ((5 \times L^2) - (3 \times b_l^2)) / (15 \times L^2) = \mathbf{13.8}$ kNm/m

Moist backfill below water table $M_{s\_m\_b} = F_{s\_m\_b\_f} \times a_l \times (2 - n)^2 / 8 = \mathbf{5.8}$ kNm/m

Saturated backfill $M_{s\_s} = F_{s\_s\_f} \times a_l \times ((3 \times a_l^2) - (15 \times a_l \times L) + (20 \times L^2)) / (60 \times L^2) = \mathbf{0.4}$ kNm/m

Water $M_{s\_water} = F_{s\_water\_f} \times a_l \times ((3 \times a_l^2) - (15 \times a_l \times L) + (20 \times L^2)) / (60 \times L^2) = \mathbf{0.9}$ kNm/m

Total moment for stem design $M_{stem} = M_{s\_sur} + M_{s\_m\_a} + M_{s\_m\_b} + M_{s\_s} + M_{s\_water} = \mathbf{28.7}$ kNm/m

Calculate moment for wall design

Surcharge $M_{w\_sur} = 9 \times F_{s\_sur\_f} \times L / 128 = \mathbf{4.4}$ kNm/m

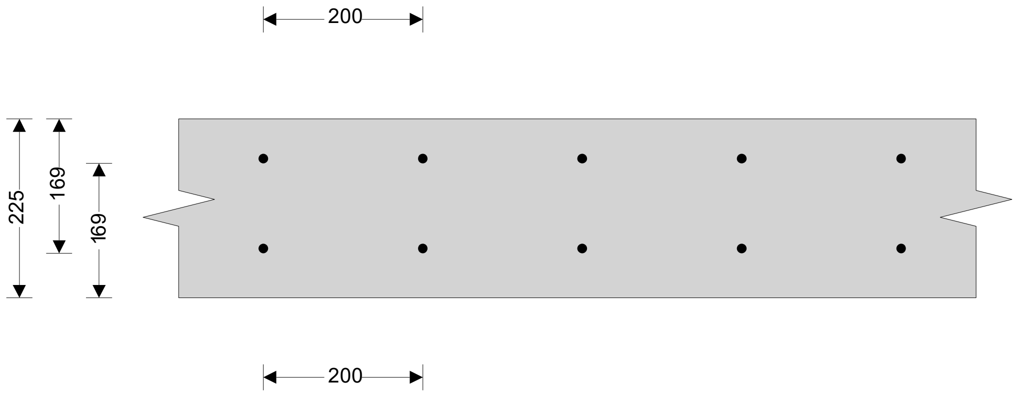
Moist backfill above water table $M_{w\_m\_a} = F_{s\_m\_a\_f} \times 0.577 \times b_l \times [(b_l^3 + 5 \times a_l \times L^2) / (5 \times L^3) - 0.577^2/3] = \mathbf{8.4}$
kNm/m

Moist backfill below water table $M_{w\_m\_b} = F_{s\_m\_b\_f} \times a_l \times [((8 - n^2 \times (4 - n))^2 / 16) - 4 + n \times (4 - n)] / 8 = \mathbf{1.5}$ kNm/m

Saturated backfill $M_{w\_s} = F_{s\_s\_f} \times [a_l^2 \times x \times ((5 \times L) - a_l) / (20 \times L^3) - (x - b_l)^3 / (3 \times a_l^2)] = \mathbf{0.1}$ kNm/m

Water $M_{w\_water} = F_{s\_water\_f} \times [a_l^2 \times x \times ((5 \times L) - a_l) / (20 \times L^3) - (x - b_l)^3 / (3 \times a_l^2)] = \mathbf{0.2}$
kNm/m

Total moment for wall design $M_{wall} = M_{w\_sur} + M_{w\_m\_a} + M_{w\_m\_b} + M_{w\_s} + M_{w\_water} = \mathbf{14.6}$ kNm/m



Check wall stem in bending

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

Depth of reinforcement — $d_{stem} = t_{wall} - c_{stem} - (\phi_{stem} / 2)$ = **169.0** mm

Constant — $K_{stem} = M_{stem} / (b \times d_{stem}^2 \times f_{cu})$ = **0.025**

Compression reinforcement is not required

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

z_{stem} = **161** mm

Area of tension reinforcement required — $As_{stem\_des} = M_{stem} / (0.87 \times f_y \times z_{stem})$ = **411** mm^2/m

Minimum area of tension reinforcement — $As_{stem\_min} = k \times b \times t_{wall}$ = **293** mm^2/m

Area of tension reinforcement required — $As_{stem\_req} = \max(As_{stem\_des}, As_{stem\_min})$ = **411** mm^2/m

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

Area of reinforcement provided — As_{stem_prov} = **565** mm^2/m

PASS - Reinforcement provided at the retaining wall stem is adequate

Check shear resistance at wall stem

Design shear stress — $v_{stem} = V_{stem} / (b \times d_{stem})$ = **0.311** N/mm^2

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

PASS - Design shear stress is less than maximum shear stress

From BS8110:Part 1:1997 – Table 3.8

Design concrete shear stress — v_{c_stem} = **0.637** N/mm^2

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

Check mid height of wall in bending

Depth of reinforcement — $d_{wall} = t_{wall} - c_{wall} - (\phi_{wall} / 2)$ = **169.0** mm

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

Compression reinforcement is not required

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

z_{wall} = **161** mm

Area of tension reinforcement required — $As_{wall\_des} = M_{wall} / (0.87 \times f_y \times z_{wall})$ = **209** mm^2/m

Minimum area of tension reinforcement — $As_{wall\_min} = k \times b \times t_{wall}$ = **293** mm^2/m

Area of tension reinforcement required — $As_{wall\_req} = \max(As_{wall\_des}, As_{wall\_min})$ = **293** mm^2/m

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

Area of reinforcement provided — As_{wall_prov} = **565** mm^2/m

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

Check retaining wall deflection

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

Design service stress $f_s = 2 \times f_y \times A_{s\_stem\_req} / (3 \times A_{s\_stem\_prov})$ = **242.5** N/mm^2

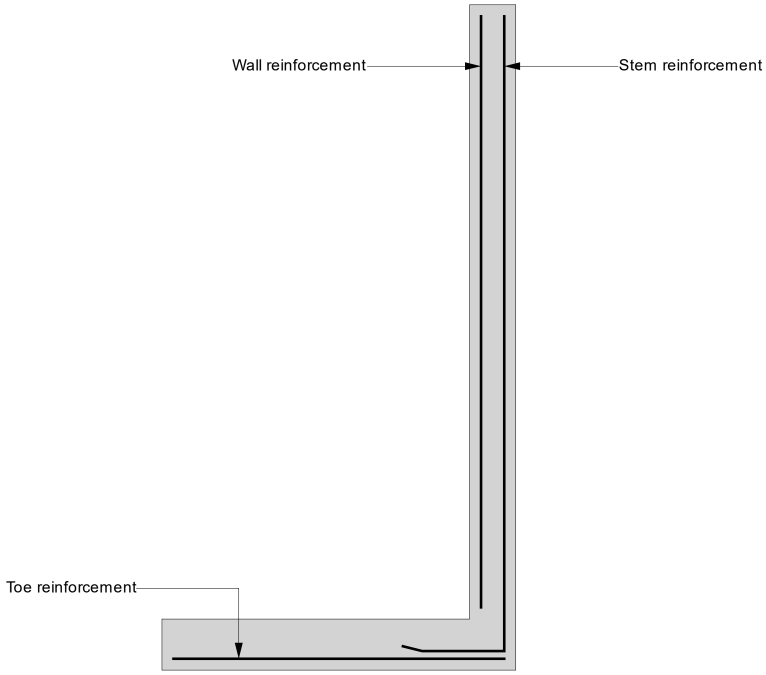
Modification factor $factor_{tens} = min(0.55 + (477 N/mm^2 - f_s)/(120 \times (0.9 N/mm^2 + (M_{stem}/(b \times d_{stem}^2)))),2)$ = **1.58**

Maximum span/effective depth ratio $ratio_{max} = ratio_{bas} \times factor_{tens}$ = **31.50**

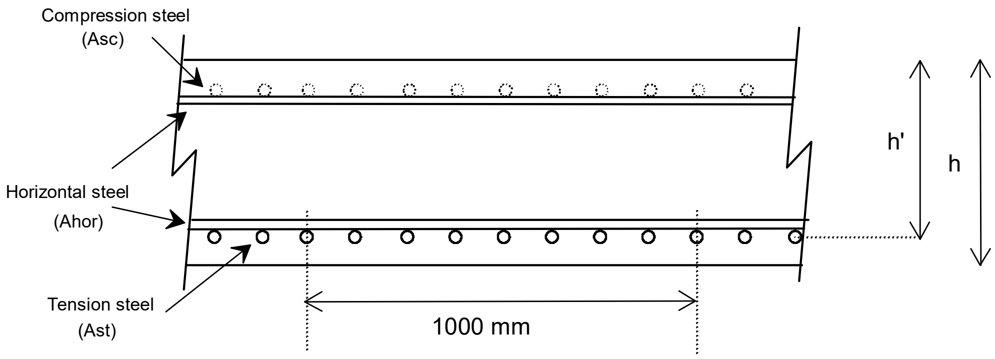
Actual span/effective depth ratio $ratio_{act} = h_{stem} / d_{stem}$ = **17.75**

PASS - Span to depth ratio is acceptable

Indicative retaining wall reinforcement diagram



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



Wall
(assumed symmetric)

RC WALL DESIGN (BS8110) WALL DESIGN TO CL 3.9.3

TEDDS calculation version 1.0.04

WALL DEFINITION

Wall thickness h = **250** mm

Cover to tension reinforcement c_w = **35** mm

Trial bar diameter D_{try} = **12** mm

Depth to tension steel

$h' = h - c_w - D_{try}/2$ = **209** mm

Materials

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

Characteristic strength of concrete f_{cu} = **35** N/mm^2

Braced Wall Design to cl 3.9.3 (Simply supported construction)

Stocky check for braced walls

Wall clear height l_o = **3000** mm

Effective height factor for simply supported braced walls (assessed for a plain wall)

β = **1.00**

$l_e = \beta \times l_o$ = **3.000** m l_e/h = **12.00**

The braced wall is stocky

Braced wall slenderness check

Effective wall height l_e = **3000** mm

Slenderness limit $l_{limit} = 40 \times h$ = **10000** mm

Slenderness limit $l_{limit1} = 45 \times h$ = **11250** mm

Wall slenderness limit OK

Define wall reinforcement

Main reinforcement in wall

<u>Provide 12 dia bars @ 200 centres in each face</u>

Area of "tension" steel A_{st} = A_{svert} = **565** mm^2/m

Area of compression steel A_{sc} = A_{st} = **565** mm^2/m

Total area of steel A_{wall} = A_{st} + A_{sc} = **1130.0** mm^2/m

Percentage of steel (A_{st} + A_{sc}) / h = **0.45** %

<u>HORIZONTAL WALL STEEL</u>

Wall thickness h = **250** mm

Area of vertical steel provided A_{wall} = **1130** mm^2/m

Percentage of vertical steel p_{vwall} = A_{wall} / h = **0.45** %

Minimum diameter of horizontal steel D_{min} = max(D_{vert}/4 , 6 mm) = **6** mm

Minimum area of horizontal steel

A_{Hmin} = If(f_y>=(460 N/mm^2),if(p_{vwall}>2 %,0.13 %,0.25%),if(p_{vwall}>2 %,0.24 %, 0.30 %)) × h/2

A_{Hmin} =**313** mm^2/m

No containment links required

Define horizontal wall steel in one face

<u>Provide 10 dia bars @ 200 centres in each face</u>

Stocky wall (simple construction) - transverse bending and axial load

Design ultimate loading

Design ultimate axial load per m of wall n_w = **70** kN/m

Design ultimate transverse moment per m of wall m_w = **17.5** kNm/m

Minimum design moments

m_{min} = min(0.05 × h, 20 mm) × n_w = **0.9** kNm/m

Design moments

m_{design} = max(abs(m_w), m_{min}) = **17.5** kNm/m

<u>CHECK OF DESIGN FORCES - SYMMETRICALLY REINFORCED WALL SECTION</u>

<u>NOTES</u>

h is the wall thickness

h' is the depth from the more highly compressed face to the "tension" steel.

Tension steel yields

Determine correct moment of resistance

n_R = if(x_{calc}<h/0.9, n_{R1} , n_{R2}) = **335.5** kN/m

m_R = if(x_{calc}<h/0.9, m_{R1} , m_{R2}) = **82.6** kNm/m

Applied axial load

n_w = **70.0** kN/m

Check for moment

m_{design} = **17.5** kNm/m

Moment check satisfied

The wall vertical reinforcement defined in each face is H12 dia bars @ 200 centres

CHECK MIN AND MAX AREAS OF STEEL

Overall thickness of wall h = **250** mm

Vertical steel

Total area of concrete per m run of wall A_c = h = **250000** mm^2/m

A_{st_min} = 0.4% × A_c = **1000** mm^2/m

A_{st_max} = 4 % × A_c = **10000** mm^2/m

Total vertical steel in wall A_{wall} = **1130** mm^2/m

Area of vertical steel in wall provided OK

Horizontal steel

Percentage of vertical steel p_{vwall} = A_{wall} / h = **0.45** %

Diameter of horizontal steel D_{hor} = **10** mm

Minimum diameter of horizontal steel D_{min} = max(D_{vert}/4,6 mm) = **6** mm

Diameter of horizontal steel in wall OK

Area of horizontal steel in one face A_{shor} = **393** mm^2/m

Minimum area of horizontal steel

A_{Hmin} = If(f_y>=(460 N/mm^2),if(p_{vwall}>2 %,0.13 %,0.25%),if(p_{vwall}>2 %,0.24,0.30 %)) × h/2

A_{Hmin} =**313** mm^2/m

Area of horizontal steel in wall provided OK

Shear Resistance of Concrete Walls - (cl 3.8.4.6)

Wall thickness h = **250** mm

Effective depth to steel h' = **209** mm

Area of concrete A_{conc} = h = **250000** mm^2/m

Design ultimate shear force through thickness per m of wall v_w = **6** kN/m

Characteristic strength of concrete f_{cu} = **35** N/mm^2

Is a check required? (3.8.4.6)

Axial load per m of wall n_w = **70.0** kN/m

Major axis moment per m of wall m_w = **17.5** kNm/m

$e = m_w / n_w$ = **250.0** mm

$e_{limit} = 0.6 \times h$ = **150.0** mm

Actual shear stress $v_x = v_w / h'$ = **0.0** N/mm^2

Allowable stress $v_{allowable} = \min((0.8\ N^{1/2}/mm) \times \sqrt{(f_{cu})}, 5\ N/mm^2)$ = **4.733** N/mm^2

Shear check required

Design shear stress to clause 3.4.5.12

f_{cu_ratio} = if (f_{cu} > 40 N/mm^2 , 40/25 , f_{cu}/(25 N/mm^2)) = **1.400**

Design concrete shear stress

$v_c = 0.79\ N/mm^2 \times \min(3,100 \times A_{st} / h')^{1/3} \times \max(1,(400\ mm) / h')^{1/4} / 1.25 * f_{cu\_ratio}{}^{1/3}$

v_c = **0.538** N/mm^2

$v_c' = v_c + 0.6 \times n_w / h \times \min(\text{abs}(v_w) \times h / m_w, 1.0)$ = **0.6** N/mm^2

$v_{allowable} = \min((0.8\ N^{1/2}/mm) \times \sqrt{(f_{cu})}, v_c', 5\ N/mm^2)$ = **0.552** N/mm^2

Actual shear stress

v_x = **0.0** N/mm^2

Shear reinforcement not necessarily required in wall

Shear stress - OK

Check of nominal cover - (BS8110:Pt 1, Table 3.4)

Wall thickness h = **250** mm

Depth to tension steel from compression face h' = **209** mm

Diameter of vertical reinforcement D_{vert} = **12** mm

Diameter of links L_{dia} = **0** mm

Cover to tension reinforcement

$c_{ten} = h - h' - D_{vert} / 2$ = **35.0** mm

Nominal cover to links steel

$c_{nom} = c_{ten} - L_{dia}$ = **35.0** mm

Permissable minimum nominal cover to all reinforcement (Table 3.4)

c_{min} = **35** mm

Cover OK

SERVICEABILITY LIMIT STATE - CRACKING IN WALLS

(BS8110:Pt 2, Cl. 3.8 & BS8007 Cl 2.6 & Appendix B)

Design serviceability loading

For a conservative assessment of crack widths, the axial compression and the compression reinforcement in the wall will be ignored.

Serviceability transverse moment per m of wall m_{SLS} = **9** kNm/m

Wall thickness h = **250** mm

Depth to steel h' = **209** mm

Characteristic strength of concrete f_{cu} = **35** N/mm^2

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

BS8110:Pt 1:Table 3.1

Diameter of wall vertical reinforcement D_{vert} = **12** mm

Spacing of vertical reinforcement bars s_{vert} = **200** mm

Area of vertical reinforcement in one face $A_{st} = \pi \times D_{vert}^2 /4 / s_{vert}$ = **565** mm^2/m

Effective depth to tension reinforcement

h' = **209.0** mm

Cover to tension reinforcement

c_{ten} = h - h' - D_{vert}/2 = **35** mm

Nominal cover to tension reinforcement

$c_{nom} = c_{ten}$ = **35.0** mm

Tension bar centres

$bar_{crs} = s_{vert}$ = **200.0** mm

MODULAR RATIO

Modulus of elasticity for reinforcement E_s = 200 kN/mm^2

BS8110:Pt 1:Cl 2.5.4

Modulus of elasticity for concrete (half the instantaneous)

E_c = ((20 kN/mm^2) + 200×f_{cu}) / 2 = **14** kN/mm^2

BS8110:Pt 2:Equation 17

Modular ratio m = E_s / E_c = **14.815**

NEUTRAL AXIS POSITION

For equilibrium F_{st} equates F_c

Therefore: m × A_{st} × [f_c×(h'-x)/x] equates to 0.5 × f_c × x

Solving for x gives the position of the neutral axis in the section:-

$x = h' \times [-1\times E_s\times A_{st}/(E_c\times h') + \sqrt{(E_s\times A_{st}/(E_c\times h') \times (2+E_s\times A_{st}/(E_c\times h')))}] =$ **51.4** mm

Depth of concrete in compression

$x =$ **51.4** mm

CONCRETE AND STEEL STRESSES

The serviceability limit state moment per m of wall $m_{SLS} =$ **9** kNm/m

Taking moments about the centreline of the reinforcement:-

Moment of resistance of concrete is $0.5 \times f_c \times x \times (h' - x/3)$

Solving for concrete stress f_c gives

$f_c = 2 \times m_{SLS} / (x \times (h' - x/3)) =$ **1.83** N/mm^2

Allowable stress $0.45 \times f_{cu} =$ **15.75** N/mm^2

Concrete stress OK

Taking moments about the centre of action of the concrete force:-

Moment of resistance of steel is $f_{st} \times A_s \times (h' - x/3)$

Solving for steel stress f_{st} gives

$f_{st} = m_{SLS} / (A_{st} \times (h' - x/3)) =$ **82.95** N/mm^2

CONCRETE AND STEEL STRAINS

Strain in the reinforcement

$\varepsilon_s = f_{st} / E_s =$ **414.7×10^{-6}**

Allowable steel strain $0.8 \times f_y / E_s =$ **2.000×10^{-3}**

Steel strain OK

BS8007:App B.4

Strain in the concrete at the level at which crack width is required

Level of crack $a' = h =$ **250** mm

$\varepsilon_1 = \varepsilon_s \times (a' - x)/(h' - x) =$ **522.6×10^{-6}**

Strain in the concrete at the level at which crack width is required adjusted for stiffening of the concrete tension zone

Allowable crack width $Crack_{Allowable} =$ **0.2** mm

BS8007:Cl 2.2.3.3

Factor for stiffening based on limiting crack width

factor = if($Crack_{Allowable}$ == (0.2 mm), (1.0 N/mm^2), (1.5 N/mm^2)) = **1** N/mm^2

$\varepsilon_m = \min(\varepsilon_1 ,\max(0, \varepsilon_1 - [factor \times (h - x) \times (a' - x) / (3\times E_s\times A_{st} \times(h' - x))])) =$ **0.0000**

BS8007:Cl 2.2.3.3

Distance from tension bar to crack in tension face between tension bars

$a_{cr} = \sqrt{((bar_{crs}/2)^2 + (c_{nom} + D_{vert}/2)^2)} - D_{vert}/2 =$ **102.1** mm

Design crack width

$Crack_{design} = 3 \times a_{cr} \times \varepsilon_m /(1 + 2\times(a_{cr} - c_{ten})/(h - x))$ = **0.000** mm

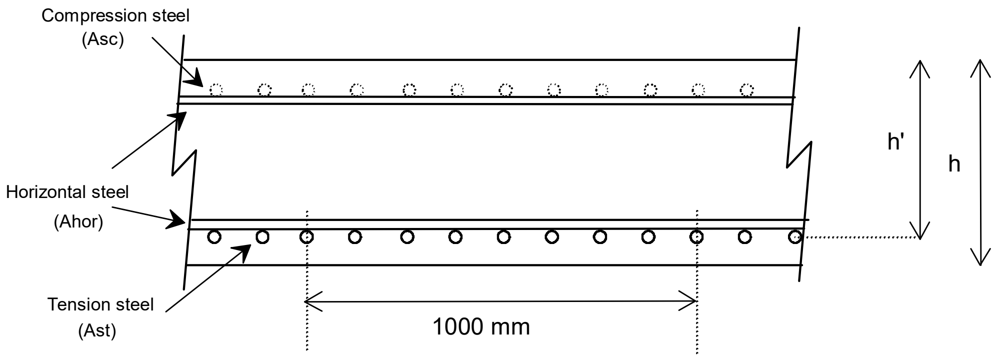
BS8007:App B.3

Max allowable crack width

$Crack_{Allowable}$ = **0.20** mm

BS8007:Cl 2.2.3.3

Design Crack width OK



Wall
(assumed symmetric)

RC WALL DESIGN (BS8110) WALL DESIGN TO CL 3.9.3

TEDDS calculation version 1.0.04

WALL DEFINITION

Wall thickness h = **300** mm

Cover to tension reinforcement c_w = **35** mm

Trial bar diameter D_{try} = **12** mm

Depth to tension steel

$h' = h - c_w - D_{try}/2$ = **259** mm

Materials

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

Characteristic strength of concrete f_{cu} = **35** N/mm^2

Braced Wall Design to cl 3.9.3 (Simply supported construction)

Stocky check for braced walls

Wall clear height l_o = **3000** mm

Effective height factor for simply supported braced walls (assessed for a plain wall)

β = **1.00**

$l_e = \beta \times l_o$ = **3.000** m l_e/h = **10.00**

The braced wall is stocky

Braced wall slenderness check

Effective wall height l_e = **3000** mm

Slenderness limit $l_{limit} = 40 \times h$ = **12000** mm

Slenderness limit $l_{limit1} = 45 \times h$ = **13500** mm

Wall slenderness limit OK

Define wall reinforcement

Main reinforcement in wall

<u>Provide 12 dia bars @ 200 centres in each face</u>

Area of "tension" steel $A_{st} = A_{svert}$ = **565** mm^2/m

Area of compression steel $A_{sc} = A_{st}$ = **565** mm^2/m

Total area of steel $A_{wall} = A_{st} + A_{sc}$ = **1130.0** mm^2/m

Percentage of steel $(A_{st} + A_{sc}) / h$ = **0.38** %

<u>HORIZONTAL WALL STEEL</u>

Wall thickness h = **300** mm

Area of vertical steel provided A_{wall} = **1130** mm^2/m

Percentage of vertical steel $p_{vwall} = A_{wall} / h$ = **0.38** %

Minimum diameter of horizontal steel $D_{min} = \max(D_{vert}/4 , 6 \text{ mm})$ = **6** mm

Minimum area of horizontal steel

A_{Hmin} = If(f_y>=(460 N/mm^2),if(p_{vwall}>2 %,0.13 %,0.25%),if(p_{vwall}>2 %,0.24 %, 0.30 %)) × h/2

A_{Hmin} =**375** mm^2/m

No containment links required

Define horizontal wall steel in one face

<u>Provide 10 dia bars @ 200 centres in each face</u>

Stocky wall (simple construction) - transverse bending and axial load

Design ultimate loading

Design ultimate axial load per m of wall n_w = **70** kN/m

Design ultimate transverse moment per m of wall m_w = **17.5** kNm/m

Minimum design moments

$m_{min} = \min(0.05 \times h, 20 \text{ mm}) \times n_w$ = **1.1** kNm/m

Design moments

$m_{design} = \max(\text{abs}(m_w), m_{min})$ = **17.5** kNm/m

<u>CHECK OF DESIGN FORCES - SYMMETRICALLY REINFORCED WALL SECTION</u>

<u>NOTES</u>

h is the wall thickness

h' is the depth from the more highly compressed face to the "tension" steel.

Tension steel yields

Determine correct moment of resistance

n_R = if(x_{calc}<h/0.9, n_{R1} , n_{R2}) = **335.5** kN/m

m_R = if(x_{calc}<h/0.9, m_{R1} , m_{R2}) = **103.2** kNm/m

Applied axial load

n_w = **70.0** kN/m

Check for moment

m_{design} = **17.5** kNm/m

Moment check satisfied

The wall vertical reinforcement defined in each face is H12 dia bars @ 200 centres

CHECK MIN AND MAX AREAS OF STEEL

Overall thickness of wall h = **300** mm

Vertical steel

Total area of concrete per m run of wall A_c = h = **300000** mm^2/m

A_{st_min} = 0.4% × A_c = **1200** mm^2/m

A_{st_max} = 4 % × A_c = **12000** mm^2/m

Total vertical steel in wall A_{wall} = **1130** mm^2/m

Less than min area of vertical steel in wall - FAIL

Horizontal steel

Percentage of vertical steel p_{vwall} = A_{wall} / h = **0.38** %

Diameter of horizontal steel D_{hor} = **10** mm

Minimum diameter of horizontal steel D_{min} = max(D_{vert}/4,6 mm) = **6** mm

Diameter of horizontal steel in wall OK

Area of horizontal steel in one face A_{shor} = **393** mm^2/m

Minimum area of horizontal steel

A_{Hmin} = If(f_y>=(460 N/mm^2),if(p_{vwall}>2 %,0.13 %,0.25%),if(p_{vwall}>2 %,0.24,0.30 %)) × h/2

A_{Hmin} =**375** mm^2/m

Area of horizontal steel in wall provided OK

Shear Resistance of Concrete Walls - (cl 3.8.4.6)

Wall thickness h = **300** mm

Effective depth to steel h' = **259** mm

Area of concrete A_{conc} = h = **300000** mm^2/m

Design ultimate shear force through thickness per m of wall v_w = **6** kN/m

Characteristic strength of concrete f_{cu} = **35** N/mm^2

Is a check required? (3.8.4.6)

Axial load per m of wall n_w = **70.0** kN/m

Major axis moment per m of wall m_w = **17.5** kNm/m

e = m_w / n_w = **250.0** mm

e_{limit} = 0.6 × h = **180.0** mm

Actual shear stress v_x = v_w / h' = **0.0** N/mm^2

Allowable stress $v_{allowable}$ = min ((0.8 N$^{1/2}$/mm) × √(f_{cu}), 5 N/mm^2) = **4.733** N/mm^2

Shear check required

Design shear stress to clause 3.4.5.12

f_{cu_ratio} = if (f_{cu} > 40 N/mm^2 , 40/25 , f_{cu}/(25 N/mm^2)) = **1.400**

Design concrete shear stress

v_c = 0.79 N/mm^2 × min(3,100 × A_{st} / h')$^{1/3}$ × max(1,(400 mm) / h')$^{1/4}$ / 1.25 * $f_{cu\_ratio}{}^{1/3}$

v_c = **0.474** N/mm^2

v_c' = v_c + 0.6 × n_w / h × min(abs(v_w) × h / m_w, 1.0) = **0.5** N/mm^2

$v_{allowable}$ = min ((0.8 N$^{1/2}$/mm) × √(f_{cu}), v_c' , 5 N/mm^2) = **0.489** N/mm^2

Actual shear stress

v_x = **0.0** N/mm^2

Shear reinforcement not necessarily required in wall

Shear stress - OK

Check of nominal cover - (BS8110:Pt 1, Table 3.4)

Wall thickness h = **300** mm

Depth to tension steel from compression face h' = **259** mm

Diameter of vertical reinforcement D_{vert} = **12** mm

Diameter of links L_{dia} = **0** mm

Cover to tension reinforcement

c_{ten} = h - h' - D_{vert} / 2 = **35.0** mm

Nominal cover to links steel

c_{nom} = c_{ten} - L_{dia} = **35.0** mm

Permissable minimum nominal cover to all reinforcement (Table 3.4)

c_{min} = **35** mm

Cover OK

SERVICEABILITY LIMIT STATE - CRACKING IN WALLS

(BS8110:Pt 2, Cl. 3.8 & BS8007 Cl 2.6 & Appendix B)

Design serviceability loading

For a conservative assessment of crack widths, the axial compression and the compression reinforcement in the wall will be ignored.

Serviceability transverse moment per m of wall m_{SLS} = **9** kNm/m

Wall thickness h = **300** mm

Depth to steel h' = **259** mm

Characteristic strength of concrete f_{cu} = **35** N/mm^2

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

BS8110:Pt 1:Table 3.1

Diameter of wall vertical reinforcement D_{vert} = **12** mm

Spacing of vertical reinforcement bars s_{vert} = **200** mm

Area of vertical reinforcement in one face $A_{st} = \pi \times D_{vert}^2 /4 / s_{vert}$ = **565** mm^2/m

Effective depth to tension reinforcement

h' = **259.0** mm

Cover to tension reinforcement

c_{ten} = h - h' - D_{vert}/2 = **35** mm

Nominal cover to tension reinforcement

$c_{nom} = c_{ten}$ = **35.0** mm

Tension bar centres

$bar_{crs} = s_{vert}$ = **200.0** mm

MODULAR RATIO

Modulus of elasticity for reinforcement E_s = 200 kN/mm^2

BS8110:Pt 1:Cl 2.5.4

Modulus of elasticity for concrete (half the instantaneous)

E_c = ((20 kN/mm^2) + 200×f_{cu}) / 2 = **14** kN/mm^2

BS8110:Pt 2:Equation 17

Modular ratio m = E_s / E_c = **14.815**

NEUTRAL AXIS POSITION

For equilibrium F_{st} equates F_c

Therefore: $m \times A_{st} \times [\, f_c \times (h'-x)/x \,]$ equates to $0.5 \times f_c \times x$

Solving for x gives the position of the neutral axis in the section:-

$x = h' \times [-1 \times E_s \times A_{st}/(E_c \times h') + \sqrt{(E_s \times A_{st}/(E_c \times h') \times (2+E_s \times A_{st}/(E_c \times h')))}] =$ **58.0** mm

Depth of concrete in compression

$x =$ **58.0** mm

CONCRETE AND STEEL STRESSES

The serviceability limit state moment per m of wall $m_{SLS} =$ **9** kNm/m

Taking moments about the centreline of the reinforcement:-

Moment of resistance of concrete is $0.5 \times f_c \times x \times (h' - x/3)$

Solving for concrete stress f_c gives

$f_c = 2 \times m_{SLS} / (x \times (h' - x/3)) =$ **1.29** N/mm^2

Allowable stress $0.45 \times f_{cu} =$ **15.75** N/mm^2

Concrete stress OK

Taking moments about the centre of action of the concrete force:-

Moment of resistance of steel is $f_{st} \times A_s \times (h' - x/3)$

Solving for steel stress f_{st} gives

$f_{st} = m_{SLS} / (A_{st} \times (h' - x/3)) =$ **66.41** N/mm^2

CONCRETE AND STEEL STRAINS

Strain in the reinforcement

$\varepsilon_s = f_{st} / E_s =$ **332.0×10^{-6}**

Allowable steel strain $0.8 \times f_y / E_s =$ **2.000×10^{-3}**

Steel strain OK

BS8007:App B.4

Strain in the concrete at the level at which crack width is required

Level of crack $a' = h =$ **300** mm

$\varepsilon_1 = \varepsilon_s \times (a' - x)/(h' - x) =$ **399.8×10^{-6}**

Strain in the concrete at the level at which crack width is required adjusted for stiffening of the concrete tension zone

Allowable crack width $Crack_{Allowable} =$ **0.2** mm

BS8007:Cl 2.2.3.3

Factor for stiffening based on limiting crack width

factor = if($Crack_{Allowable}$ == (0.2 mm), (1.0 N/mm^2), (1.5 N/mm^2)) = **1** N/mm^2

$\varepsilon_m = \min(\varepsilon_1 ,\max(0, \varepsilon_1 - [factor \times (h - x) \times (a' - x) / (3 \times E_s \times A_{st} \times (h' - x))])) =$ **0.0000**

BS8007:Cl 2.2.3.3

Distance from tension bar to crack in tension face between tension bars

$a_{cr} = \sqrt{ (bar_{crs}/2)^2 + (c_{nom} + D_{vert}/2)^2} - D_{vert}/2 =$ **102.1** mm

Design crack width

$Crack_{design} = 3 \times a_{cr} \times \varepsilon_m /(1 + 2\times(a_{cr} - c_{ten})/(h - x))$ = **0.000** mm

BS8007:App B.3

Max allowable crack width

$Crack_{Allowable}$ = **0.20** mm

BS8007:Cl 2.2.3.3

Design Crack width OK

RC SLAB DESIGN (BS8110:PART1:1997)

TEDDS calculation version 1.0.04

CONCRETE SLAB DESIGN (CL 3.5.3 & 4)

SIMPLE ONE WAY SPANNING SLAB DEFINITION

Overall depth of slab h = **150** mm

Cover to tension reinforcement resisting sagging c_b = **50** mm

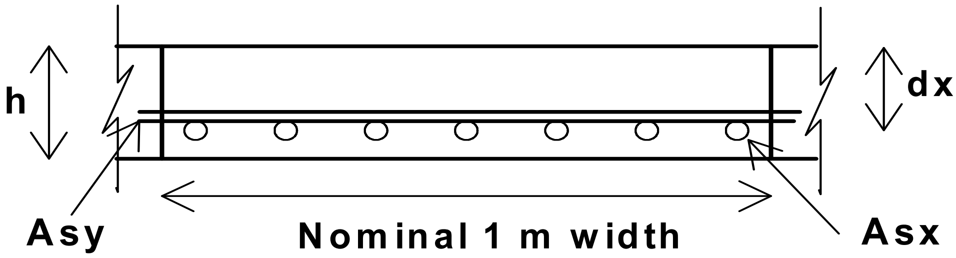
Trial bar diameter D_{tryx} = **10** mm

Depth to tension steel (resisting sagging)

$d_x = h - c_b - D_{tryx}/2$ = **95** mm

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

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



One-way spanning slab

(simple)

ONE WAY SPANNING SLAB (CL 3.5.4)

MAXIMUM DESIGN MOMENTS IN SPAN

Design sagging moment (per m width of slab) m_{sx} = **6.0** kNm/m

CONCRETE SLAB DESIGN – SAGGING – OUTER LAYER OF STEEL (CL 3.5.4)

Design sagging moment (per m width of slab) m_{sx} = **6.0** kNm/m

Moment Redistribution Factor β_{bx} = **1.0**

Area of reinforcement required

$K_x = abs(m_{sx}) / (d_x^2 \times f_{cu})$ = **0.017**

$K'_x = \min (0.156 , (0.402 \times (\beta_{bx} - 0.4)) - (0.18 \times (\beta_{bx} - 0.4)^2))$ = **0.156**

Outer compression steel not required to resist sagging

One-way Spanning Slab requiring tension steel only (sagging) - mesh

$z_x = \min ((0.95 \times d_x),(d_x \times (0.5+\sqrt{(0.25-K_x/0.9)})))$ = **90** mm

Neutral axis depth $x_x = (d_x - z_x) / 0.45$ = **11** mm

Area of tension steel required

A_{sx_req} = abs(m_{sx}) / (1/γ_{ms} × f_y × z_x) = **153** mm²/m

Tension steel

Use A393 Mesh

A_{sx_prov} = A_{sl} = **393** mm²/m A_{sy_prov} = A_{st} = **393** mm²/m

D_x = d_{sl} = **10** mm D_y = d_{st} = **10** mm

Area of tension steel provided sufficient to resist sagging

Check min and max areas of steel resisting sagging

Total area of concrete A_c = h = **150000** mm²/m

Minimum % reinforcement k = **0.13** %

A_{st_min} = k × A_c = **195** mm²/m

A_{st_max} = 4 % × A_c = **6000** mm²/m

Steel defined:

Outer steel resisting sagging A_{sx_prov} = **393** mm²/m

Area of outer steel provided (sagging) OK

Inner steel resisting sagging A_{sy_prov} = **393** mm²/m

Area of inner steel provided (sagging) OK

SHEAR RESISTANCE OF CONCRETE SLABS (CL 3.5.5)

Outer tension steel resisting sagging moments

Depth to tension steel from compression face d_x = **95** mm

Area of tension reinforcement provided (per m width of slab) A_{sx_prov} = **393** mm²/m

Design ultimate shear force (per m width of slab) V_x = **12** kN/m

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

Applied shear stress

v_x = V_x / d_x = **0.13** N/mm²

Check shear stress to clause 3.5.5.2

$v_{allowable}$ = min ((0.8 $N^{1/2}$/mm) × √(f_{cu}), 5 N/mm²) = **5.00** N/mm²

Shear stress - OK

Shear stresses to clause 3.5.5.3

Design shear stress

f_{cu_ratio} = if (f_{cu} > 40 N/mm² , 40/25 , f_{cu}/(25 N/mm²)) = **1.600**

v_{cx} = 0.79 N/mm² × min(3,100 × A_{sx_prov} / d_x)$^{1/3}$ × max(0.67,(400 mm / d_x)$^{1/4}$) / 1.25 × $f_{cu\_ratio}^{1/3}$

v_{cx} = **0.79** N/mm²

Applied shear stress

v_x = **0.13** N/mm²

No shear reinforcement required

CONCRETE SLAB DEFLECTION CHECK (CL 3.5.7)

Slab span length l_x = **2.000** m

Design ultimate moment in shorter span per m width m_{sx} = **6** kNm/m

Depth to outer tension steel d_x = **95** mm

Tension steel

Area of outer tension reinforcement provided A_{sx_prov} = **393** mm^2/m

Area of tension reinforcement required A_{sx_req} = **153** mm^2/m

Moment Redistribution Factor β_{bx} = **1.00**

Modification Factors

Basic span / effective depth ratio (Table 3.9) $ratio_{span\_depth}$ = **20**

The modification factor for spans in excess of 10m (ref. cl 3.4.6.4) has not been included.

$f_s = 2 \times f_y \times A_{sx\_req} / (3 \times A_{sx\_prov} \times \beta_{bx})$ = **129.7** N/mm^2

$factor_{tens} = \min(2, 0.55 + (477\text{ N/mm}^2 - f_s) / (120 \times (0.9\text{ N/mm}^2 + m_{sx} / d_x^2)))$ = **2.000**

Calculate Maximum Span

This is a simplified approach and further attention should be given where special circumstances exist. Refer to clauses 3.4.6.4 and 3.4.6.7.

Maximum span $l_{max} = ratio_{span\_depth} \times factor_{tens} \times d_x$ = **3.80** m

Check the actual beam span

Actual span/depth ratio l_x / d_x = **21.05**

Span depth limit $ratio_{span\_depth} \times factor_{tens}$ = **40.00**

Span/Depth ratio check satisfied

CHECK OF NOMINAL COVER (SAGGING) – (BS8110:PT 1, TABLE 3.4)

Slab thickness h = **150** mm

Effective depth to bottom outer tension reinforcement d_x = **95.0** mm

Diameter of tension reinforcement D_x = **10** mm

Diameter of links L_{diax} = **0** mm

Cover to outer tension reinforcement

$c_{tenx} = h - d_x - D_x / 2$ = **50.0** mm

Nominal cover to links steel

$c_{nomx} = c_{tenx} - L_{diax}$ = **50.0** mm

Permissable minimum nominal cover to all reinforcement (Table 3.4)

c_{min} = **50** mm

Cover over steel resisting sagging OK

RC SLAB DESIGN (BS8110:PART1:1997)

TEDDS calculation version 1.0.04

TWO WAY SPANNING SLAB DEFINITION – SIMPLY SUPPORTED

Overall depth of slab h = **200** mm

Outer sagging steel

Cover to outer tension reinforcement resisting sagging c_{sag} = **35** mm

Trial bar diameter D_{tryx} = **10** mm

Depth to outer tension steel (resisting sagging)

$d_x = h - c_{sag} - D_{tryx}/2$ = **160** mm

Inner sagging steel

Trial bar diameter D_{tryy} = **10** mm

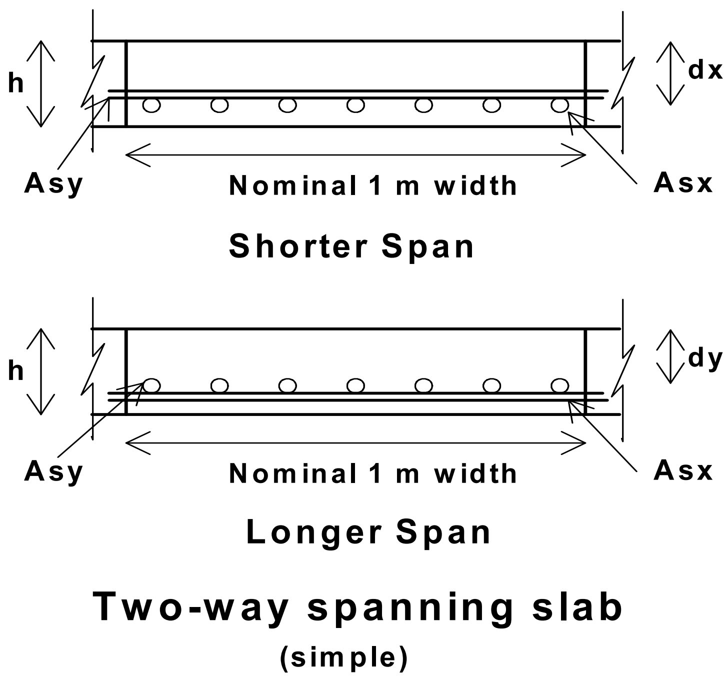
Depth to inner tension steel (resisting sagging)

$d_y = h - c_{sag} - D_{tryx} - D_{tryy}/2$ = **150** mm

Materials

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

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



MAXIMUM DESIGN MOMENTS

Length of shorter side of slab l_x = **3.400** m

Length of longer side of slab l_y = **5.000** m

Design ultimate load per unit area n_s = **12.0** kN/m^2

Moment coefficients

$\alpha_{sx} = (l_y / l_x)^4 / (8 \times (1+(l_y / l_x)^4))$ = **0.103**

$\alpha_{sy} = (l_y / l_x)^2 / (8 \times (1+(l_y / l_x)^4))$ = **0.048**

Maximum moments per unit width - simply supported slabs

$m_{sx} = \alpha_{sx} \times n_s \times l_x^2$ = **14.3** kNm/m

$m_{sy} = \alpha_{sy} \times n_s \times l_x^2$ = **6.6** kNm/m

CONCRETE SLAB DESIGN – SAGGING – OUTER LAYER OF STEEL (CL 3.5.4)

Design sagging moment (per m width of slab) m_{sx} = **14.3** kNm/m

Moment Redistribution Factor β_{bx} = **1.0**

Area of reinforcement required

$K_x = abs(m_{sx}) / (d_x^2 \times f_{cu})$ = **0.014**

$K'_x = min (0.156 , (0.402 \times (\beta_{bx} - 0.4)) - (0.18 \times (\beta_{bx} - 0.4)^2))$ = **0.156**

Outer compression steel not required to resist sagging

Concrete Slab Design - Sagging - Inner layer of steel (cl. 3.5.4)

Design sagging moment (per m width of slab) m_{sy} = **6.6** kNm/m

Moment Redistribution Factor β_{by} = **1.0**

Area of reinforcement required

$K_y = abs(m_{sy}) / (d_y^2 \times f_{cu})$ = **0.007**

$K'_y = min (0.156 , (0.402 \times (\beta_{by} - 0.4)) - (0.18 \times (\beta_{by} - 0.4)^2))$ = **0.156**

Inner compression steel not required to resist sagging

Two way Spanning Slab requiring tension steel only - mesh (sagging)

$z_x = min ((0.95 \times d_x),(d_x \times (0.5+\sqrt{(0.25-K_x/0.9)})))$ = **152** mm

Neutral axis depth $x_x = (d_x - z_x) / 0.45$ = **18** mm

$z_y = min ((0.95 \times d_y),(d_y \times (0.5+\sqrt{(0.25-K_y/0.9)})))$ = **142** mm

Neutral axis depth $x_y = (d_y - z_y) / 0.45$ = **17** mm

Area of outer tension steel required

$A_{sx\_req} = abs(m_{sx}) / (1/\gamma_{ms} \times f_y \times z_x)$ = **216** mm^2/m

Area of inner tension steel required

$A_{sy\_req} = abs(m_{sy}) / (1/\gamma_{ms} \times f_y \times z_y)$ = **107** mm^2/m

Tension steel

Provide A393 Mesh tension steel resisting sagging

$A_{sx\_prov} = A_{sl}$ = **393** mm^2/m $A_{sy\_prov} = A_{st}$ = **393** mm^2/m

$D_x = d_{sl} =$ **10** mm $D_y = d_{st} =$ **10** mm

Area of tension steel provided sufficient to resist sagging

<u>Check min and max areas of steel resisting sagging</u>

Total area of concrete $A_c = h =$ **200000** mm^2/m

Minimum % reinforcement $k =$ **0.13** %

$A_{st\_min} = k \times A_c =$ **260** mm^2/m

$A_{st\_max} = 4\ \% \times A_c =$ **8000** mm^2/m

Steel defined:

Outer steel resisting sagging $A_{sx\_prov} =$ **393** mm^2/m

Area of outer steel provided (sagging) OK

Inner steel resisting sagging $A_{sy\_prov} =$ **393** mm^2/m

Area of inner steel provided (sagging) OK

<u>SHEAR RESISTANCE OF CONCRETE SLABS (CL 3.5.5)</u>

Outer tension steel resisting sagging moments

Depth to tension steel from compression face $d_x =$ **160** mm

Area of tension reinforcement provided (per m width of slab) $A_{sx\_prov} =$ **393** mm^2/m

Design ultimate shear force (per m width of slab) $V_x =$ **20** kN/m

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

Applied shear stress

$v_x = V_x / d_x =$ **0.13** N/mm^2

Check shear stress to clause 3.5.5.2

$v_{allowable} = \min((0.8\ N^{1/2}/mm) \times \sqrt{(f_{cu})}, 5\ N/mm^2) =$ **5.00** N/mm^2

Shear stress - OK

Shear stresses to clause 3.5.5.3

Design shear stress

$f_{cu\_ratio} = \text{if}\ (f_{cu} > 40\ N/mm^2, 40/25, f_{cu}/(25\ N/mm^2)) =$ **1.600**

$v_{cx} = 0.79\ N/mm^2 \times \min(3, 100 \times A_{sx\_prov} / d_x)^{1/3} \times \max(0.67, (400\ mm / d_x)^{1/4}) / 1.25 \times f_{cu\_ratio}^{1/3}$

$v_{cx} =$ **0.58** N/mm^2

Applied shear stress

$v_x =$ **0.13** N/mm^2

No shear reinforcement required

<u>SHEAR RESISTANCE OF CONCRETE SLABS (CL 3.5.5)</u>

Inner tension steel resisting sagging moments

Depth to tension steel from compression face $d_y =$ **150** mm

Area of tension reinforcement provided (per m width of slab) $A_{sy\_prov} =$ **393** mm^2/m

Design ultimate shear force (per m width of slab) V_y = **30** kN/m

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

Applied shear stress

$v_y = V_y / d_y$ = **0.20** N/mm^2

Check shear stress to clause 3.5.5.2

$v_{allowable}$ = min ((0.8 N$^{1/2}$/mm) × √(f_{cu}), 5 N/mm^2) = **5.00** N/mm^2

Shear stress - OK

Shear stresses to clause 3.5.5.3

Design shear stress

f_{cu_ratio} = if (f_{cu} > 40 N/mm^2 , 40/25 , f_{cu}/(25 N/mm^2)) = **1.600**

v_{cy} = 0.79 N/mm^2 × min(3,100 × A_{sy_prov} / d_y)$^{1/3}$ × max(0.67,(400 mm) / d_y)$^{1/4}$ / 1.25 × $f_{cu\_ratio}^{1/3}$

v_{cy} = **0.60** N/mm^2

Applied shear stress

v_y = **0.20** N/mm^2

No shear reinforcement required

CONCRETE SLAB DEFLECTION CHECK (CL 3.5.7)

Slab span length l_x = **3.400** m

Design ultimate moment in shorter span per m width m_{sx} = **14** kNm/m

Depth to outer tension steel d_x = **160** mm

Tension steel

Area of outer tension reinforcement provided A_{sx_prov} = **393** mm^2/m

Area of tension reinforcement required A_{sx_req} = **216** mm^2/m

Moment Redistribution Factor β_{bx} = **1.00**

Modification Factors

Basic span / effective depth ratio (Table 3.9) $ratio_{span\_depth}$ = **26**

The modification factor for spans in excess of 10m (ref. cl 3.4.6.4) has not been included.

$f_s = 2 \times f_y \times A_{sx\_req} / (3 \times A_{sx\_prov} \times \beta_{bx})$ = **183.3** N/mm^2

$factor_{tens}$ = min (2 , 0.55 + (477 N/mm^2 - f_s) / (120 × (0.9 N/mm^2 + m_{sx} / d_x^2))) = **2.000**

Calculate Maximum Span

This is a simplified approach and further attention should be given where special circumstances exist. Refer to clauses 3.4.6.4 and 3.4.6.7.

Maximum span l_{max} = $ratio_{span\_depth} \times factor_{tens} \times d_x$ = **8.32** m

Check the actual beam span

Actual span/depth ratio l_x / d_x = **21.25**

Span depth limit $ratio_{span\_depth} \times factor_{tens}$ = **52.00**

Span/Depth ratio check satisfied

CHECK OF NOMINAL COVER (SAGGING) – (BS8110:PT 1, TABLE 3.4)

Slab thickness h = **200** mm

Effective depth to bottom outer tension reinforcement d_x = **160.0** mm

Diameter of tension reinforcement D_x = **10** mm

Diameter of links L_{diax} = **0** mm

Cover to outer tension reinforcement

$c_{tenx} = h - d_x - D_x / 2$ = **35.0** mm

Nominal cover to links steel

$c_{nomx} = c_{tenx} - L_{diax}$ = **35.0** mm

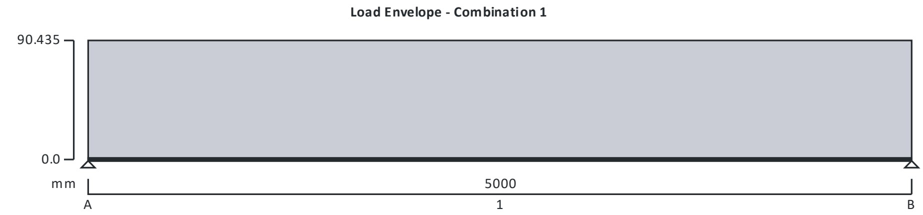
Permissable minimum nominal cover to all reinforcement (Table 3.4)

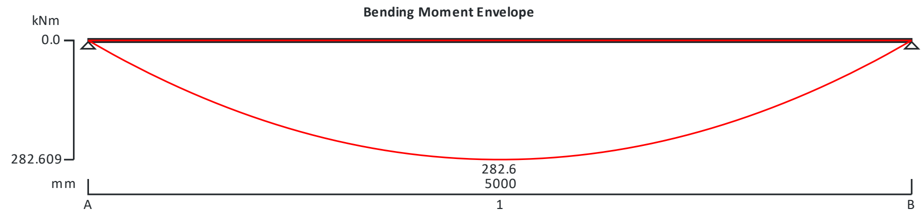
c_{min} = **35** mm

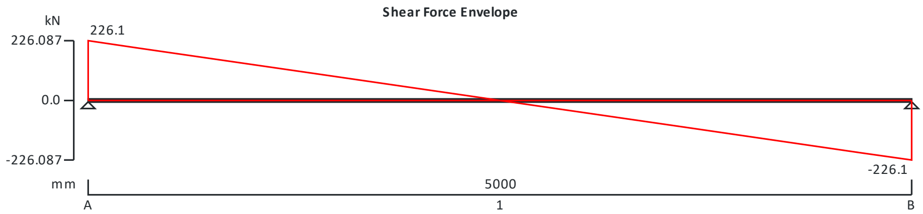
Cover over steel resisting sagging OK

RC BEAM ANALYSIS & DESIGN BS8110

TEDDS calculation version 2.1.14







Support conditions

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

Applied loading

Roof	Imposed full UDL 1.73 kN/m
Roof	Dead full UDL 2.78 kN/m
Frst floor	Imposed full UDL 3 kN/m
Frst floor	Dead full UDL 1.18 kN/m
Wall	Dead full UDL 25 kN/m
con floor	Imposed full UDL 8.75 kN/m
con floor	Dead full UDL 10.5 kN/m
	Dead self weight of beam × 1

Load combinations

Load combination 1	Support A	Dead × 1.40
		Imposed × 1.60
	Span 1	Dead × 1.40
		Imposed × 1.60

Support B — Dead × 1.40

Imposed × 1.60

Analysis results

Maximum moment support A	M_{A_max} = **0** kNm	M_{A_red} = **0** kNm
Maximum moment span 1 at 2500 mm	M_{s1_max} = **283** kNm	M_{s1_red} = **283** kNm
Maximum moment support B	M_{B_max} = **0** kNm	M_{B_red} = **0** kNm
Maximum shear support A	V_{A_max} = **226** kN	V_{A_red} = **226** kN
Maximum shear support A span 1 at 401 mm	$V_{A\_s1\_max}$ = **185** kN	$V_{A\_s1\_red}$ = **185** kN
Maximum shear support B	V_{B_max} = **-226** kN	V_{B_red} = **-226** kN
Maximum shear support B span 1 at 4599 mm	$V_{B\_s1\_max}$ = **-185** kN	$V_{B\_s1\_red}$ = **-185** kN
Maximum reaction at support A	R_A = **226** kN	
Unfactored dead load reaction at support A	R_{A_Dead} = **123** kN	
Unfactored imposed load reaction at support A	$R_{A\_Imposed}$ = **34** kN	
Maximum reaction at support B	R_B = **226** kN	
Unfactored dead load reaction at support B	R_{B_Dead} = **123** kN	
Unfactored imposed load reaction at support B	$R_{B\_Imposed}$ = **34** kN	

Rectangular section details

Section width — b = **900** mm

Section depth — h = **450** mm

Concrete details

Concrete strength class — **C32/40**

Characteristic compressive cube strength — f_{cu} = **40** N/mm²

Modulus of elasticity of concrete — $E_c = 20kN/mm^2 + 200 \times f_{cu}$ = **28000** N/mm²

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

Reinforcement details

Characteristic yield strength of reinforcement — f_y = **500** N/mm²

Characteristic yield strength of shear reinforcement — f_{yv} = **500** N/mm²

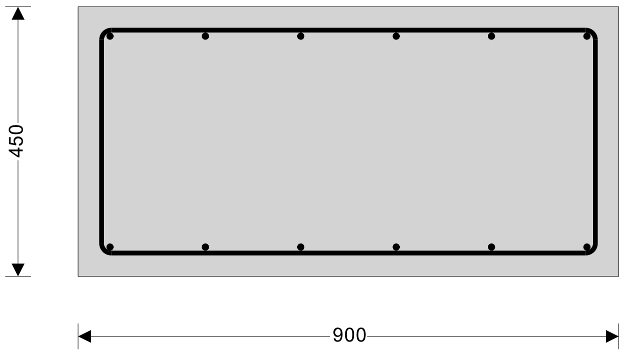
Nominal cover to reinforcement

Nominal cover to top reinforcement — c_{nom_t} = **35** mm

Nominal cover to bottom reinforcement — c_{nom_b} = **35** mm

Nominal cover to side reinforcement — c_{nom_s} = **35** mm

Support A



6 x 12ϕ bars

4 x 8ϕ shear legs at 200 c/c

6 x 12ϕ bars

Rectangular section in shear

Design shear force span 1 at 401 mm — $V = \max(V_{A\_s1\_max}, V_{A\_s1\_red}) = \mathbf{185}$ kN

Design shear stress — $v = V / (b \times d) = \mathbf{0.513}$ N/mm^2

Design concrete shear stress — $v_c = 0.79 \times \min(3,[100 \times A_{s,prov} / (b \times d)]^{1/3}) \times \max(1, (400 /d)^{1/4}) \times (\min(f_{cu}, 40) / 25)^{1/3} / \gamma_m$

$v_c = \mathbf{0.423}$ N/mm^2

Allowable design shear stress — $v_{max} = \min(0.8 \text{ N/mm}^2 \times (f_{cu}/1 \text{ N/mm}^2)^{0.5}, 5 \text{ N/mm}^2) = \mathbf{5.000}$ N/mm^2

PASS - Design shear stress is less than maximum allowable

Value of v from Table 3.7 — $0.5 \times v_c < v < (v_c + 0.4 \text{ N/mm}^2)$

Design shear resistance required — $v_s = \max(v - v_c, 0.4 \text{ N/mm}^2) = \mathbf{0.400}$ N/mm^2

Area of shear reinforcement required — $A_{sv,req} = v_s \times b / (0.87 \times f_{yv}) = \mathbf{828}$ mm^2/m

Shear reinforcement provided — 4 × 8ϕ legs at 200 c/c

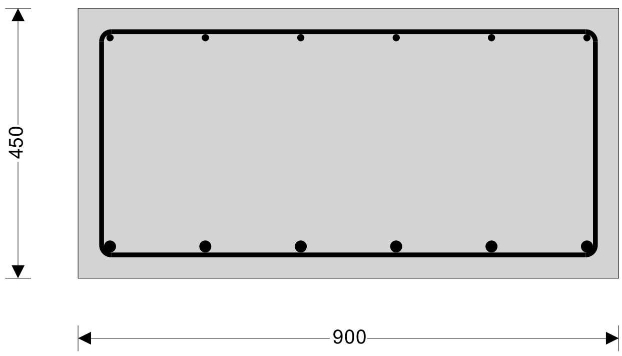
Area of shear reinforcement provided — $A_{sv,prov} = \mathbf{1005}$ mm^2/m

PASS - Area of shear reinforcement provided exceeds minimum required

Maximum longitudinal spacing — $s_{vl,max} = 0.75 \times d = \mathbf{301}$ mm

PASS - Longitudinal spacing of shear reinforcement provided is less than maximum

Mid span 1



6 x 12ϕ bars

4 x 8ϕ shear legs at 200 c/c

6 x 20ϕ bars

Design moment resistance of rectangular section (cl. 3.4.4) - Positive moment

Design bending moment — $M = \text{abs}(M_{s1\_red}) = \mathbf{283}$ kNm

Depth to tension reinforcement — $d = h - c_{nom\_b} - \phi_v - \phi_{bot} / 2 = \mathbf{397}$ mm

Redistribution ratio — $\beta_b = \min(1 - m_{rs1}, 1) = \mathbf{1.000}$

$K = M / (b \times d^2 \times f_{cu}) = \mathbf{0.050}$

$K' = 0.156$

K' > K - No compression reinforcement is required

Lever arm $z = \min(d \times (0.5 + (0.25 - K / 0.9)^{0.5}), 0.95 \times d) = \mathbf{374}$ mm

Depth of neutral axis $x = (d - z) / 0.45 = \mathbf{52}$ mm

Area of tension reinforcement required $A_{s,req} = M / (0.87 \times f_y \times z) = \mathbf{1739}$ mm^2

Tension reinforcement provided 6 × 20ϕ bars

Area of tension reinforcement provided $A_{s,prov} = \mathbf{1885}$ mm^2

Minimum area of reinforcement $A_{s,min} = 0.0013 \times b \times h = \mathbf{527}$ mm^2

Maximum area of reinforcement $A_{s,max} = 0.04 \times b \times h = \mathbf{16200}$ mm^2

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

Rectangular section in shear

Shear reinforcement provided 4 × 8ϕ legs at 200 c/c

Area of shear reinforcement provided $A_{sv,prov} = \mathbf{1005}$ mm^2/m

Minimum area of shear reinforcement (Table 3.7) $A_{sv,min} = 0.4N/mm^2 \times b / (0.87 \times f_{yv}) = \mathbf{828}$ mm^2/m

PASS - Area of shear reinforcement provided exceeds minimum required

Maximum longitudinal spacing (cl. 3.4.5.5) $s_{vl,max} = 0.75 \times d = \mathbf{298}$ mm

PASS - Longitudinal spacing of shear reinforcement provided is less than maximum

Design concrete shear stress $v_c = 0.79N/mm^2 \times \min(3,[100 \times A_{s,prov} / (b \times d)]^{1/3}) \times \max(1, (400mm /d)^{1/4}) \times (\min(f_{cu}, 40N/mm^2) / 25N/mm^2)^{1/3} / \gamma_m = \mathbf{0.598}$ N/mm^2

Design shear resistance provided $v_{s,prov} = A_{sv,prov} \times 0.87 \times f_{yv} / b = \mathbf{0.486}$ N/mm^2

Design shear stress provided $v_{prov} = v_{s,prov} + v_c = \mathbf{1.084}$ N/mm^2

Design shear resistance $V_{prov} = v_{prov} \times (b \times d) = \mathbf{387.4}$ kN

Shear links provided valid between 0 mm and 5000 mm with tension reinforcement of 1885 mm^2

Spacing of reinforcement (cl 3.12.11)

Actual distance between bars in tension $s = (b - 2 \times (c_{nom\_s} + \phi_v + \phi_{bot}/2)) /(N_{bot} - 1) - \phi_{bot} = \mathbf{139}$ mm

Minimum distance between bars in tension (cl 3.12.11.1)

Minimum distance between bars in tension $s_{min} = h_{agg} + 5$ mm $= \mathbf{25}$ mm

PASS - Satisfies the minimum spacing criteria

Maximum distance between bars in tension (cl 3.12.11.2)

Design service stress $f_s = (2 \times f_y \times A_{s,req}) / (3 \times A_{s,prov} \times \beta_b) = \mathbf{307.5}$ N/mm^2

Maximum distance between bars in tension $s_{max} = \min(47000\ N/mm / f_s, 300\ mm) = \mathbf{153}$ mm

PASS - Satisfies the maximum spacing criteria

Span to depth ratio (cl. 3.4.6)

Basic span to depth ratio (Table 3.9) $span\_to\_depth_{basic} = \mathbf{20.0}$

Design service stress in tension reinforcement $f_s = (2 \times f_y \times A_{s,req})/ (3 \times A_{s,prov} \times \beta_b) = \mathbf{307.5}$ N/mm^2

Modification for tension reinforcement

$$f_{tens} = \min(2.0, 0.55 + (477N/mm^2 - f_s) / (120 \times (0.9N/mm^2 + (M / (b \times d^2))))) = \mathbf{1.038}$$

Modification for compression reinforcement

$$f_{comp} = \min(1.5, 1 + (100 \times A_{s2,prov} / (b \times d)) / (3 + (100 \times A_{s2,prov} / (b \times d)))) = \mathbf{1.060}$$

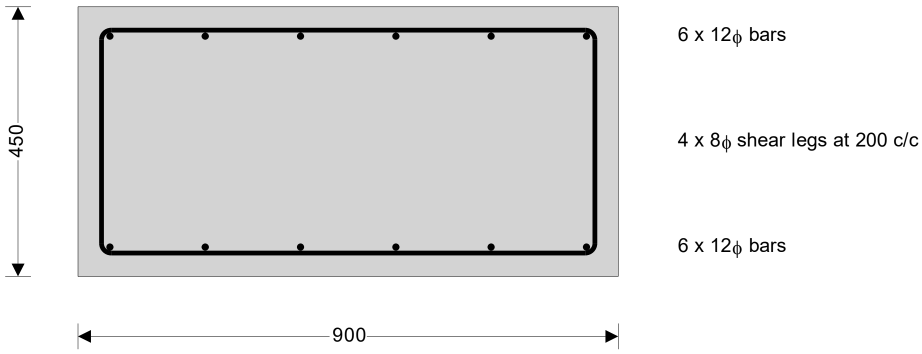
Modification for span length $f_{long} = 1.000$

Allowable span to depth ratio $span\_to\_depth_{allow} = span\_to\_depth_{basic} \times f_{tens} \times f_{comp} = \mathbf{22.0}$

Actual span to depth ratio $span\_to\_depth_{actual} = L_{s1} / d = \mathbf{12.6}$

PASS - Actual span to depth ratio is within the allowable limit

Support B



Rectangular section in shear

Design shear force span 1 at 4599 mm — $V = abs(min(V_{B\_s1\_max}, V_{B\_s1\_red}))$ = **185** kN

Design shear stress — $v = V / (b \times d)$ = **0.513** N/mm^2

Design concrete shear stress — $v_c = 0.79 \times min(3,[100 \times A_{s,prov} / (b \times d)]^{1/3}) \times max(1, (400 /d)^{1/4}) \times (min(f_{cu}, 40) / 25)^{1/3} / \gamma_m$

v_c = **0.423** N/mm^2

Allowable design shear stress — $v_{max} = min(0.8 N/mm^2 \times (f_{cu}/1 N/mm^2)^{0.5}, 5 N/mm^2)$ = **5.000** N/mm^2

PASS - Design shear stress is less than maximum allowable

Value of v from Table 3.7 — $0.5 \times v_c < v < (v_c + 0.4 N/mm^2)$

Design shear resistance required — $v_s = max(v - v_c, 0.4 N/mm^2)$ = **0.400** N/mm^2

Area of shear reinforcement required — $A_{sv,req} = v_s \times b / (0.87 \times f_{yv})$ = **828** mm^2/m

Shear reinforcement provided — 4 × 8ϕ legs at 200 c/c

Area of shear reinforcement provided — $A_{sv,prov}$ = **1005** mm^2/m

PASS - Area of shear reinforcement provided exceeds minimum required

Maximum longitudinal spacing — $s_{vl,max} = 0.75 \times d$ = **301** mm

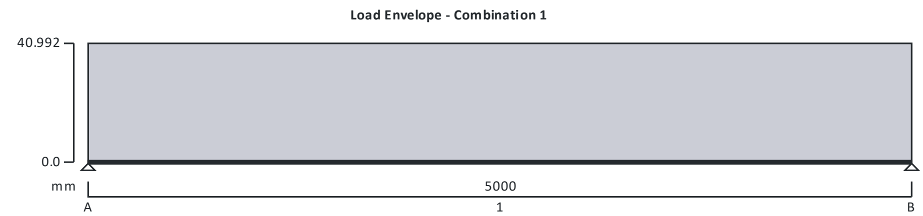
PASS - Longitudinal spacing of shear reinforcement provided is less than maximum

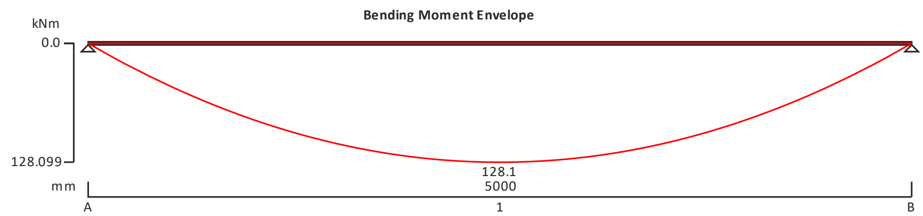
Tekla Tedds NMN Partnership 9 Chamberlain Lane Pinner HA5 2PH	Project 26 Amyand Park Road TW1 3HE				Job no. 23 227	
	Calcs for Ground foor RC slab support beam				Start page no./Revision B13 91	
	Calcs by NM	Calcs date 03/05/2024	Checked by	Checked date	Approved by	Approved date

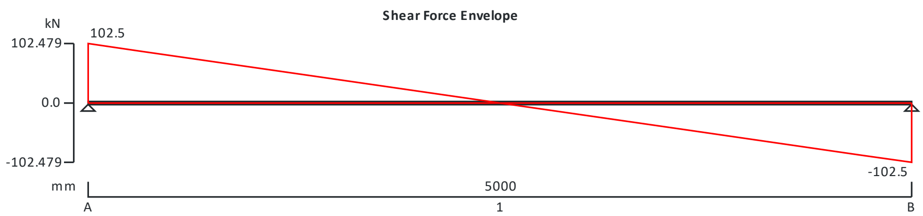
STEEL BEAM ANALYSIS & DESIGN (BS5950)

In accordance with BS5950-1:2000 incorporating Corrigendum No.1

TEDDS calculation version 3.0.07







Support conditions

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

Applied loading

Beam loads	Screed & finishes - Dead full UDL 1 kN/m
	Partiiton - Dead full UDL 1.87 kN/m
	Ground floor - Imposed full UDL 6.62 kN/m
	Ground floor RC slab - Dead full UDL 18 kN/m
	Dead self weight of beam × 1

Load combinations

Load combination 1	Support A	Dead × 1.40
		Imposed × 1.60
		Dead × 1.40
		Imposed × 1.60
	Support B	Dead × 1.40

Imposed × 1.60

Analysis results

Maximum moment	M_{max} = **128.1** kNm	M_{min} = **0** kNm
Maximum shear	V_{max} = **102.5** kN	V_{min} = **-102.5** kN
Deflection	δ_{max} = **11.9** mm	δ_{min} = **0** mm
Maximum reaction at support A	R_{A_max} = **102.5** kN	R_{A_min} = **102.5** kN
Unfactored dead load reaction at support A	R_{A_Dead} = **54.3** kN	
Unfactored imposed load reaction at support A	$R_{A\_Imposed}$ = **16.6** kN	
Maximum reaction at support B	R_{B_max} = **102.5** kN	R_{B_min} = **102.5** kN
Unfactored dead load reaction at support B	R_{B_Dead} = **54.3** kN	
Unfactored imposed load reaction at support B	$R_{B\_Imposed}$ = **16.6** kN	

Section details

Section type **UKC 203x203x86 (Tata Steel Advance)**

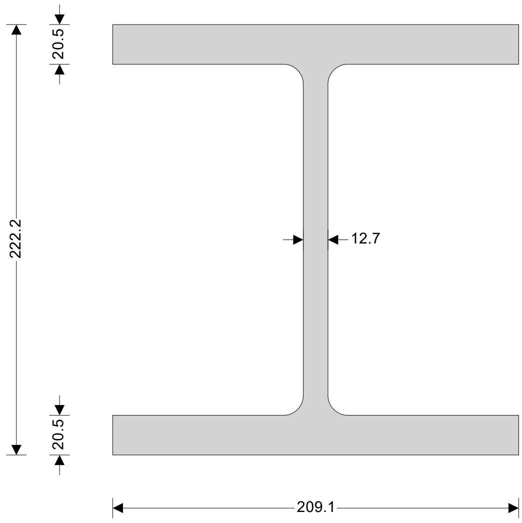
Steel grade **S275**

From table 9: Design strength p_y

Thickness of element max(T, t) = **20.5** mm

Design strength p_y = **265** N/mm^2

Modulus of elasticity E = **205000** N/mm^2



Lateral restraint

Span 1 has full lateral restraint

Effective length factors

Effective length factor in major axis K_x = **1.00**

Effective length factor in minor axis K_y = **1.00**

Effective length factor for lateral-torsional buckling $K_{LT.A}$ = **1.00**

$K_{LT.B}$ = **1.00**

Classification of cross sections - Section 3.5

$\varepsilon = \sqrt{[275 \text{ N/mm}^2 / p_y]}$ = **1.02**

Internal compression parts - Table 11

Depth of section d = **160.8** mm

$d / t = 12.4 \times \varepsilon \le 80 \times \varepsilon$ Class 1 plastic

Outstand flanges - Table 11

Width of section — $b = B / 2 =$ **104.6** mm

$b / T = 5.0 \times \varepsilon <= 9 \times \varepsilon$ — Class 1 plastic

Section is class 1 plastic

Shear capacity - Section 4.2.3

Design shear force — $F_v = max(abs(V_{max}), abs(V_{min})) =$ **102.5** kN

$d / t < 70 \times \varepsilon$

Web does not need to be checked for shear buckling

Shear area — $A_v = t \times D =$ **2822** mm^2

Design shear resistance — $P_v = 0.6 \times p_y \times A_v =$ **448.7** kN

PASS - Design shear resistance exceeds design shear force

Moment capacity - Section 4.2.5

Design bending moment — $M = max(abs(M_{s1\_max}), abs(M_{s1\_min})) =$ **128.1** kNm

Moment capacity low shear - cl.4.2.5.2 — $M_c = min(p_y \times S_{xx}, 1.2 \times p_y \times Z_{xx}) =$ **258.8** kNm

PASS - Moment capacity exceeds design bending moment

Check vertical deflection - Section 2.5.2

Consider deflection due to dead and imposed loads

Limiting deflection — $\delta_{lim} = L_{s1} / 360 =$ **13.889** mm

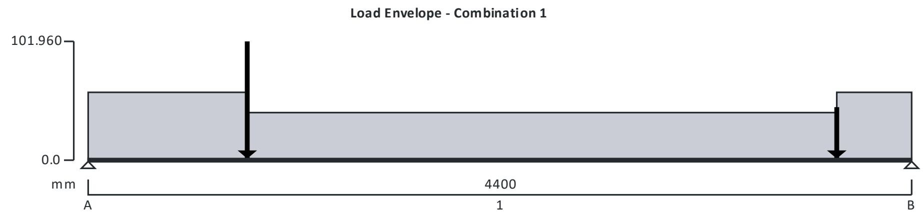
Maximum deflection span 1 — $\delta = max(abs(\delta_{max}), abs(\delta_{min})) =$ **11.904** mm

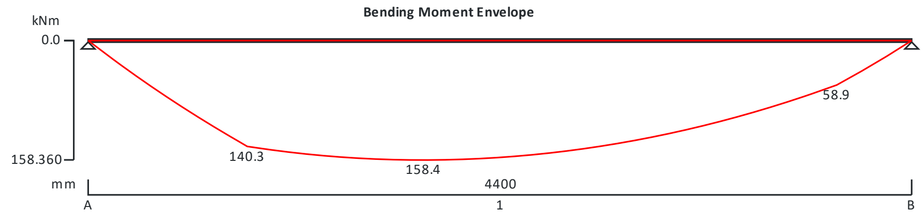
PASS - Maximum deflection does not exceed deflection limit

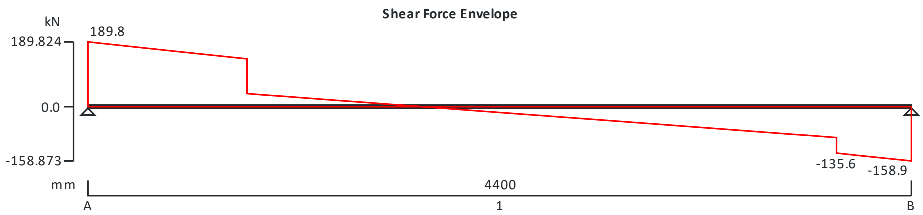
STEEL BEAM ANALYSIS & DESIGN (BS5950)

In accordance with BS5950-1:2000 incorporating Corrigendum No.1

TEDDS calculation version 3.0.07







Support conditions

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

Applied loading

Beam loads	reaction B10 - Imposed point load 1.5 kN at 4000 mm
	reaction B10 - Dead point load 30.8 kN at 4000 mm
	reaction B10 - Imposed point load 9.3 kN at 850 mm
	reaction B10 - Dead point load 62.2 kN at 850 mm
	Wall - Dead partial UDL 12.48 kN/m from 4000 mm to 4800 mm
	Wall - Dead partial UDL 12.48 kN/m from 0 mm to 850 mm
	Screed & finishes - Dead full UDL 1 kN/m
	Partiiton - Dead full UDL 1.87 kN/m
	Ground floor - Imposed full UDL 5.85 kN/m
	Ground floor RC slab - Dead full UDL 18.72 kN/m
	Dead self weight of beam × 1

Load combinations

Load combination 1	Support A	Dead × 1.40
		Imposed × 1.60
		Dead × 1.40
		Imposed × 1.60
	Support B	Dead × 1.40
		Imposed × 1.60

Analysis results

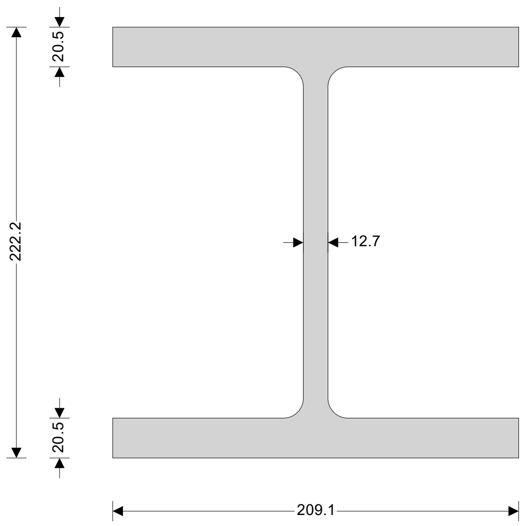
Maximum moment	M_{max} = **158.4** kNm	M_{min} = **0** kNm
Maximum shear	V_{max} = **189.8** kN	V_{min} = **-158.9** kN
Deflection	δ_{max} = **11.9** mm	δ_{min} = **0** mm
Maximum reaction at support A	R_{A_max} = **189.8** kN	R_{A_min} = **189.8** kN
Unfactored dead load reaction at support A	R_{A_Dead} = **112.1** kN	
Unfactored imposed load reaction at support A	$R_{A\_Imposed}$ = **20.5** kN	
Maximum reaction at support B	R_{B_max} = **158.9** kN	R_{B_min} = **158.9** kN
Unfactored dead load reaction at support B	R_{B_Dead} = **95.2** kN	
Unfactored imposed load reaction at support B	$R_{B\_Imposed}$ = **16** kN	

Section details

Section type	**UKC 203x203x86 (Tata Steel Advance)**
Steel grade	**S275**

From table 9: Design strength p_y

Thickness of element	max(T, t) = **20.5** mm
Design strength	p_y = **265** N/mm^2
Modulus of elasticity	E = **205000** N/mm^2



Lateral restraint

Span 1 has full lateral restraint

Effective length factors

Effective length factor in major axis	K_x = **1.00**
Effective length factor in minor axis	K_y = **1.00**
Effective length factor for lateral-torsional buckling	$K_{LT.A}$ = **1.00**

$K_{LT.B}$ = **1.00**

Classification of cross sections - Section 3.5

$\varepsilon = \sqrt{[275\ N/mm^2 / p_y]}$ = **1.02**

Internal compression parts - Table 11

Depth of section — d = **160.8** mm

$d / t = 12.4 \times \varepsilon <= 80 \times \varepsilon$ — Class 1 plastic

Outstand flanges - Table 11

Width of section — b = B / 2 = **104.6** mm

$b / T = 5.0 \times \varepsilon <= 9 \times \varepsilon$ — Class 1 plastic

Section is class 1 plastic

Shear capacity - Section 4.2.3

Design shear force — $F_v = max(abs(V_{max}), abs(V_{min}))$ = **189.8** kN

$d / t < 70 \times \varepsilon$

Web does not need to be checked for shear buckling

Shear area — $A_v = t \times D$ = **2822** mm^2

Design shear resistance — $P_v = 0.6 \times p_y \times A_v$ = **448.7** kN

PASS - Design shear resistance exceeds design shear force

Moment capacity - Section 4.2.5

Design bending moment — $M = max(abs(M_{s1\_max}), abs(M_{s1\_min}))$ = **158.4** kNm

Moment capacity low shear - cl.4.2.5.2 — $M_c = min(p_y \times S_{xx}, 1.2 \times p_y \times Z_{xx})$ = **258.8** kNm

PASS - Moment capacity exceeds design bending moment

Check vertical deflection - Section 2.5.2

Consider deflection due to dead and imposed loads

Limiting deflection — $\delta_{lim} = L_{s1} / 360$ = **12.222** mm

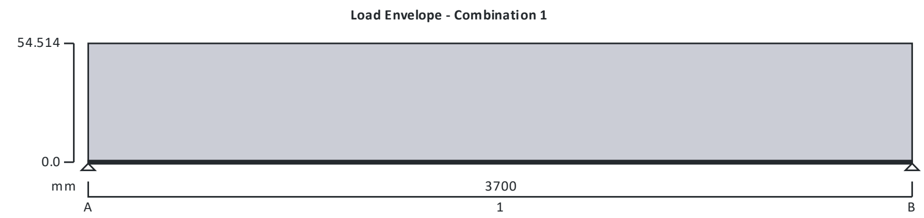
Maximum deflection span 1 — $\delta = max(abs(\delta_{max}), abs(\delta_{min}))$ = **11.883** mm

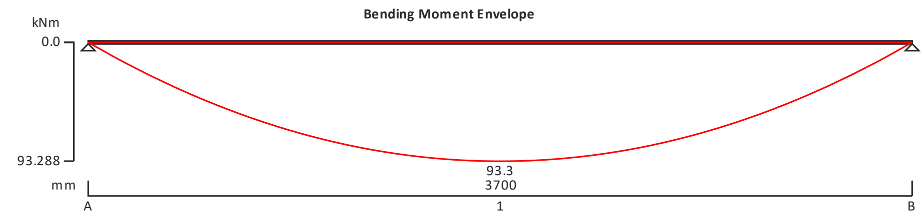
PASS - Maximum deflection does not exceed deflection limit

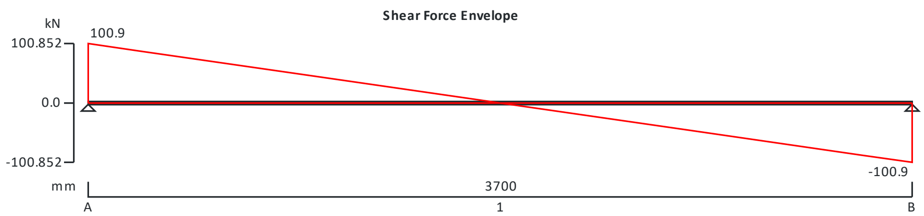
STEEL BEAM ANALYSIS & DESIGN (BS5950)

In accordance with BS5950-1:2000 incorporating Corrigendum No.1

TEDDS calculation version 3.0.07







Support conditions

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

Applied loading

Beam loads	Screed & finishes - Dead full UDL 4.8 kN/m
	Partiiton - Dead full UDL 2.4 kN/m
	Ground floor - Imposed full UDL 7.2 kN/m
	Ground floor RC slab - Dead full UDL 23 kN/m
	Dead self weight of beam × 1

Load combinations

Load combination 1	Support A	Dead × 1.40
		Imposed × 1.60
		Dead × 1.40
		Imposed × 1.60
	Support B	Dead × 1.40

Imposed × 1.60

Analysis results

Maximum moment	M_{max} = **93.3** kNm	M_{min} = **0** kNm
Maximum shear	V_{max} = **100.9** kN	V_{min} = **-100.9** kN
Deflection	δ_{max} = **8.6** mm	δ_{min} = **0** mm
Maximum reaction at support A	R_{A_max} = **100.9** kN	R_{A_min} = **100.9** kN
Unfactored dead load reaction at support A	R_{A_Dead} = **56.8** kN	
Unfactored imposed load reaction at support A	$R_{A\_Imposed}$ = **13.3** kN	
Maximum reaction at support B	R_{B_max} = **100.9** kN	R_{B_min} = **100.9** kN
Unfactored dead load reaction at support B	R_{B_Dead} = **56.8** kN	
Unfactored imposed load reaction at support B	$R_{B\_Imposed}$ = **13.3** kN	

Section details

Section type — **UKC 203x203x52 (Tata Steel Advance)**

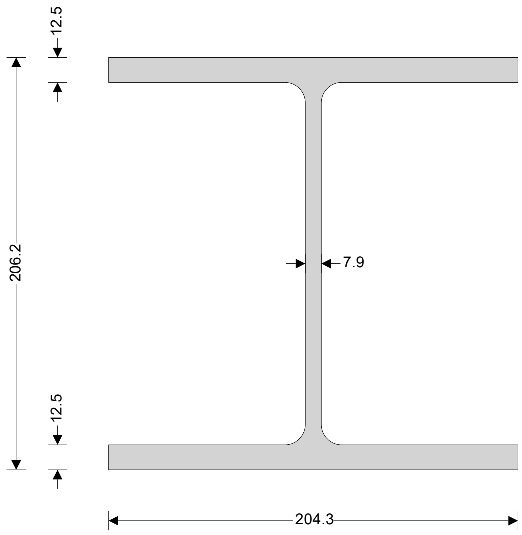
Steel grade — **S275**

From table 9: Design strength p_y

Thickness of element — max(T, t) = **12.5** mm

Design strength — p_y = **275** N/mm²

Modulus of elasticity — E = **205000** N/mm²



Lateral restraint

Span 1 has full lateral restraint

Effective length factors

Effective length factor in major axis — K_x = **1.00**

Effective length factor in minor axis — K_y = **1.00**

Effective length factor for lateral-torsional buckling — $K_{LT.A}$ = **1.00**

$K_{LT.B}$ = **1.00**

Classification of cross sections - Section 3.5

$\varepsilon = \sqrt{[275 \text{ N/mm}^2 / p_y]}$ = **1.00**

Internal compression parts - Table 11

Depth of section — d = **160.8** mm

$d / t = 20.4 \times \varepsilon <= 80 \times \varepsilon$ — Class 1 plastic

Outstand flanges - Table 11

Width of section $b = B / 2 =$ **102.2** mm

$b / T = 8.2 \times \varepsilon <= 9 \times \varepsilon$ Class 1 plastic

Section is class 1 plastic

Shear capacity - Section 4.2.3

Design shear force $F_v = max(abs(V_{max}), abs(V_{min})) =$ **100.9** kN

$d / t < 70 \times \varepsilon$

Web does not need to be checked for shear buckling

Shear area $A_v = t \times D =$ **1629** mm^2

Design shear resistance $P_v = 0.6 \times p_y \times A_v =$ **268.8** kN

PASS - Design shear resistance exceeds design shear force

Moment capacity - Section 4.2.5

Design bending moment $M = max(abs(M_{s1\_max}), abs(M_{s1\_min})) =$ **93.3** kNm

Moment capacity low shear - cl.4.2.5.2 $M_c = min(p_y \times S_{xx}, 1.2 \times p_y \times Z_{xx}) =$ **156** kNm

PASS - Moment capacity exceeds design bending moment

Check vertical deflection - Section 2.5.2

Consider deflection due to dead and imposed loads

Limiting deflection $\delta_{lim} = L_{s1} / 360 =$ **10.278** mm

Maximum deflection span 1 $\delta = max(abs(\delta_{max}), abs(\delta_{min})) =$ **8.581** mm

PASS - Maximum deflection does not exceed deflection limit

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Structural Calculation for Temporary support to Party wall

Walling timber check
Wind load check
Soldier check
Fixing check

Project 26 Amyand Park Road
Twickenham TW1 3HE

Prepared By Nathan Masil BEng, MSc, ICIOB

Date : 09.05.2024

Document: Calculation 23 227-03

Codes and standards Used:
BS5268
BSEN 1991-01-4 2005

NMN Partnership

PROJECT

6 Amyand Road TW1 3H

Twickenham

TW1

Sheet No.	1
Job No	23 227
Design by	NM
Date	09.05.24
Checked by	
Date	

Checker's comments

TITLE Wind load on façade

Wind Calc to BS EN 1991-1-4:2005+A1:2010

This spreadsheet performs calculations in accordance with the method described in Annex A of BS EN 1991-1-4:2005+A1:2010. Data from the IStructE is binlinearly interpolated to calculate coefficient values from the inputs given.

Location (first part of postcode)		TW1		London	
Height of structure above ground	z	6	m		
Altitude of site	A	2	m AOD		
Distance from open water		66.0	km		
Distance within town terrain		2.0	km	Set to zero if in country	
Reciprocal of annual probability of exceedence	1/p	50	years	(ie. 1 in 2	
Fundamental basic wind	$v_{b,map}$	22.5	m/s		Fig NA.1
Directional factor	c_{dir}	1			Tble NA.1
Seasonal factor	c_{season}	1			Tble NA.2
Altitude factor	c_{alt}	1.002			NA.2a
Probability factor	c_{prob}	1.000			4.2
				50	

Basic Wind Velocity $v_b = v_{b,map} \cdot c_{alt} \cdot c_{dir} \cdot c_{season} \cdot c_{prob}$

v_b 22.55 m/s

Reference basic velocity pressure $q_b = 0.613 \times v_b^2$

$q_b =$ 0.312 kN/m^2

Exposure factor	$c_e(z)$	2.03	Fig NA.7
Town correction factor	$c_{e,T}$	0.82	Fig NA.8

Peak velocity pressure $q_p(z) = c_e(z) \cdot q_b$ for country terrain — NA.3a

$q_p(z) = c_e(z) \cdot c_{e,T} \cdot q_b$ for town terrain — NA.3b

Work to Cpi and Cpe From here

$q_p(z) =$ 0.516 kN/m^2

Panel length	l	8	m	
Panel height	h	6	m	
	l/h	1.333		
	C_{pnet}	1.4		Tble 7.9

Wind pressure $W = q_p \times C_{pnet}$

W = <u>0.723</u> kN/m^2

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SHEET No 01 JOB No 23 227 Date 9 CALCS : NM

JOB 26 Amyand Park Road TW1 3HE OUTPUT

ELEMENT Temporary supports to Party wall

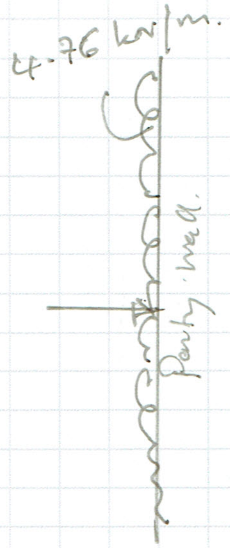
Party wall height = 6m.

wind load = 0.72 kN/m^2 = 0.72x6 = 4.32kN/m.

notional Horizontal load at 1st Floor (Prop).

$= \{(5.2 \times 3) + (1.16 \times 3/2 \cos 30)\} \frac{2.5}{100} = 0.44$ kN/M.

Total = (4.32 + 0.44) = 4.76kN/m.



Props are spaced at 2m c/c.

Load on Prop = 4.76 x 2 = 9.52 kN

Prop load = $\frac{9.52}{\cos 45}$ = 13.46 kN

Capacity of RMD Slimsoldier Prop = 100kN ok

Check bending moment of Soldier.

load = 0.72 x 2 = 1.44 kN/m.

Height of Soldier = 5m.

Prop location = 3.5m.

Cantilever Section = 1.5m.

Moment = $\left(1.44 \times \frac{1.5^2}{2}\right) + \left(1.44 \times \frac{3.5}{8}\right)$ = 3.28 kNm.

Joint moment capacity of RMD = 12 kNm ok

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ELEMENT **Temporary supports to Party wall**

Check walling timbers.

Spacing = 1m c/c.

load = 0.73 × 1 = 0.73 kN/m.

Span 2m

moment = $0.73 \times 2^2 / 8$ = 0.37 kNm.

Zxx required = $\frac{0.37 \times 10^6}{5.3}$ = $0.69 \times 10^3 mm^3$.

Timber to be used = 170×60 C16

Zxx = $\frac{bd^2}{6} = \frac{170 \times 60^2}{6} = 102 \times 10^3 mm^3 > 69 cm^3$ ok ✓

Check bolts in tension

Bolt = 12mm ø Studs

bolt tension capacity = 2.4 kN into brick work.

Max Tension = 0.73 kN < 2.4 kN ok ✓

Bolt to be resin anchord to brick work with min 8D = 96 mm. (Say 100mm)

Connection Slim Solder to Slim Solder = 4No M16

Timber to Timber = 2No M10

Timber/solder to wall M12 bolts at 1m c/c.

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SHEET No 03 JOB No 23 227 Date 9 CALCS : NM

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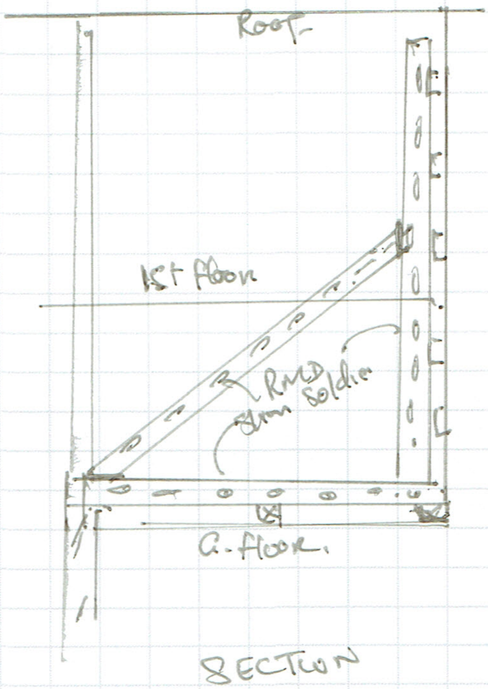
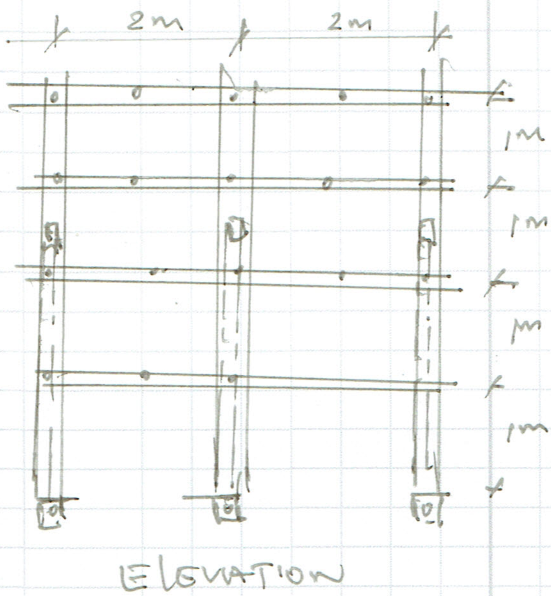
ELEMENT Temporary supports to Party wall

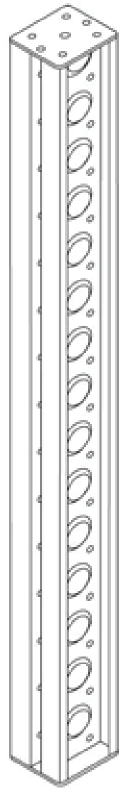




1.1.3. Section Properties

Soldier characteristics

Area: Gross	26.06 cm^2
Area: Nett	19.64 cm^2
I xx	1916 cm^4
I yy	658 cm^4
r xx	9.69 cm
r yy	5.70 cm
Z xx	161 cm^3
Z yy	61 cm^3
EI xx	4020 kNm^2
EI yy	300 kNm^2
GAxx	17350 kN
M max x	40 kNm
M max y	6.24 kNm
Max Joint Moment (4 M16 bolts)	12 kNm
Max Joint Moment (6 M16 bolts)	18 kNm
Max Joint Moment (stiffeners see 1.2.1. sheet 16)	20 kNm
Max Joint Tension (4 M16 bolts)	100 kN
Max Joint Tension (6 M16 bolts)	140 kN
Max Joint Tension (4 M16 bolts and stiffeners)	150 kN
Mean compressive yield stress	370 N/mm^2
Mean Self weight for Analysis	0.235 kN/m run*



* Self weight varies depending on makeup / length (see 1.1.1)

Effective area (Ae) for wind calculation purposes

Direction A	0.177 m^2/m
Direction B	0.130 m^2/m
Direction C	0.286 m^2/m