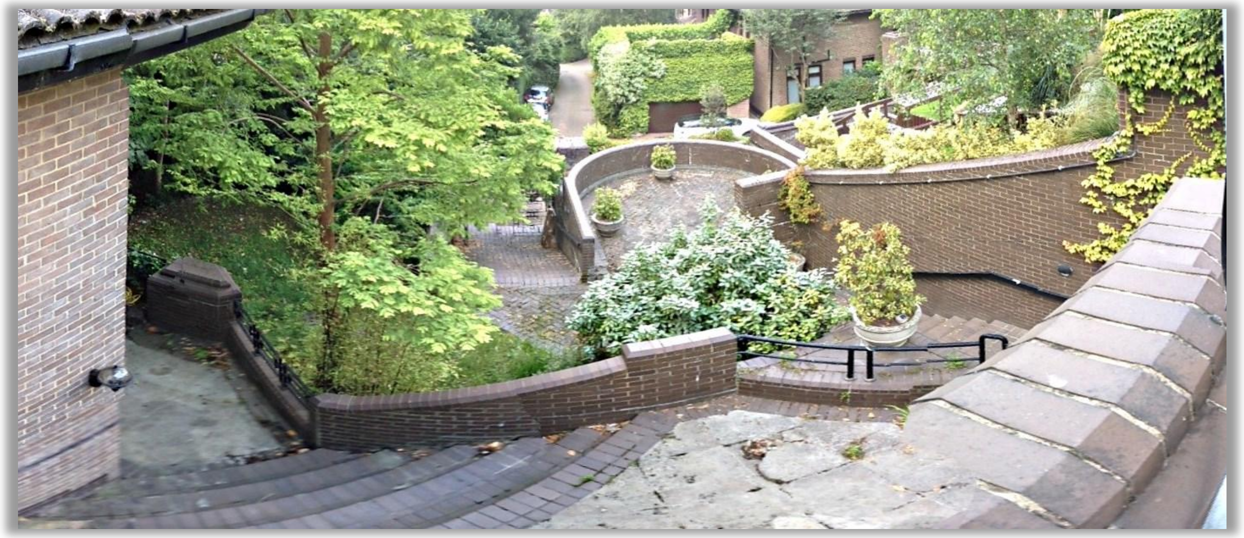
Residential

Commercial

Retail

Conservation



Project Number:	2014-207
Address:	5 Highfields Grove, Highgate, London SN6 6HN
Client:	Safran Holdings Ltd
Title:	Engineer's Construction Method Statement.
Date:	03rd October 2014.
Revision:	00
Prepared by:	Chartered Engineer (See end of report for details).

Elite Designers Ltd, 12 Princeton Court, 55 Felsham Road, London SW15 1AZ
Tel 020 8785 4499 Fax 020 8785 4999 E info@elitedesigners.co.uk

Prepared by: Marcin Dylowski 03/10/2014

Checked by: John Fitzpatrick 03/10/2014

PREAMBLE:

This report has been prepared by Elite Designers on the instruction of Yeates Design + Architecture, acting on behalf of the client and is for the sole use and benefit of the client.

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TERMS OF REFERENCE:

We were appointed by the client to prepare a Structural design Statement in support of a planning application for the refurbishment works at 5 Highfields Grove, Highgate, London, SN6 6HN.

Table of Contents

***1.0 ~ Introduction:* ... 3**

***2.0 ~ Party wall:* ... 3**

***3.0 ~ General descriptions of works:* ... 4**

***4.0 ~ Historic Background:* ... 4**

***5.0 ~ Ground Conditions / Geology:* ... 4**

5.1 Ground Bearing Pressure & Suitability: ... 5

5.2 Slope Stability: ... 5

***6.0 ~ Watercourses and Existing Trees:* ... 5**

6.1 Watercourses: ... 5

6.2 Existing Trees: ... 5

6.3 Flooding: ... 5

***7.0 ~ Description of Proposed Structure:* ... 5**

***8.0 ~ Construction Method:* ... 6**

8.1 Traditional underpin concept used for excavation: ... 6

8.2 Traditional underpin step by step: ... 6

8.3 Temporary Works: ... 6

***9.0 ~ Potential Ground Movements to Adjoining Properties:* ... 7**

***10.0 ~ Underground Structures & Existing services:* ... 7**

***11.0 ~ Drainage and Ground Water* ... 7**

***12.0 ~ Excavation of soil:* ... 7**

***13.0 ~ Waterproofing and Drainage:* ... 7**

***14.0 ~ Demolition, Recycling, Dust/Noise Control & Site Hoarding:* ... 7**

***15.0 ~ Conclusion:* ... 8**

***Appendix A: Drawings* ... 9**

***Appendix B: Preliminary Calculations* ... 10**

***Appendix C: Geotechnical & Services* ... 11**

***Appendix D: Damage category classification from CIRIA C580* ... 12**

***Appendix E: De-watering Method Statement* ... 12**

Construction Method Statement

Project information

Client: Safran Holding Ltd.
Address: 5 Highfields Grove, London SN6 6HN.
Nature of Works: Refurbishment of existing structure at above address to include construction of a full area basement as detailed in drawings provided by Yeates Design + Architecture.

1.0 ~ Introduction:

This report sets out the design philosophy for the proposed refurbishment works to 5 Highfield Grove. It should be read in conjunction with the detailed planning stage structural drawing and calculations attached in appendices which detail both the temporary and permanent design stages of the subterranean development. The aim of the method statement is to ensure safe and proper construction of the proposed works and ensure no adverse affects to existing or neighbouring structures.

While considering the most appropriate method of retaining the soil around the basement level in both the temporary & permanent conditions several potential methods were assessed. A feasibility study was undertaken to determine the most appropriate construction method. The first stage of the feasibility was to assess the Architect's proposal and to provide advice on alterations to the project where necessary from a structural point of view. The study allowed for an appraisal of the different potential construction methods available and suggestions were made as to the most suitable from both a time and cost point of view as well as their suitability for the given site conditions.

In this study the merits and shortcomings of sheet piling, bored piling and traditional underpinning techniques were examined. From the conclusion of this study it was felt that at this stage the most appropriate solution would be for a traditional underpinning technique to be employed. The construction sequence will deal with any issues of excavations under or adjacent to an existing property while minimising the potential losses of usable floor area. Given the preference to minimise any inconvenience to neighbouring properties and to maximise usable floor area of the proposed development, an underpinning solution would lend itself best to fulfilling all of the aforementioned, and the structural requirements of this development. For these reasons it was decided to detail the proposed solution shown in the appendix A drawings.

Following this a series of calculations were carried out (a summary of which is attached in the appendices) to allow for the production of planning stage drawings. These can be used to prepare preliminary budget costs to the project and can be submitted as a viable proposed engineering solution for planning; in addition they will allow the party wall process to be commenced and will form a solid base for engineering discussion for the proposed solution. These ensure the overall structural integrity of both the existing and neighbouring structures is retained throughout development. The stability of the building in all stages of construction and in the completed stage is provided for by careful sequencing of works to support the new building above the proposed basement works. The addition of a rigid interlocking set of reinforcing underpins which are to sit below the existing ground floor level will further stabilise the building in all directions. These boxes are created by the interaction between the proposed new concrete floors at ground floor and the proposed reinforced concrete retaining wall to the perimeter of the development. Above ground floor level it is proposed to introduce a steel frame which will work in tandem with load bearing masonry walls to replace the stability system of the existing structure. Due to this the proposed structure will remain stable. The current system employs a series of internal walls which provide stability in the transverse directions in addition to transferring vertical loads from above to strip footings below.

Due to the nature and makeup of the existing underlying soil types, slope instabilities are not of concern and loading patterns have been checked to ensure they will not occur. This is particularly evident with retaining wall solutions as the size and speed of the excavations under or adjacent to existing structure can be carefully controlled and propped as necessary to ensure no rotations of the wall segments, individually or as a group can occur. The proposed solution ensures no instabilities are created or allowed to occur within the soil mass during both the construction process and in the permanent state therefore any settlement to the surrounding area will be negligible, and therefore following the details laid down in the step by step installation method below, any adverse effects on neighbouring properties will be minimised/mitigated.

A visual inspection (by Elite Designers) of the existing building was carried out in order to determine the condition of the existing structure and its ability to deal with the proposed development. The existing structure is in a good state of repair in general. There are no signs of significant degradation or subsidence. The roof and floors appear to be of a traditional timber construction. The floor and roof structures are in turn supported on structural masonry which sits on concrete beams and piles as foundations. The existing foundations appear to be sufficient for supporting the existing structure and will work in tandem with the new proposal.

Responsibility for site safety and the implementation of applicable building practices and British Standards are the responsibility of the Main Contractor. This method statement is not exhaustive and assumes the Main Contractor has the competence and relevant experience to undertake building works of this nature.

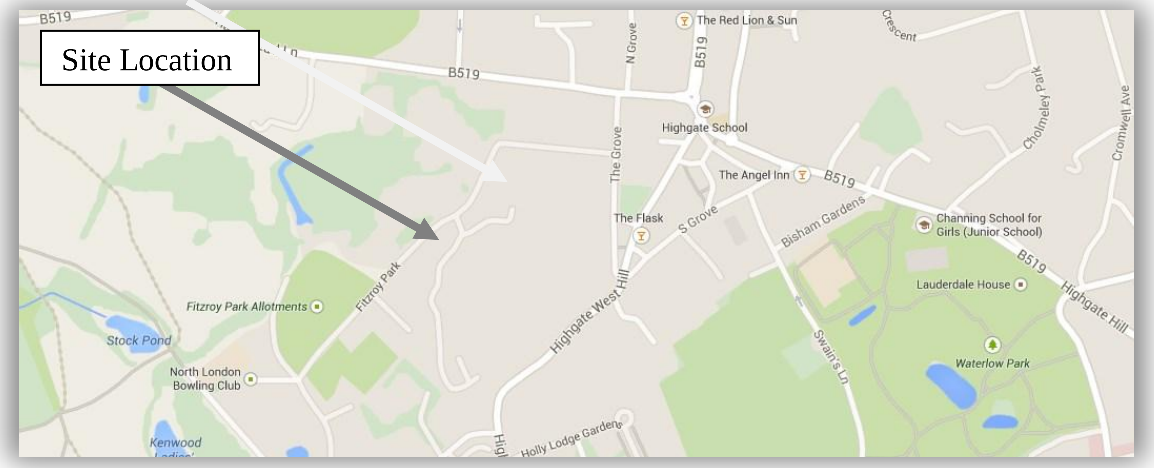
2.0 ~ Party wall:

No parts of these works will require a party wall agreement which will detail allowable construction tolerances and impacts on the neighboring properties (currently there are no foreseen affects to the integrity of surrounding structures).

3.0 ~ General descriptions of works:

The proposal is to construct a first basement area beneath the existing front footprint of the property, to a depth of approximately 2.7m, and a second basement area in addition to extending in part to the rear garden area, to a depth of approximately 2.5m. Following the construction of the basement level the existing building will be refurbished to provide a new layout to the existing dwelling. The alterations are detailed in the architectural drawings included with the application.

The property is a one storey four bedroom detached house and the surrounding properties appear to be of similar construction and age.

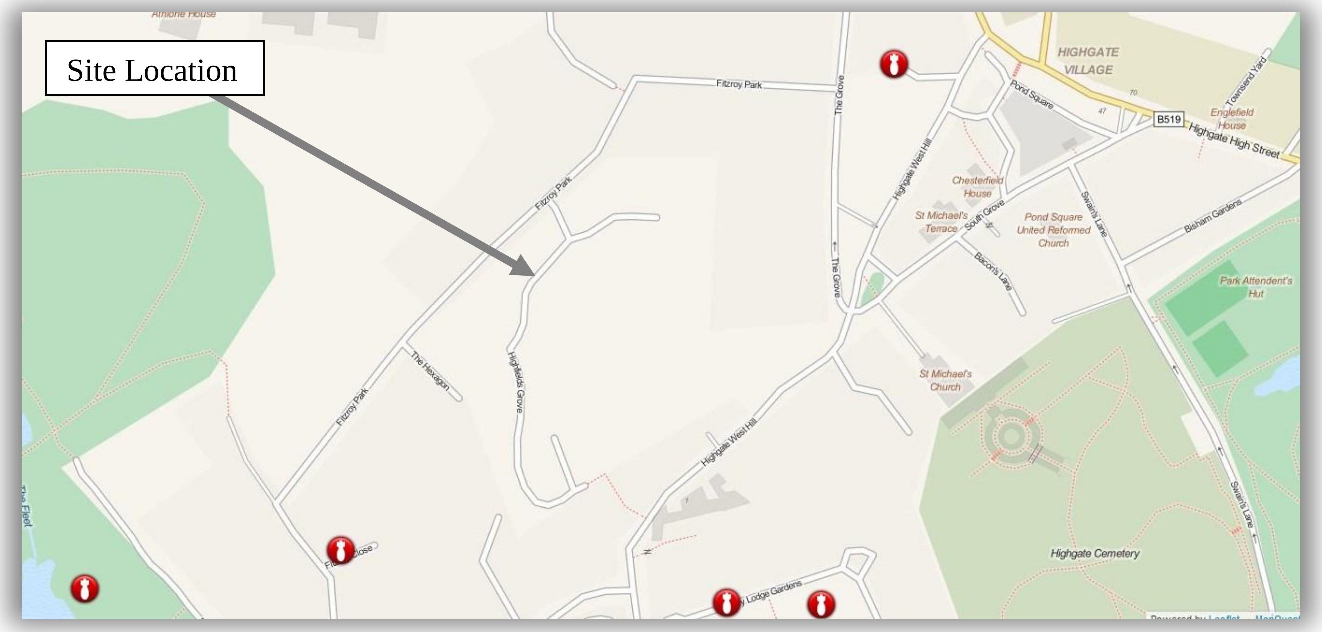


The site is situated on Highfields Grove towards the southern end of Fitzroy Park leading to The Grove, just off B519 (Hampstead Lane) in the London Borough of Camden. Highfields Grove is a residential street consisting of a varied mix of residential houses.

Access for materials and the removal of spoil will be via the front of the property. The exact method in which soil is to be removed from the site will be detailed in the traffic management plan.

4.0 ~ Historic Background:

The site appears to have escaped any bomb damage according to a review of the WW2 bomb maps. A reproduced extract map doesn't show any potential strike sites to the whole site.



5.0 ~ Ground Conditions / Geology:

Local knowledge of the area back up by review of British Geological Survey (attached in appendix C) suggest the underlying soil to be moderate thicknesses of made ground (to 1.0m) over the London clay (1.0m to 129m) which over lies thanet sands formations (129m to 147m) and chalk with flints (147m to 204m). The water table doesn't appear up to 10m borehole datum and should not interfere with the proposed construction of the basement. In line with design standards we need to allow for uplift within the design of the base floor slab. The uplift forces can be easily counteracted by the self weight of the basement structure itself in addition to the use of tension piles if necessary.

Given the depths at which the water table appears and the proposed depth to which it is planned to excavate the lower ground levels, it is not likely that the construction may project into the water level. However, given minimal intrusion during construction it is safe to conclude there will be no adverse affects by the development to the local hydrology of the area; however this will be discussed in more detail in the hydrological report which forms part of the application.

5.1 Ground Bearing Pressure & Suitability:

Gravels and clay, in particular the London clays are considered to stand up well for the proposed type of construction and can easily assume bearing pressures in excess of 150kN/m^2 which have been assumed in the design of the structure at this stage. We have constructed similar basements using the proposed typical basement retaining wall techniques.

5.2 Slope Stability:

The site is situated on a hill and proposed basement will be cut into the side of hill.

Slope stability has been considered and allowed for within the design of the retaining structure, therefore slope stability will not be of concern to the project going forward.

See Appendix C for full geotechnical report

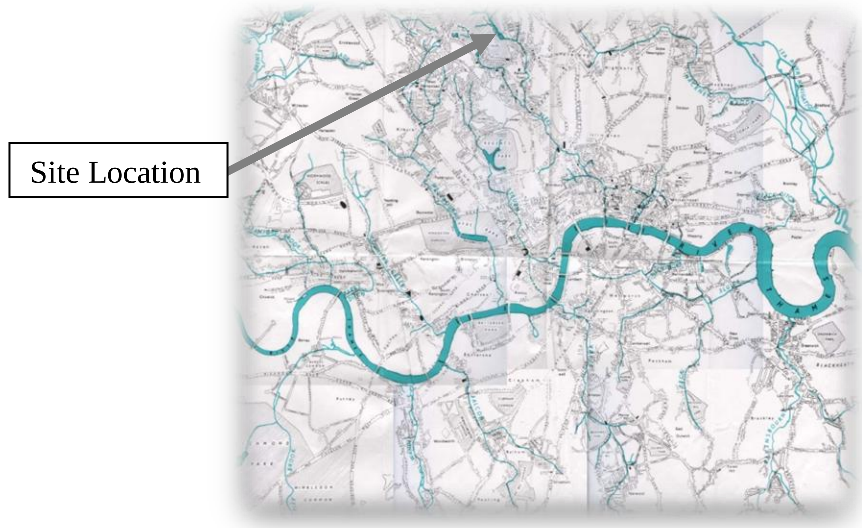
6.0 ~ Watercourses and Existing Trees:

6.1 Watercourses:

A desk top study and review of the “Lost Rivers of London” indicates that a sources of waterways known as “Hackney Brook” and “Sleek” run from approximately 400m and 1000m away to the south and into the River Thames.

Location of the site (hill situated) and the natural terrain shape indicate that neither of these is expected to have an effect on the proposed basement works.

The substratum is suspected as London Clay. These layers are permeable and some perched water could be expected on site. Seasonal variations in the ground water are to be expected and the contractor will be required to have considered suitable remediation measures during excavations and general basement works.



6.2 Existing Trees:

There are trees surrounding existing and proposed development. A detailed arboricultural report will deal with the impact on this in detail however, it is expected that construction will not significantly harm the roots as existing foundations will have acted as a root barrier. The contractor will provide in his method statement measures to be taken to protect the tree from both aerial and subterranean damage. The depth of influence of the tree in terms of soil shrinkage is not expected to be greater than 2.5m below ground and as the depth of the proposed foundations is significantly beyond this; there is no risk of the tree causing movements of the foundation.

6.3 Flooding:

A review on the environment agency website has shown that the site is not at risk of flooding from the river, sea and nearby reservoirs and it is understood that there has been no history of surcharging of local combined sewer systems in periods of heavy rainfall.

Due to the present hydrological status we would not expect the proposal to have an adverse affect on the ground water flow in the area and this is discussed further in the basement impact and hydrological assessment included in appendix C.

7.0 ~ Description of Proposed Structure:

The proposal is to construct two new basements partly under the footprint of the existing building using a traditional concrete underpin designed with adequate capacity to support the structure as per the current architect’s proposal.

A series of steel frames and beams will be installed at ground floor level and above, through the building to replace some of the current load bearing structural masonry walls allowing for excavation of the basement.

The following gives a proposed overall view of the installation sequence of the proposed development.

1. A part of the existing ground floor structure to be demolished to allow for the construction of the basement in line with the architect demolition drawings.
2. Temporary works to support the retained existing structure are to be implemented as necessary. The first level of traditional retaining walls can be installed in the standard hit and miss pattern.
3. Once cured, the main bulk excavation can take place in line with traffic management plans.
4. The basement will be constructed in line with the sequencing and structural engineer's drawings.
5. Upon completion of the basement works and casting of the basement floor slab, the remaining adjustments and construction of the above ground superstructure can be carried out.

See appendix A with feasibility stage drawings showing further details of the proposed structural solution.

It is recommended these works are carried out by a suitable experienced contractor familiar with this type of construction and the techniques required to produce the desired end result.

8.0 ~ Construction Method:

In addition to the detailed description of the underpinning sequence given below, reference should be made to the drawing attached in Appendix A which gives a visual representation of the proposed works.

8.1 Traditional underpin concept used for excavation:

The retaining wall will be formed in reinforced concrete approximately 350mm thick. They will be used to form the external walls of the basement level.

The walls will be constructed in short sections in a hit and miss pattern typical of this type of underpinning, approximately 1.0 to 1.4m wide and connected with steel dowels in the normal manner for this type of construction. The walls will need to remain back propped until the concrete has sufficiently cured.

When forming each cantilevering L-shaped section of wall, an access trench is dug down to the formation level of the base slab. Piles are installed and remaining soil removed. Piles are cut and pile caps are poured. Reinforcement is fixed and the base of the underpin is poured. Following this the wall reinforcement is fixed and the wall shuttered and poured. By using hit and miss sequencing it is possible to work on more than one pin at a time safely up to a maximum of four pins around the perimeter of the building.

8.2 Traditional underpin step by step:

i. Mark out datum line to determine various surface heights.
ii. Following sequencing guidance from engineers drawings mark out proposed digging area for current sequence.
iii. Begin digging within marked area to depth of 1m, using laser meter to determine appropriate depths.
iv. Install sheeting against the retained earth face, planking and strutting segment made up of two sheets of 18mm plywood across all side of pit, with timber struts of 125mm x 50mm at 500mm centres, reinforced with mini-acrow steel props set at 1m centres as per details on drawings.
v. Install 1m high timber railing guard around pit.
vi. If site manager deems it appropriate, install timber guard to prevent loose material from falling onto workers whilst digging.
vii. Continue digging for further 1m, and then install further planking and strutting segment to same specifications as above.
viii. From 2m depths, continue digging in 600mm segments with planking and strutting segment to same specifications as above.
ix. Water table should be lower than this level of excavation but if necessary it should be lowered below the level of basement excavation. This is to be achieved through the installation of appropriate submersible pumps to remove water locally from the area being excavated. Should ingress become more than a minor flow, stop digging and back fill immediately. Seek advice from engineer.
x. In sequences, set between two other sequences (or adjacent to each other) already completed, install dowel bars 1100mm long and 12mm diameter at 200mm centres as proposed by engineers in completed underpins either side.
xi. Install shuttering.
xii. Pour concrete mix (engineer's specification) into shuttered mould.
xiii. Underpin will connect into basement floor slab.
xiv. After 48 hours, remove timber shuttering.
xv. Begin next sequence as directed in accordance with direction of engineers.
xvi. Continue above steps until all the wall sequences have been completed.
xvii. Once the shuttering has been removed from the last sequence and piles have been installed, the central mass of soil can start to be removed in sections to allow for installation of temporary propping or the floor slab with pile caps.

8.3 Temporary Works:

No Structural works will commence without a detailed temporary works design, drawing and calculation package in place including all necessary method statements.

Structural drawings give proposed acceptable details for the excavations and a proposed sequence for the works. By following this sequence, the extent of temporary supporting works can be minimised.

The depth of construction is approximately 3.5m below the existing garden level and if the basement is constructed as per the suggested method on drawings, then minimal temporary works should be required. This comes about because the underpinning in the permanent case is propped by the new structural floor. Therefore they will not develop any slope instabilities in any of the neighboring properties if constructed as described. However the contractor is advised to have some sheeting available to deal with any unexpected pockets of poor ground.

9.0 ~ Potential Ground Movements to Adjoining Properties:

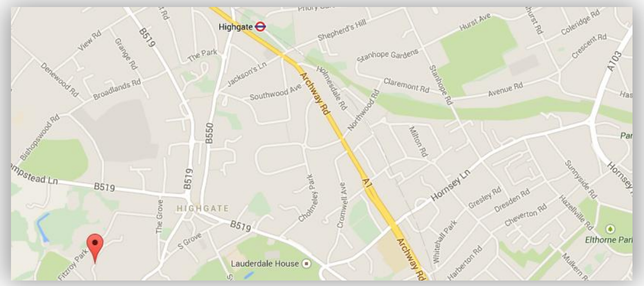
Anticipated movements are expected to be minimal and suppressed by the stiffness of the above structure and those adjoining.
The category of movement expected for this element of works would be a category 0-1 of the building damage classification table based on CIRIA C580 guidance (see appendix D).
A suitable experienced contractor familiar with propping techniques and sequential operations should be appointed. The designer has considered the risk to adjoining properties and the proposed foundation system offer an inherently strong foundation to load bearing walls.
Monitoring of the surrounding building will be carried out during the works to assess possible movements and the findings will be reported to the adjoining surveyors periodically if necessary.

10.0 ~ Underground Structures & Existing services:

A desk top investigation has been carried out in order to establish the positions of any underground utilities, main drainage or infrastructure to ensure no impact on these. Investigations show the positions of services however; the contractor should carry out works under the assumption that there may be additional unknown service locations, taking all necessary precautions. It is the contractor's responsibility to coordinate any alterations of these incoming services with the appropriate service suppliers. All appropriate measures to be taken for any required alterations.

A drainage report has been carried out, all other services i.e. gas and electricity are common to the site address only.

A preliminary search shows that the closest underground station to the development is Highgate Northern Line), however as the distance is in excess of 500m away the proposed works will therefore not have any influence on these structures. It will therefore not be necessary to advise London underground asset protection department to check alignments and agreed works will not affect any existing tunnels or access shafts. No other underground structures, tunnels or vaults are expected in the vicinity of the proposed works.



11.0 ~ Drainage and Ground Water

Where possible, the existing drainage and sewage connections will be maintained. It will be necessary to carry out some works to the drainage locally within the curtilage of the development to allow for the new requirements on both surface and foul water drainage of the new layouts but these will not impact in any way on the neighboring properties. A sustainable, environmentally friendly and responsible approach will be taken in the design of the surface water for the development. The new drainage layout will be design in accordance with best practice and the SUDS framework directive.

The proposed works will not alter the current state of the property, which will remain as a single family residence. Therefore, the expected volume of both foul and surface water is expected to remain at similar levels for a property of this size and so will not have a negative impact. The borehole log indicates that ground water levels are greater than 6m below the pavement level, the planned excavation being approximately 3m is considerably above the ground water level and it is fair to conclude that the proposed basement will not affect current ground water levels or flows.

12.0 ~ Excavation of soil:

The soil will be excavated and removed using small excavators / conveyor belts up to ground level and transferred to normal 7m skips as per the traffic management plan. Public rights of way will be maintained where necessary and the footpaths and street adjacent to the site will be cleaned each evening. The frequency of vehicle movements will be confirmed by the chosen contractor and approved by the council before works commence.

13.0 ~ Waterproofing and Drainage:

Concrete elements where practically possible will be design to BS8007 in order to minimise water ingress. In addition to this a drainage system (cavity type or other) is to be installed in accordance with BS8102 to provide a fully water proof envelope in the event of any water ingress through the concrete.

Sump pumps and drainage will be required to remove any water ingress through the concrete structure and these will need to be designed by a specialist drainage engineer.

14.0 ~ Demolition, Recycling, Dust/Noise Control & Site Hoarding:

Demolition work is to take place within the hoarded confines of the site. Materials such as stock bricks, re-usable timbers; steel beams etc are to be recycled where possible. To minimize dust and dirt from demolition, it is recommended the following measures shall be implemented:

- Any debris or dust / dirt falling on the street and public highway will be cleared as it occurs by designated cleaners and washed down fully every night.
- Demolished materials are to be removed to a skip placed in front of the site which will be emptied regularly as required.

Building work which can be heard at the boundary of the site should not be carried out on Sundays or bank holidays. It is suggested the contractor allow for this when programming the works

15.0 ~ Conclusion:

We do not anticipate any damage to adjoining structures as a consequence of these works if carried out in the approved manner as described above by competent contractors. There should not be any impact on the integrity of the adjoining structures. Due to the soil conditions, dense gravels and stiff clay give a safe bearing pressure in excess of 150 Kn/m^2; we do not anticipate any significant settlement following the excavation. There will be no slope stability issues as a result of the development. The proposed structure is a traditional underpin solution, this form of construction will provide adequate support to the adjoining gardens and structures and we anticipate no adverse effects on the surrounding properties.

There are a number of small trees surrounding the development but consideration of the protection of the root zone has been undertaken and we consider that all these trees of worth will remain unaffected by the works.

Excessive temporary works are not deemed necessary for the proposed basement excavation as the structure has been developed to allow for all loading which may occur during both the construction phases and the permanent load cases.

In the permanent case a steel frame and load bearing elements will be designed to allow for all possible loading scenarios but the contractor will need to design a suitable set of temporary works for the installation along with methods statements which the engineer should approve.

It is my opinion that the proposed works can be carried out within a safe and cost effective manner by a suitable contractor.

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Marcin Dylowski
Structural Engineer
Elite Designers Ltd.

____

John Fitzpatrick ***B (Struct) Eng, CEng, M.I.E.I., M.I.C.E***
Senior Chartered Structural Engineer
Elite Designers Ltd.

Appendix A: Drawings

GENERAL NOTES:

- TO BE READ IN CONJUNCTION WITH ALL RELEVANT METHOD STATEMENTS & SPECIFICATIONS ISSUED FOR THE JOB.
- ALL DIMENSIONS ARE IN mm U.N.O.
- NO DIMENSIONS TO BE SCALED FROM THESE DRAWINGS, WORK TO FIGURED DIMENSIONS ONLY. FOR DETAILS OF SETTING OUT REFER TO SETTING OUT DRAWINGS.
- ALL ELEVATIONS ON PLANS ARE WITH RESPECT TO ARBITRARY DATUM TO BE ESTABLISHED ON SITE.
- ALL DIMENSIONS TO BE CHECKED ON SITE BY THE CONTRACTOR AND ANY DISCREPANCIES BROUGHT TO THE ENGINEER'S ATTENTION.
- REASONABLE OPPORTUNITY TO BE GIVEN TO ELITE DESIGNER LTD TO INSPECT ALL STRUCTURAL WORKS BEFORE COVERING UP.
- ALL CONCRETE WORKS TO COMPLY WITH THE NATIONAL STRUCTURAL CONCRETE SPECIFICATION FOR BUILDING CONSTRUCTION.
- ALL STEELWORK TO BE IN ACCORDANCE WITH THE NATIONAL STRUCTURAL STEELWORK SPECIFICATION (N.S.S.S.)
- ALL WORKS TO BE CARRIED OUT IN ACCORDANCE WITH THE CURRENT EDITION OF THE NATIONAL BUILDING REGULATIONS & B.S STANDARDS.

FOUNDATIONS:

- ALL CONCRETE SHALL BE GRADE C40N20, UNLESS NOTED OTHERWISE.
- ALL EXCAVATIONS FOR FOUNDATIONS SHALL BE INSPECTED AND APPROVED BY ELITE DESIGNERS LTD PRIOR TO CONCRETING.
- EXISTING FOUNDATIONS SHALL NOT BE UNDERMINED OR INTERFERED WITH IN ANY MANNER AND EVERY PRECAUTION SHALL BE TAKEN TO ENSURE THAT THE FORMATION LEVEL REMAINS DRY, FORMATION LEVELS AS SHOWN MAY VARY DEPENDING ON CONDITIONS ENCOUNTERED ON SITE.
- CONTRACTOR TO REPORT IMMEDIATELY TO THE ENGINEER IF IT IS DISCOVERED THAT THE SUBSOIL CONDITION IS POORER THAN THE EXPECTED SOIL CONDITION ESTIMATED BASED ON SOIL INVESTIGATION REPORT.
- ALL MATERIALS & WORKMANSHIP TO CONFORM TO BS 8002.
- WHERE DEEMED NECESSARY BY THE ENGINEER ALL FOUNDATIONS TO BE TAKEN DOWN WITH MASS CONCRETE (GRADE C20) AS FAR AS GROUND WITH ADEQUATE BEARING CAPACITY.
- ALL WALLS TO BE CENTERED ON FOUNDATIONS, RISING WALLS TO GROUND FLOOR TO BE CONSTRUCTED IN SOLID CONCRETE BLOCK WORK UP TO DPC LEVEL, THEREAFTER, AS SHOWN ON PLAN.

PILES:

- PILES SHALL BE DESIGNED BY AN APPROVED PILE DESIGNER IN ACCORDANCE WITH BS 8110 & THE I.C.E SPECIFICATION FOR PILING AND EMBEDDED WALLS AND FORWARDED TO ELITE DESIGNERS LTD PRIOR TO WORK COMMENCING.
- PILES SHALL BE DESIGN AS UN-PROPPED DURING CONSTRUCTION.
- PILES TO HAVE LOAD CAPACITY AS INSTRUCTED SEPERATELY BY ELITE DESIGNERS LTD. CONTRACTOR SHALL ENSURE THAT ALL ADJACENT LIVE SERVICES HAVE BEEN LOCATED.
- PILE TESTING, INTEGRITY TESTING 100%, DYNAMIC TEST 5 NO PILES IN ACCORDANCE WITH I.C.E. SPECIFICATION.
- PILE LOADING TO BE CONFIRMED. FINAL PILE LAYOUT TO BE DETERMINED SUBJECT TO FULL SITE INVESTIGATION.

REINFORCEMENT ESTIMATES:

- RC PILE CAPS ALLOW 115 Kg/m³.
  (60% T25, 25% T16 & 15% Links)
- RC PAD FOOTINGS ALLOW 90 Kg/m³.
  (95% T 16 & 5% Links)
- RC GROUND BEAMS ALLOW 230 Kg/m³.
  (70% T25, 10% T12 & 20% Links)
- RC GROUND BEARING SLABS ALLOW 85 Kg/m³.
  (100% Mesh)
- RC STAIRS & LANDINGS 135 Kg/m³.
  (60% T16, 35% T12 & 10 % Links)
- RC WALLS 65Kg/m³.
  (80% T16 & 20% T12)

REINFORCED CONCRETE NOTES:

- CONCRETE ABOVE SUB STRUCTURE SHALL BE GRADE C35N20 UNLESS NOTED OTHERWISE.
- COVER TO REINFORCEMENT SHALL BE 35MM UNLESS NOTED OTHERWISE.
- ALL REINFORCEMENT SHALL BE HIGH TENSILE DEFORMED TYPE 2 BARS WITH A HIGH YIELD STRENGTH OF 500 N/mm² UNLESS NOTED OTHERWISE.
- ALL STEEL MESH SHALL HAVE A HIGH YIELD TENSILE STRENGTH OF 485 N/mm² UNLESS NOTED OTHERWISE.
- MINIMUM OF 400mm LAPS SHALL BE PROVIDED IN MESH REINFORCEMENT.
- ALL REINFORCEMENT TO BE INSPECTED AND APPROVED BY ELITE DESIGNERS LTD PRIOR TO POURING OF CONCRETE.
- CONCRETING WORKS SHALL NOT BE CARRIED OUT IF THE AIR TEMPERATURE IS LOWER THAN 2 DEGREES OR IF FROST IS EXPECTED.
- BEFORE PLACING STRUCTURAL CONCRETE ON HARDCORE OR OTHER ABSORBENT STRATA, LAY DAMP PROOF MEMBRANE ON SAND BLINDING. ENSURE MINIMUM LAPS AND SEAL TO MANUFACTURE'S REQUIREMENTS. ADEQUATELY PROTECT MEMBRANE FROM PUNCTURING, AND CAREFULLY REPAIR ANY PUNCTURES WHICH DO OCCUR.
- UNLESS AN ARCHITECTURAL SCREED IS TO BE PROVIDED, ALL FLOOR SLABS TO RECEIVE POWER TROWELED FINISH, APPLYING SUFFICIENT PRESSURE TO CLOSE THE SURFACE, TO GIVE A UNIFORM SMOOTH FINISH FREE FROM TROWEL MARKS AND OTHER BLEMISHES. AFTER CURING, APPLY AN APPROVED RESIN SEALER TO CONCRETE WEARING SURFACE FLOORS IN ACCORDANCE WITH MANUFACTURE'S RECOMMENDATIONS. ALL SLABS SHALL BE WET CURED FOR AT LEAST 7 DAYS AFTER CASTING, SUBMIT CURING DETAILS FOR REVIEW AND ACCEPTANCE.
- MAXIMUM POUR SIZE TO BE 15m IN LENGTH AND 200m² IN AREA, THE RATIO OF THE SIDES IS NOT TO EXCEED 1:1.5. JOINTS ARE TO BE ARRANGED SO AS TO MINIMISE THE OCCURRENCE OF SHRINKAGE CRACKS.
- SUDDEN IRREGULARITIES IN CONCRETE FINISH ARE NOT PERMITTED. THE VARIATION IN SURFACE FINISH IS TO BE NOT MORE THAN 5mm UNDER A 3m STRAIGHTEDGE AND/OR 2mm UNDER A 1m STRAIGHTEDGE.
- ALL PRECAST CONCRETE TO BE DESIGNED AND DETAILED BY PRECAST SUPPLIER. DESIGN CALCULATIONS AND DRAWINGS TO BE SUBMITTED FOR REVIEW AND ACCEPTANCE BY ENGINEER PRIOR TO FABRICATION.

STEELWORK:

- ALL STEELWORK TO BE AT LEAST GRADE S275 TO B.S. EN 10025 U.N.O
- ALL INTERNAL BOLTS TO BE GRADE 8.8 TO B.S. 3692 GALVANISED TO B.S. 729 OR B.S.4921-(43 MICRONS). ALL EXTERNAL BOLTS, NUTS AND WASHERS TO BE STAINLESS STEEL WITH EPDM WASHERS.
- CORROSION PROTECTION: ALL INTERNAL STEEL WORK TO BE SHOT BLASTED TO SWEDISH STANDARD SA2.5 AND PAINTED WITH TWO COATS OF ZINC PHOSPHATE PRIMER TO A MINIMUM DRY FILM THICKNESS OF 75 MICRONS. ALL EXTERNAL STEELWORK TO BE SHOT BLASTED TO SWEDISH STANDARD SA2.5 AND HOT DIP GALVANISED TO 140 MICRONS.
- ALL STEELWORK BELOW GROUND LEVEL SHALL BE ENCASED IN CONCRETE.
- ALL WELDS SHALL BE 6mm FULL PROFILE FILLET WELDS UNLESS NOTED OTHERWISE.
- STEEL BEAMS SUPPORTED BY MASONRY WALLS SHOULD BEAR ONTO CONCRETE PAD STONE AS SHOWN.
- FIREPROOFING TO CONSIST OF INTUMESCENT PAINT APPLIED BY SPECIALISED CONTRACTOR TO BS 476 WITH 60MIN FIRE RATING AND TO BE COMPATABLE WITH CORRISION PROTECTION OF STEEL.

LINTELS:

- ALL LINTELS TO INTERNAL BLOCK WORK TO HAVE MINIMUM END BEARING OF 200mm.
- ALL LINTELS TO BE PRECAST CONCRETE LINTELS OR STAINLESS STEEL TYPE WITH CAPACITIES AS FOLLOWS:
  - FOR CLEAR SPANS UP TO 1200mm USE 100 X 65mm dp LINTELS WITH CAPACITY OF 1.2 Kn/m, 140 X 65mm dp LINTELS WITH CAPACITY OF 1.7 Kn/m & 215 X 65mm dp LINTELS WITH CAPACITY OF 2.5 Kn/m.
  - FOR CLEAR SPANS UP TO 1800mm USE 100 X 140mm dp LINTELS WITH CAPACITY OF 1.2 Kn/m, 140 X 140mm dp LINTELS WITH CAPACITY OF 1.7 Kn/m & 215 X 140mm dp LINTELS WITH CAPACITY OF 2.5 Kn/m.

MASONRY:

- ALL BLOCK WORK WALLS TO BE AGGREGATE CONCRETE BLOCKS OF MINIMUM 7N/MM² COMPRESSIVE STRENGTH, UNLESS NOTED OTHERWISE.
- ALL MORTAR TO BE TYPE (iii) TO BS5628-1 : 2005.
- WALL TIES TO BE EITHER POLYPROPYLENE OR STAINLESS STEEL VERTICAL TWIST TYPE TO BS 845, WITH MINIMUM EMBEDDMENT OF 50mm IN EACH LEAF. TIES TO BE SPACED @ 450mm CENTRES VERTICALLY AND 750mm CENTRES HORIZONTALLY. ADDITIONAL TIES TO BE PROVIDED WITHIN 225mm OF ALL OPENINGS AND MOVEMENT JOINTS @ 225mm CENTRES VERTICALLY.
- ALL BLOCK WORK WALLS TO BE TIED TO STEELWORK STANCHIONS @ 225mm CENTRES VERTICALLY USING PROPRIETARY STAINLESS STEEL TIES SECURED TO COLUMNS SUCH AS ANCON BRICLOK OR SIMILAR APPROVED.
- MOVEMENT JOINTS TO BE PROVIDED IN MASONRY WALLS AS INDICATED ON PLAN OR AS FOLLOWS: BRICK WORK = 12M CENTRES, BLOCK WORK = 8M CENTRES.
- ALL DPC's TO BE LDPE DPC TO BS 6515.
- FACING BRICKS FROM THE IBSTOCK BRICK RANGE WITH RECESSED POINTING. COLOR TO CLIENTS SPECIFICATION.

LINTELS:

ALTERATIONS ARE REQUIRED TO BOTH SOLID BRICK WALLS AND CAVITY WALLS. THE FOLLOWING LINTELS ARE TO BE USED FOR DIFFERENT WIDTH AND THICKNESS OF WALLS

SOLID WALLS

4" THICK WALLS UP TO 1M WIDE OPENING PRECAST LINTEL	65X100.
4" THICK WALLS UP TO 1.5M WIDE OPENING PRECAST LINTEL	100X100.
4" THICK WALLS UP TO 2.5M WIDE OPENING PRECAST LINTEL	215X100.
9" THICK WALLS UP TO 1M WIDE OPENING PRECAST LINTEL	2NO. 65X100.
9" THICK WALLS UP TO 1.5M WIDE OPENING PRECAST LINTEL	2NO.100X100.
9" THICK WALLS UP TO 2.5M WIDE OPENING PRECAST LINTEL	2NO.215X100.
13" THICK WALLS UP TO 1M WIDE OPENING PRECAST LINTEL	3NO. 65X100.
13" THICK WALLS UP TO 1.5M WIDE OPENING PRECAST LINTEL	3NO.100X100.
13" THICK WALLS UP TO 2.5M WIDE OPENING PRECAST LINTEL	3NO.215X100.

CAVITY WALLS

300MM THICK WALLS UP TO 1M WIDE OPENING CATNIC LINTEL CG	90/100.
300MM THICK WALLS UP TO 1.5M WIDE OPENING CATNIC LINTELCG	90/100.
300MM THICK WALLS UP TO 2.5M WIDE OPENING CATNIC LINTELCH	90/100.

LINTELS ARE NOT TO BE USED OVER OPENING WHICH ARE CREATED UNDER POINT LOADS FROM EXISTING OR PROPOSED BEAMS. ONLY TIMBER FLOOR CAN BE SUPPORTED WITH THE ZONE OF INFLUENCE BY THESE LINTELS. ENGINEER IS TO BE INFORMED FOR LINTEL DESIGN IN THESE CASES.

Notes

1. This drawing is to be read in conjunction with all relevant architects, engineers & specialist sub-contractors drawings and the specification.

2. Any discrepancies between the site conditions and these drawings to be reported to Elite Designers. Dimensions must not be scaled and should be checked on site.

3. All dimensions are in millimetres, levels are in metres a.o.d. (above ordnance datum).

4. Foundations have been designed on a safe increase in bearing pressure of 150kN/m² bearing 200mm into sandy gravel strata.

5. All new steelwork to be grade S275 and be supplied to site blast cleaned to swedish standard SA2½ painted with high build zinc phosphate alkyd primer to 80 microns after fabrication. Any mechanical damage to coating to be touched up on site in accordance with the specification.

6. All new steel beams to have a minimum of 100mm bearing either end.

7. Lengths of all members are to be verified on site by the Contractor.

8. Catnic type lintels to have a minimum bearing of 150mm either end.

9. All temporary works to ensure the structural stability of all elements in the temporary state during construction are to be the responsibility of the contractor.

10. Cover to reinforcement to be 25mm to all bars unless noted otherwise.

11. Checking the location of the existing services in relation to the elements of the new construction works is the responsibility of the principal contractor. Any discrepancy between the existing services and the new construction works should be reported to Elite Designers before the commencement of the works.

12. The principal contractor is to provide all necessary flexible sleeves or lintels where drainage pipes pass through walls or foundations.

13. The principal contractor is to ensure that at all times the excavations shall remain free from standing water.

14. Movement joints to be positioned @ 6m c/c in blockwork and @ 12m c/c in brickwork.

15. Movement joints to be 15mm hydrocell or similar joint filler with a 15x15mm two part polysulphate sealant. (colour and fire resistance of sealant to be advised by architect).

16. All load bearing blockwork below DPC to be 7N/mm² dense concrete block.

17. Provide Ancon ST1 wall ties in accordance with DD140 @ 450 c/c vertically and @ 900 c/c horizontally, staggered u.n.o.

18. All bolts to be Grade 8.8 M20 unless noted otherwise.

19. All insulation details have been produced to comply with relevant regulations where possible. However, the responsibility for checking the compliance and execution of insulation details lies with the main contractor.

20. Floor joists spanning in excess of 2.5m should be strutted by one or more rows of solid or herrringbone strutting as follows:

Joists <2.5m - None required
Joists 2.5 - 4.5m - One row required
Joists >4.5m - Two rows required

21. All beam end reactions shown are unfactored unless noted otherwise.

PLANNING

Rev	Description	Chked	Date

Project
5 Highfield Grove
Highgate

Client

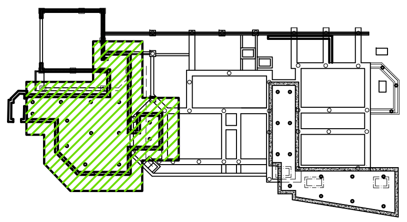
Drawing Title

General Notes

Drawn	Checked	Drawing No:	
MD	NJR	2014-207-00	
Date	Job No:	Scale	Rev
06/10/2014	2014-207	1:50 at A1 1:100 at A3	0

ELITE Designers
12 Princeton Court, 55 Felsham Road, London SW15 1AZ
Tel: 020 8785 4499 Fax: 020 8785 4999

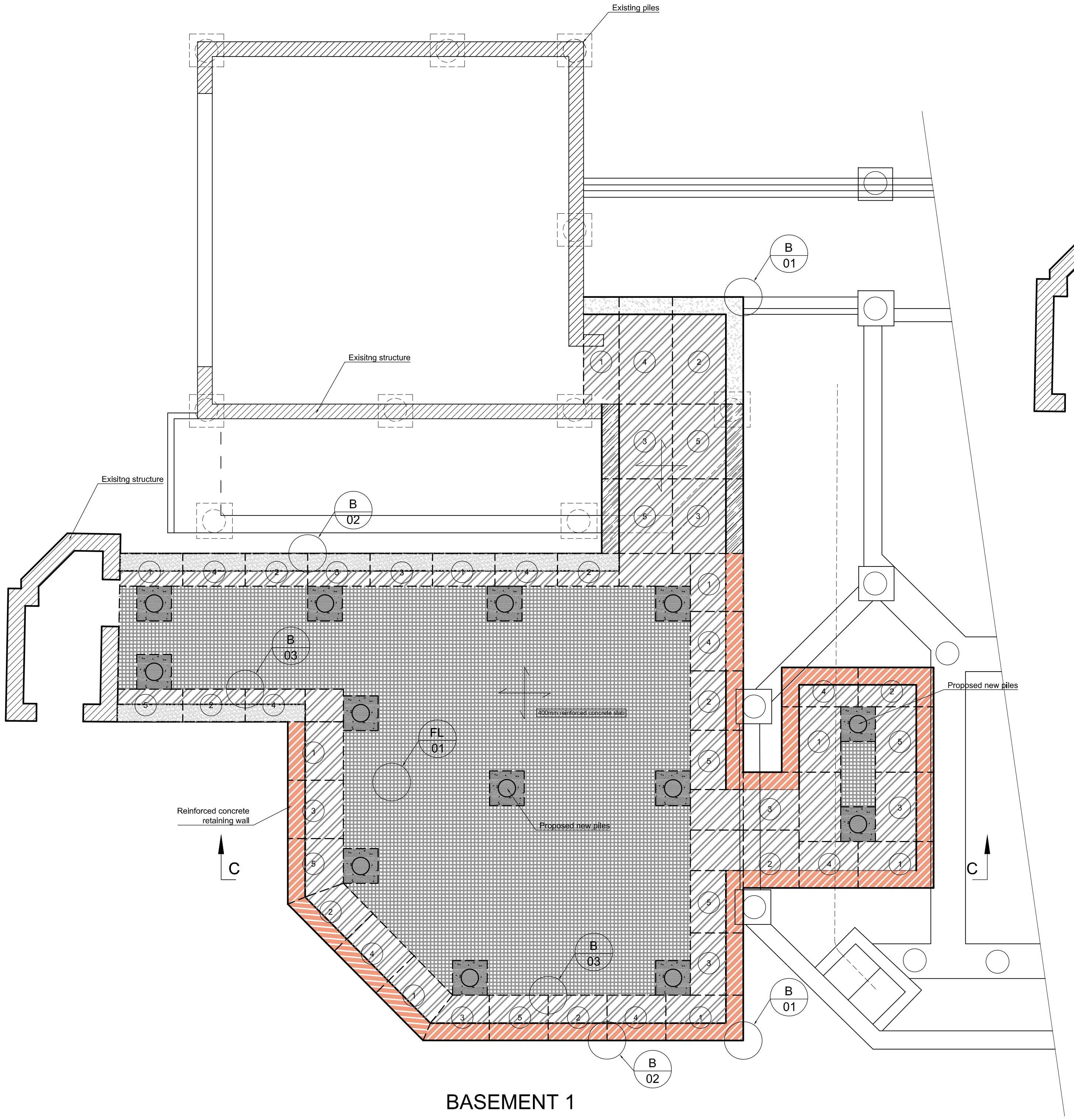
Key Section



Area of interest

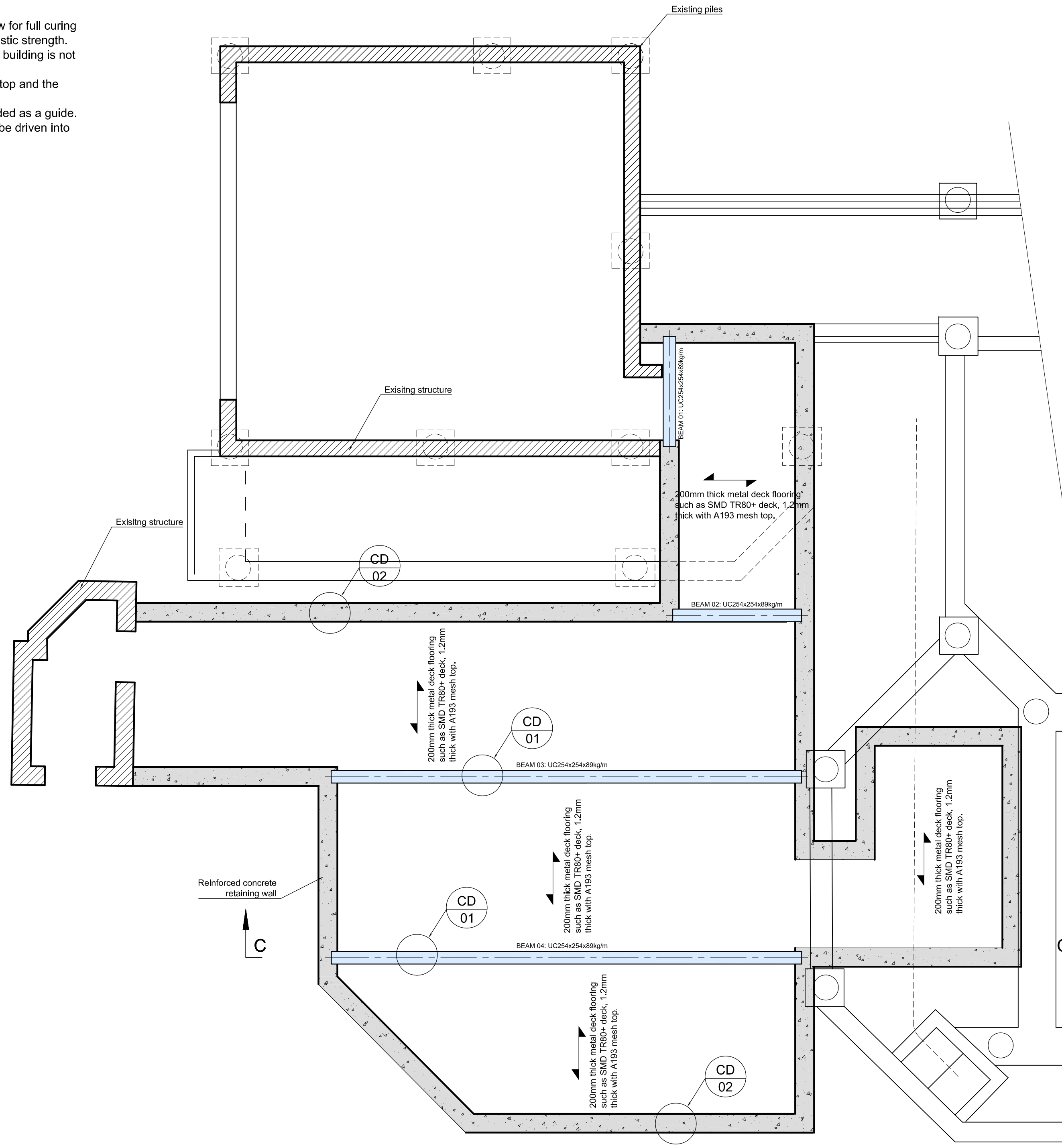
NOTES:

1. Pins to be constructed in maximum widths of 1.2m as described in the method statement.
2. When constructing a pin beside a previously cast pin, a sufficient period of time must have passed to allow for full curing of the original. Recommend seven days as concrete will have reached a minimum of 70% of its characteristic strength.
3. A maximum of 4 number pins may be under construction at any one time. This ensures the stability of the building is not compromised.
4. Upon completion of the pin and when it has fully cured it is to be carefully and full dry packed between its top and the overlying brickwork with a high strength expanding grout as per details.
5. Temporary works are the responsibility of the main contractor but sequencing information has been provided as a guide.
6. Individual pins are to be doweled together to ensure wall acts monolithically in its final state. Dowels can be driven into soil adjacent before pins are cast or can be chemically anchored after.



BASEMENT 1
PROPOSED FOUNDATION PLAN.
(1:50 @A1)

0 1 2 3 4 5



BASEMENT 1
PROPOSED BASEMENT PLAN.
(1:50 @A1, Structure Above.)

0 1 2 3 4 5

PLANNING

TYPE 1 PINS.	TYPE 2 PINS.	RC BASE.
TYPE 3 PINS.	TYPE 4 PINS.	

Rev	Description	Chked	Date

Project
5 Highfield Grove
Highgate

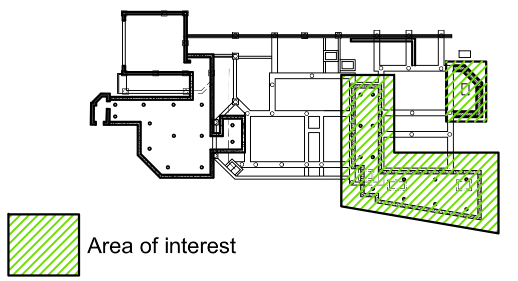
Client

Drawing Title
Proposed Foundations & Steel Above

Drawn	Checked	Drawing No:	
MD	NJR	2014-207-01	
Date	Job No:	Scale	Rev
06/10/2014	2014-207	1:50 at A1 1:100 at A3	0

ELITE Designers
12 Princeton Court, 55 Felsham Road, London SW15 1AZ
Tel: 020 8785 4499 Fax: 020 8785 4999

Key Section



NOTES:

1. Pins to be constructed in maximum widths of 1.2m as described in the method statement.
2. When constructing a pin beside a previously cast pin, a sufficient period of time must have passed to allow for full curing of the original. Recommend seven days as concrete will have reached a minimum of 70% of its characteristic strength.
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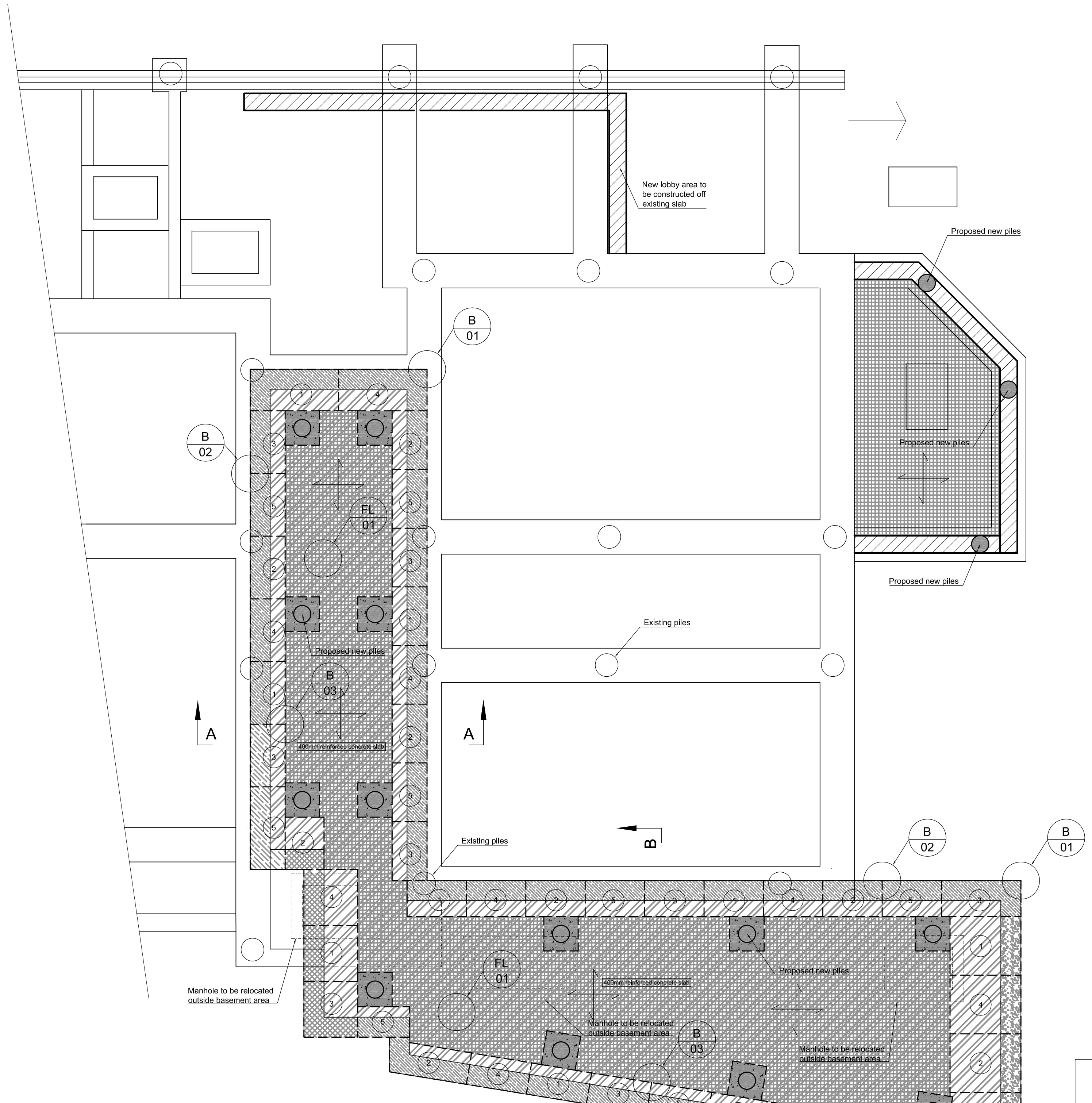
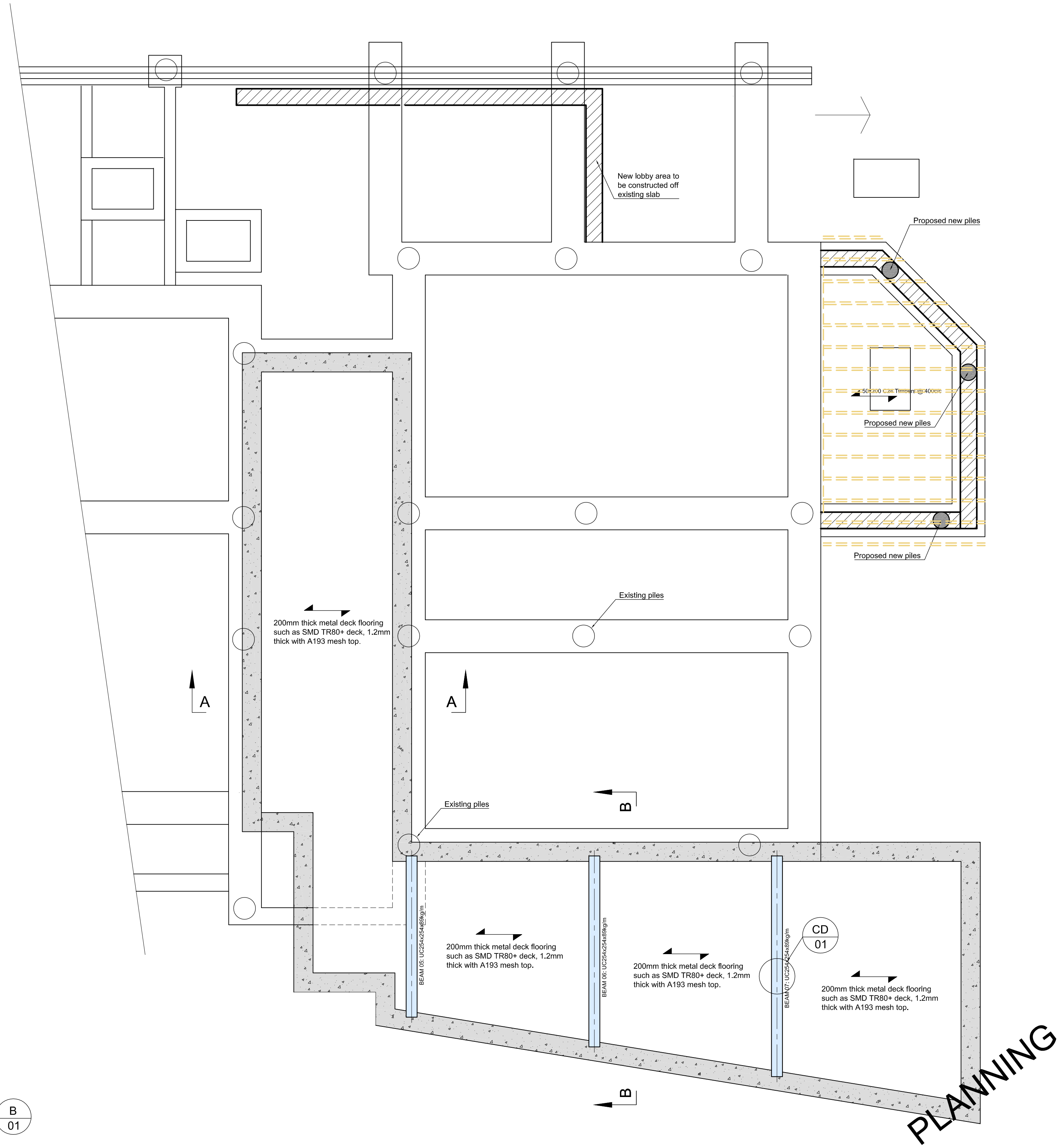


BASEMENT 2
PROPOSED FOUNDATION PLAN.
(1:50 @A1)

0 1 2 3 4 5



BASEMENT 2
PROPOSED BASEMENT PLAN.
(1:50 @A1, Structure Above.)

0 1 2 3 4 5

TYPE 5 PINS. TYPE 6 PINS. TYPE 7 PINS. RC BASE.

Rev	Description	Chked	Date

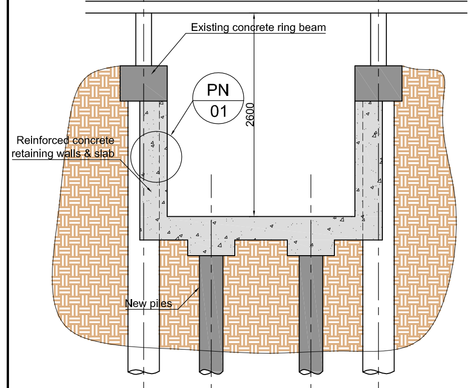
Project
5 Highfield Grove
Highgate

Client

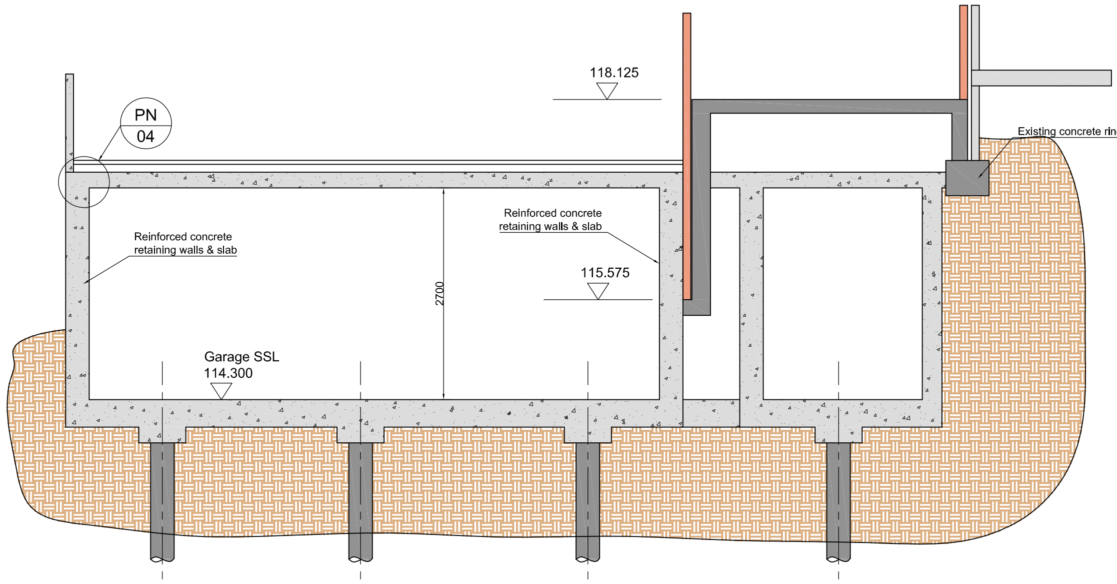
Drawing Title
Proposed Foundations & Steel Above

Drawn	Checked	Drawing No:	
MD	NJR	2014-207-02	
Date	**Job No:**	**Scale**	**Rev**
06/10/2014	2014-207	1:50 at A1 1:100 at A3	0

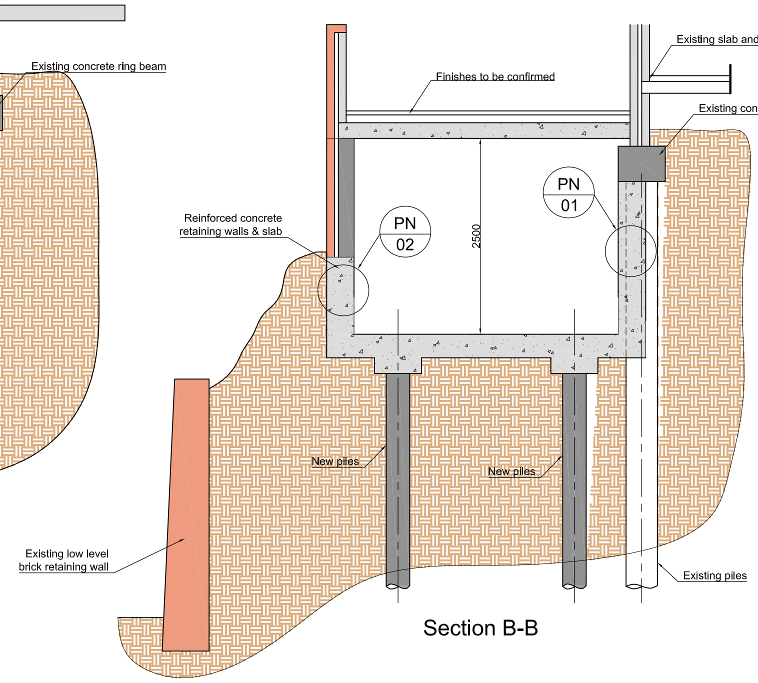
ELITE Designers
12 Princeton Court, 55 Felsham Road, London SW15 1AZ
Tel: 020 8785 4499 Fax: 020 8785 4999



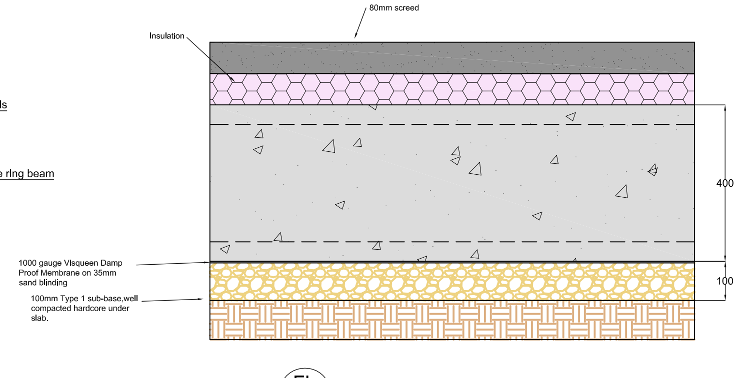
Section A-A



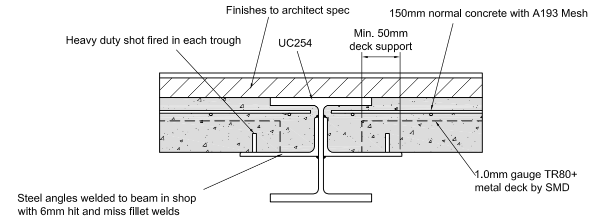
Section C-C



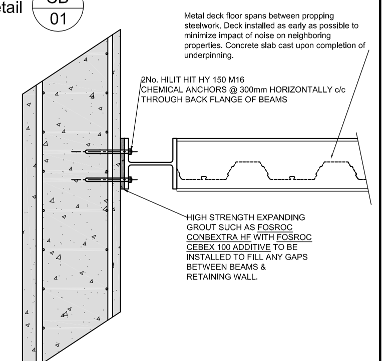
Section B-B



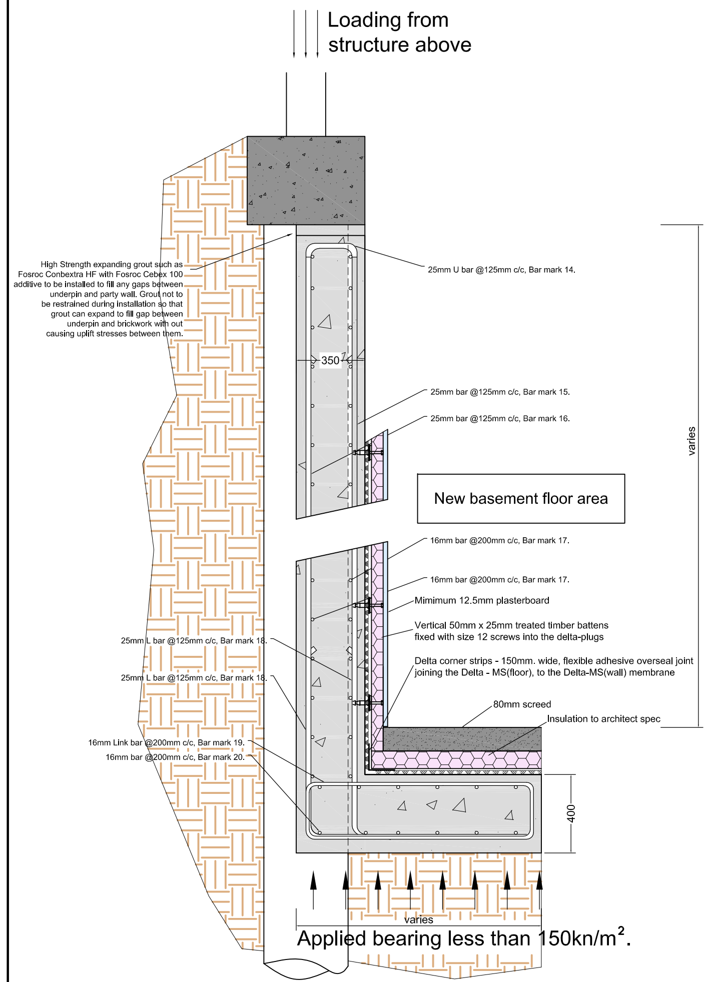
FL 01 Typical Floor Slab details



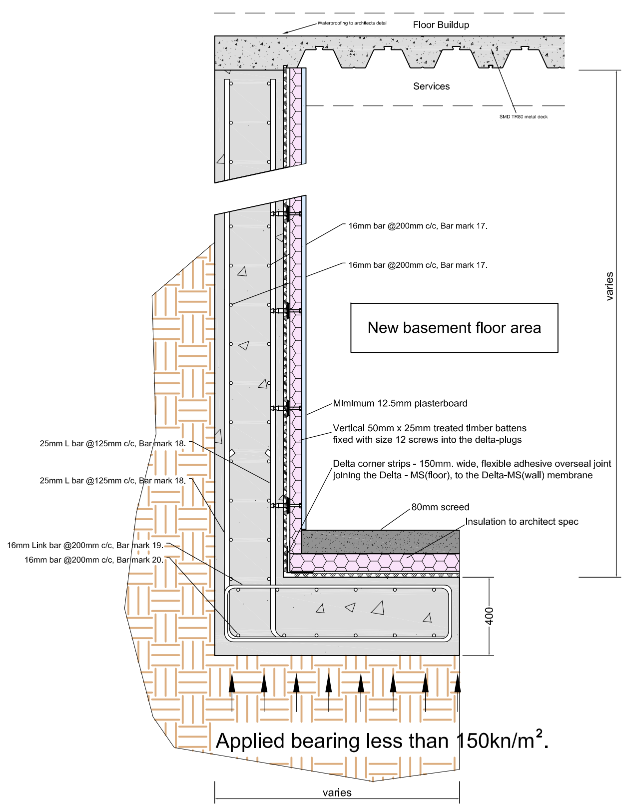
Deck between beams Detail CD 01



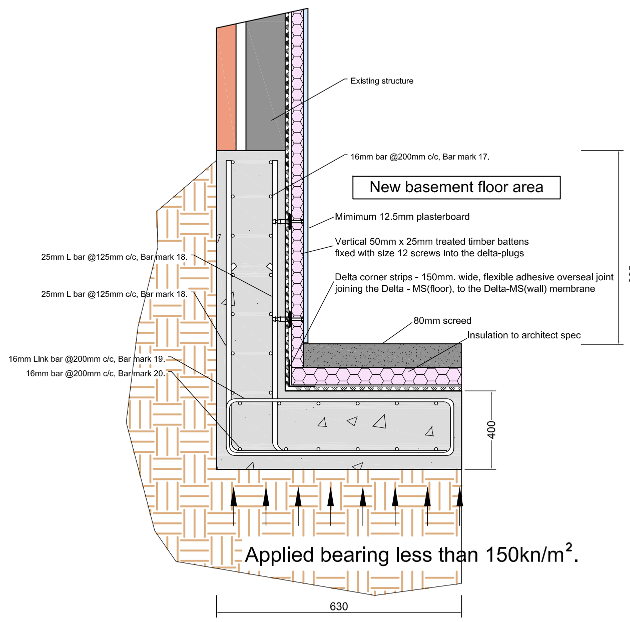
CD 02 BEAM CONNECTION TO UNDERPIN



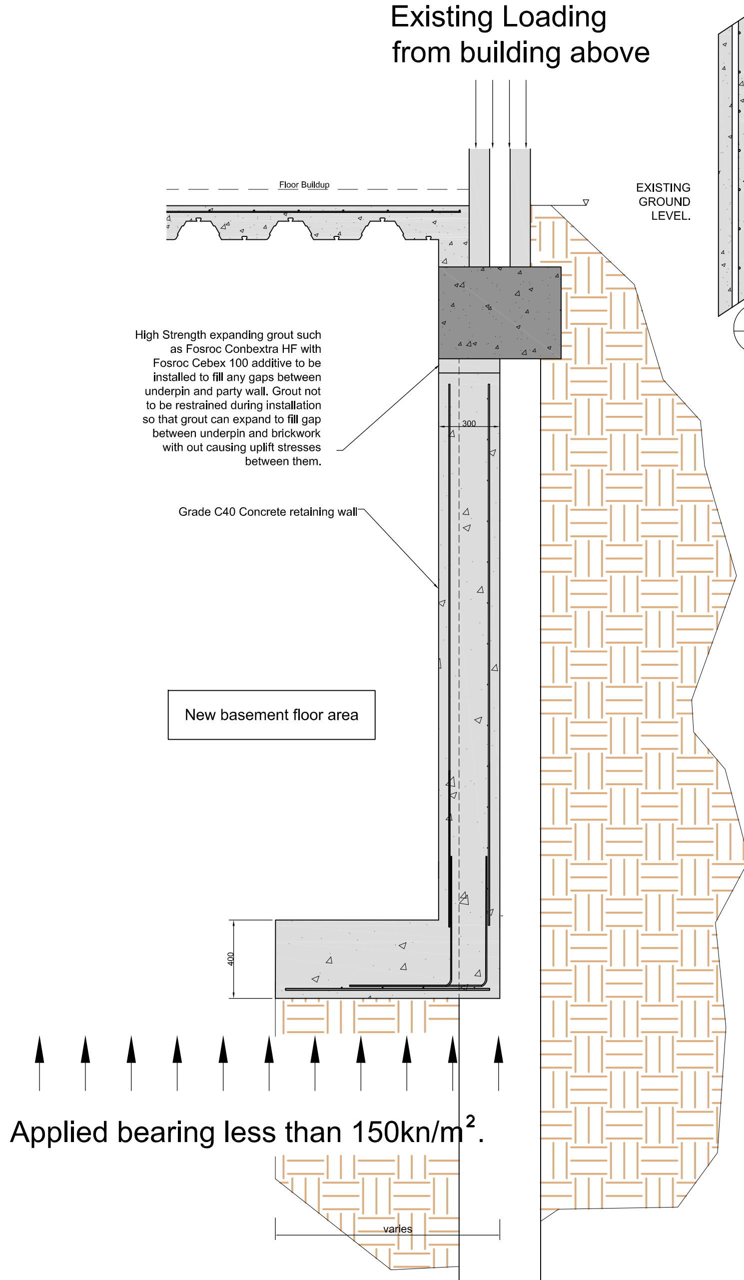
PN 01 Typical Section Below Existing Structure 1



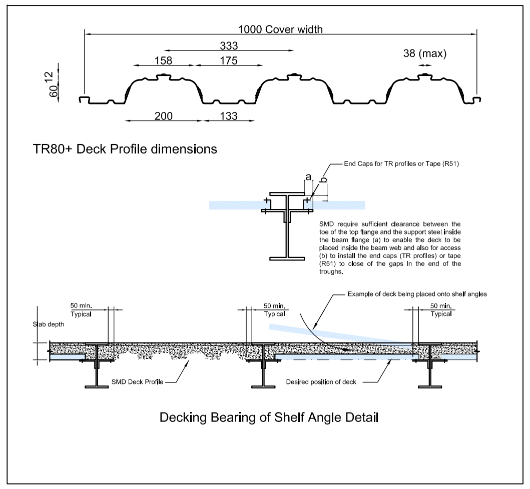
PN 04 Typical Section

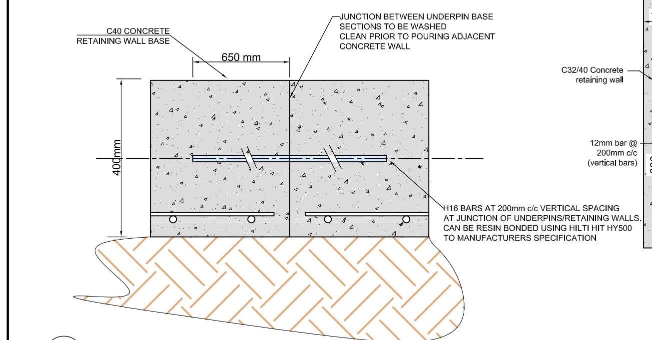


PN 02 Typical Section Below Existing Structure 2

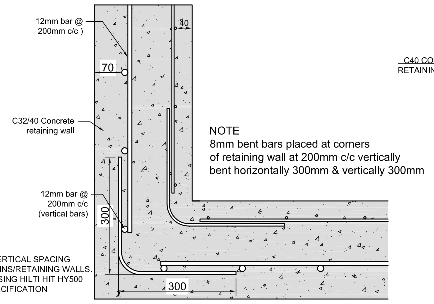


PN 03 Typical Section Below Existing Structure 3

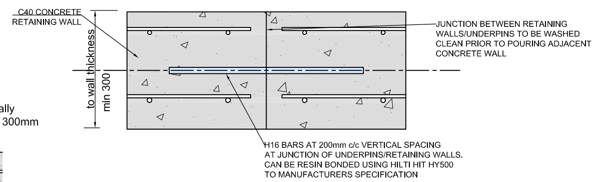




B 03 TYPICAL UNDERPIN BASE JUNCTION DETAIL



Retaining wall corner B 01

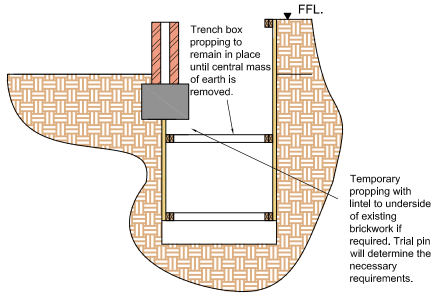


TYPICAL RETAINING WALL JUNCTION DETAIL B 02

PLANNING

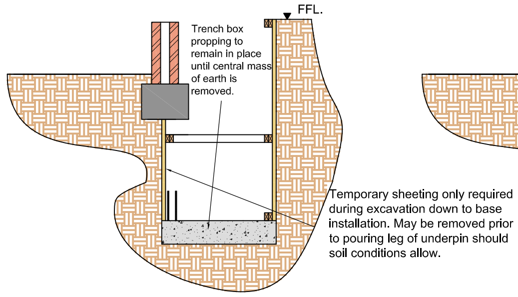
Rev	Description	Chked	Date
Project	5 Highfield Grove Highgate		
Client			
Drawing Title	Proposed Details		
Drawn: MD	Checked: NJR	Drawing No: 2014-207-03	
Date: 07/10/2014	Job No: 2014-207	Scale: 1:50 at A1 1:100 at A3	Rev: 0

ELITE Designers



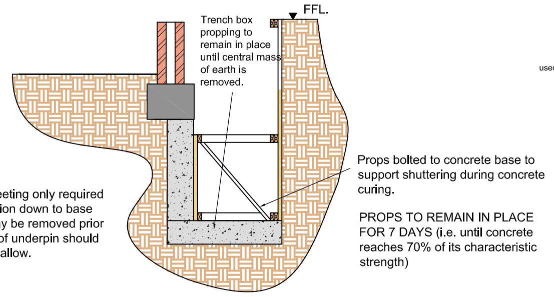
Stage 1:

Excavate working space to a depth level with the bottom of the base of the underpin. Temporary support to excavation as per detail. Existing foundation at ground level will span over the the excavation.



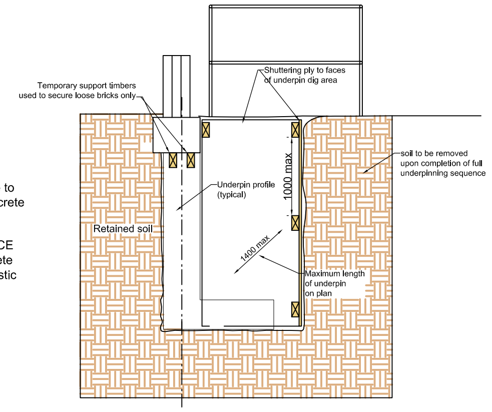
Stage 2:

Base of underpin is cast after reinforcing steelwork has been placed.



Stage 3:

Leg of underpin is poured, in this case it can be achieved with a single pour. Once concrete has been cured, neighboring pin can be excavated.

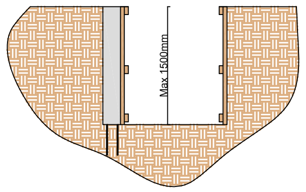


Basement Underpin Temporary propping detail (typical)

Sequence for Pin installation.

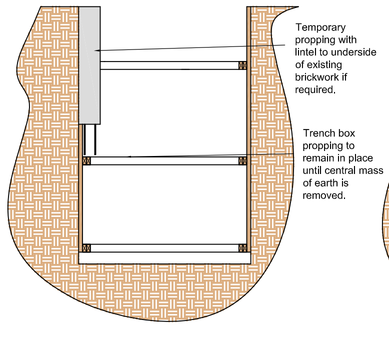
- Pins marked 1 can be installed first to a maximum width of 1400mm, but to their full required depth using temporary supports as shown.
- Adjacent pins marked 2 can be installed after completion of the installation of number 1 pin but only after the first pins have been allowed to fully cure.
- Finally infill pins marked 3 can be installed.

Method 1: Used for underpins in areas where vertical load from existing structure above is present. As long as each pin is no more that 1200mm wide and carries loading from above then the pin is stable in all temporary conditions without the presence of the basement floor slab. The existing foundation at ground level will have adequate capacity to span the 1200mm of the excavation necessary for the installation of the pin.



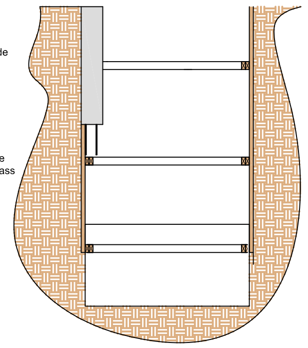
Stage 1:

A trench is dug and the top 1500mm of the retaining wall is cast along the full length of this type of retaining wall. This acts as a ring beam along the excavation.



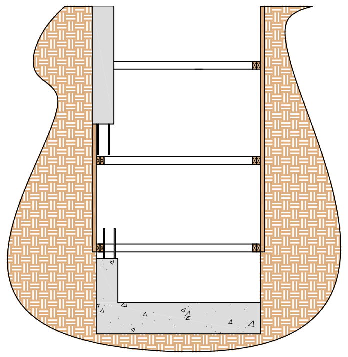
Stage 2:

A 1.4m wide section is excavated down to the base of the wall of the pin being considered.



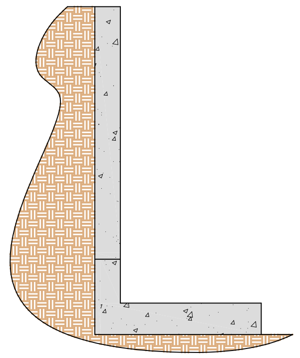
Stage 3:

As per method 1 the excavation is prepared for the base to be poured.



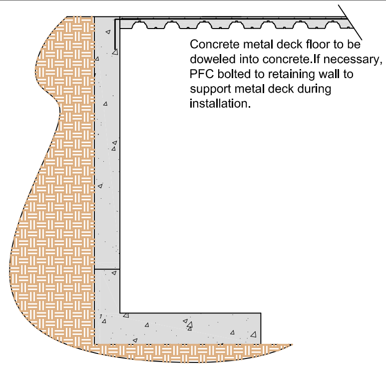
Stage 4:

Base of underpin is cast with kicker to rise above water table.Bar left for full lap length connection to rest of wall stem.



Stage 5:

Final section of stem is cast and once cured propping can be removed and neighbouring pin section can be excavated.



Stage 6:

Ground floor slab is installed.

Method 2: Used for pins in areas where little / no vertical load is applied to wall in permanent load case.

Notes

1. This drawing is to be read in conjunction with all relevant architects, engineers & specialist sub-contractors drawings and the specification.

2. Any discrepancies between the site conditions and these drawings to be reported to Elite Designers. Dimensions must not be scaled and should be checked on site.

3. All dimensions are in millimetres, levels are in metres a.o.d. (above ordnance datum).

4. Foundations have been designed on a safe increase in bearing pressure of 150kN/m² bearing 200mm into sandy gravel strata.

5. All new steelwork to be grade S275 and be supplied to site blast cleaned to swedish standard SA2½ painted with high build zinc phosphate alkyd primer to 80 microns after fabrication. Any mechanical damage to coating to be touched up on site in accordance with the specification.

6. All new steel beams to have a minimum of 100mm bearing either end.

7. Lengths of all members are to be verified on site by the Contractor.

8. Catnic type lintels to have a minimum bearing of 150mm either end.

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10. Cover to reinforcement to be 25mm to all bars unless noted otherwise.

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13. The principal contractor is to ensure that at all times the excavations shall remain free from standing water.

14. Movement joints to be positioned @ 6m c/c in blockwork and @ 12m c/c in brickwork.

15. Movement joints to be 15mm hydrocell or similar joint filler with a 15x15mm two part polysulphate sealant. (colour and fire resistance of sealant to be advised by architect).

16. All load bearing blockwork below DPC to be 7N/mm² dense concrete block.

17. Provide Ancon ST1 wall ties in accordance with DD140 @ 450 c/c vertically and @ 900 c/c horizontally, staggered u.n.o.

18. All bolts to be Grade 8.8 M20 unless noted otherwise.

19. All insulation details have been produced to comply with relevant regulations where possible. However, the responsibility for checking the compliance and execution of insulation details lies with the main contractor.

20. Floor joists spanning in excess of 2.5m should be strutted by one or more rows of solid or herrringbone strutting as follows:

Joists <2.5m - None required
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21. All beam end reactions shown are unfactored unless noted otherwise.

PLANNING

Rev	Description	Chked	Date

Project
5 Highfield Grove
Highgate

Client

Drawing Title

Proposed Construction Sequences

Drawn	Checked	Drawing No:	
MD	NJR	2014-207-04	
Date	Job No:	Scale	Rev
06/10/2014	2014-207	1:50 at A1 1:100 at A3	0

ELITE Designers
12 Princeton Court, 55 Felsham Road, London SW15 1AZ
Tel: 020 8785 4499 Fax: 020 8785 4999

Appendix B: Preliminary Calculations

5 Highfields Grove, SN6 6HN

Structural Design Criteria

Job No. 2014-207 October 2014

1.0 Project Description

Elite designers were engaged to consult on the structural engineering of the project.
The details set out below are the standard criteria and documentation used by Elite designers to assess the project from a structural engineering point of view. It details out standard materials and there specifications to be used in addition to the minimum standards and quality the materials must meet in order to be compliant with both our design, British and European standards.

2.0 Design Standards

The following are the principal standards used in the design:

BS6399: Part 1:1996	British Standards: Loading for buildings. Part 1: Code of Practice for dead and imposed loads.
BS En 1991-1	Euro code 1. Code of Practice for wind loads
BS6399: Part 3:1988	British Standard: Loading for Buildings (amended May 1997). Part 3: Code of Practice for imposed roof loads.
BS En 1992 -1	Euro code 2 Code of Practice for design and construction of concrete structures.
BS En 1993 -1	Euro code 3 Structural use of steelwork in building.
BS8004:1986	British Standard: Code of practice for foundations.

3.0 Materials

3.1 Concrete

Normal weight concrete to BS 8500.

Assumed concrete grades and cover to reinforcement in given locations are as follows:

Concrete Grade	Location	Cover
C40	Foundations	50mm for formed sides 75mm for cast against ground
C35	Internal areas	35mm (typical)

Concrete Properties:

Density:	24 kN/m^3 (normal-weight concrete)
Young's Modulus (short-term):	Ec = 27,000 N/mm^2 for Grade C35
Poisson's Ratio:	ν = 0.15
Coefficient of thermal expansion:	α = 10 x 10 –6/°C
Long term elastic modulus	$Ec_{long\ term}$ = 13,500 N/mm^2 for Grade C35

3.2 Reinforcement

Deformed reinforcing bars: BS 4449, Grade 460 (fy = 460 N/mm^2).

Steel fabric: BS 4483 (minimum fy = 460 N/mm^2).

3.3 Structural Steelwork

Hot-rolled sections, bars and plates: BS EN 10025, Grades S275 and S355.

Steel Designation	Minimum yield strength (N/mm2) by nominal thickness						Minimum tensile strength (N/mm2)	
	t<16	>16 <40	>40 <63	>63 <80	>80 <100	>100 <150	t<100	>100 <150
S275	275	265	255	245	235	225	410	400
S355	355	345	335	325	315	295	490	470

Steel hollow sections: BS EN 10210, Grade S355 (and Grade S275).
Steel shapes shall be selected from BS 4 and BS EN 10210.
Angle shapes shall be selected from BS4848.

Steel properties:

Density:	78 kN/m^3
Young's Modulus (short-term):	E = 205,000 N/mm^2
Poisson's Ratio:	ν = 0.30
Coefficient of thermal expansion:	α = 11.7 x 10 –6/°C

3.4 Bolts

HSFG bolts: BS 4395. Preferred sizes are 20 ∅ and 24∅.

Bearing bolts: BS3692, Grade 8.8. Preferred sizes are 20 ∅ and 24∅.

3.5 Welding

For S275 steel: Grade E43 to BS639.
For S355 steel: Grade E51 to BS639.

4.0 Gravity Loads

4.1 Material Self-Weight

Dead loads have been calculated using the following material densities:

Concrete (normal weight)	24 kN/m^3
Steel	77 kN/m^3
Concrete block work walls	20 kN/m^3
Concrete fill (normal weight)	24 kN/m^3

Dead loads are to be calculated from detail information of floor and roof build ups as shown in detailed drawings.

4.2 Live Loads – General

Live loads assumed for each occupancy are as follows:

	Uniform Load (kN/m^2)	*Concentrated Load (kN)
Roof (with access)	1.5	1.8
Roof (without access)	0.6	0.9
Offices	2.5	2.7
Restaurants, Bars and Lounges	5.0	3.6
Reception Areas	5.0	3.6
Changing Rooms and Toilets	2.0	1.8
Corridors & stairs	4.0	4.5
Plant rooms	7.5 NR	4.5
Car Parks	2.5	9.0
Mezzanine storage	2.4Kn per metre height of storage	

* Concentrated loads shall act over an area 50mm x 50mm unless otherwise noted.
"NR" denotes uniform loads that are non-reducible. Other live loads may be reduced in accordance with codes.

5.0 Wind Load Criteria

5.1 Basic Wind Speed

According to the wind speed map for Great Britain and Ireland the basic wind speed at the site is 21 m/s.

5.2 Wind Speed

The site wind speed is determined from the basic wind speed taking into consideration the influence of the site altitude, direction, seasonal changes in climate and a probability factor.

Altitude factor, S_a = 1 + 0.001 ΔS = 1 + 0.001 x m = 1.02

Direction factor, S_d = 1.0

Seasonal factor S_s: as the building is considered to be exposed to wind for a period greater than 6 months, no reduction applies. S_s = 1.0.

Probability factor Sp: the standard probability of exceeding the basic wind speed is used. Sp = 1.0.

Site wind speed, Vs = Vb x Sa x Sd x Ss x Sp = 21.42 = m/s

5.3 Effective Wind Speed

The effective wind speed takes into account the effective height of the building (effect of neighbouring buildings), the closest distance to the sea and the location of the site (town or country).

Effective height He: conservatively take He = Hr = 20 m.

Closest distance to sea: 30km

Town/Country: the building site is located within town.

Terrain and building factor S_b = 1.96

Effective wind speed $V_e = V_s \times S_b$ = 41.98

Dynamic Pressure, $q_s = 0.613 \times V_e^2$ = 1.1 kN/m²

Further reduction in the wind loading may be achieved through more accurate means of wind loading.

6.0 Foundation Design

Refer to soil investigation report for further detail of ground properties.

Allowable bearing capacity	=	170	Kn/m2
Density, ρ	=	20	kN/m³
Angle of internal friction, ϕ'	=	30°	

Groundwater was found to be generally up to 11m OD MH but for design purposes the ground water will be taken to be at **6m OD MH**.

7.0 Performance Design Criteria

7.1 Beam and Slab Deflections

Slabs and beams have typically been designed to the span/effective depth limits stated in BS En1992. Per BS En 1992, these span/effective depth limits "are based on limiting the **total deflection** to span/250 and this should normally ensure that the part of the deflection occurring after construction of finishes and partitions **(imposed load deflection)** will be limited to span/500.

7.2 Building Sway

The building sway (measured at the highest occupied level, relative to foundation level) is limited to:

H/500 for wind loading (for 50 year return period)

7.3 Interstory Drift

For concrete structures subject to wind loads, the interstorey drift (racking component) is limited to:

H/500 (H = storey height).

For steel structures subject to wind loads, the interstorey drift is limited to the following:

H/500 for sway frames

H/300 for other systems

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7.4 Floor Vibration

The natural frequency of long span floor beams shall not be less than 4 Hz.

8.0 Load Combinations

The table below gives the different loading combinations applied to the structure in order to provide the worst case loading scenario for all elements of the structure. The resulting maximum and minimum loads from each of the five combinations below must be checked through the design. The ultimate limit state load combinations factors for concrete and steel are as follows:

Load Combination	**Load type**						
	Dead		Imposed		Earth & water pressure	Wind	
	Adverse	Beneficial	Adverse	Beneficial			
1. Dead and imposed (and earth and water pressure)	1.4	1.0	1.6	0	1.4	-	
2. Dead and wind (and earth and water pressure)	1.4	1.0	-	-	1.4	1.4	
3. Dead and wind and imposed (and earth and water pressure)	1.2	1.2	1.2	1.2	1.2	1.2	
*4. Dead and seismic (and earth and water pressure).	1.4	1.0	-	-	1.4	-	
*5. Dead and seismic and imposed (and earth and water pressure)	1.2	1.2	1.2	1.2	1.2	-	

Structural Calculations - 5 Highfields Grove, London, SN6 6HN

Project: New Basement & House Renovation

The following calculations ascertain the structural integrity of the proposed alterations to the address above. Reference should be made to Elite Designers Ltd sketches for structural details and layout drawings.

Loadings from BS648 & BS6399 : Part 1 : 1984

Dead Loads

Ceiling	Thermal Insulation	$c_1 := 0.01\cdot 10^3\cdot newton\cdot m^{-2}$	
	Ceiling Joists	$c_2 := 0.16\cdot 10^3\cdot newton\cdot m^{-2}$	
	Plaster Skim	$c_3 := 0.03\cdot 10^3\cdot newton\cdot m^{-2}$	
	Plaster Board	$c_4 := 0.11\cdot 10^3\cdot newton\cdot m^{-2}$	
	Total Ceiling Load	$C1 := c_1 + c_2 + c_3 + c_4$	$C1 = 310\,m^{-2}\cdot newton$
Flat Roof	Asphalt 2 layers 19mm	$r_3 := 0.41\cdot 10^3\cdot newton\cdot m^{-2}$	
	Joists with decking	$r_4 := 0.25\cdot 10^3\cdot newton\cdot m^{-2}$	
	Total Ceiling Load	$R_2 := r_3 + r_4 + C1 - c_2$	$R_2 = 810\,m^{-2}\cdot newton$
Roof 37deg Pitch	Slate Tiling	$r_1 := 0.5\cdot 10^3\cdot newton\cdot m^{-2}$	
	Roof Rafters	$r_2 := 0.16\cdot 10^3\cdot newton\cdot m^{-2}$	
	Total Roof Load	$R_1 := r_1 + r_2 + C1 - c_2$	$R_1 = 810\,m^{-2}\cdot newton$
Wall Loads	Stud, Lathe and Plaster	$w_1 := 0.76\cdot 10^3\cdot newton\cdot m^{-2}$	
	Brick 300mm cavity	$w_2 := 3.76\cdot 10^3\cdot newton\cdot m^{-2}$	
	Brick 9" solid	$w_3 := 5.33\cdot 10^3\cdot newton\cdot m^{-2}$	
	Brick 13" solid	$w_4 := 7.69\cdot 10^3\cdot newton\cdot m^{-2}$	
	Brick 4.5" solid	$w_5 := 2.655\cdot 10^3\cdot newton\cdot m^{-2}$	
	New Stud Walls	$w_6 := 0.5\cdot 10^3\cdot newton\cdot m^{-2}$	
	140mm Blockwork Wall	$w_7 := 1.5\cdot 10^3\cdot newton\cdot m^{-2}$	
Floor Loads	225x50 Joists With Decking	$f_1 := 0.32\cdot 10^3\cdot newton\cdot m^{-2} + c_1 + c_3 + c_4$	
		$f_1 = 470\,m^{-2}\cdot newton$	
Low Profile Deck (Lewis Dovetailed Sheeting over Joists)	Lewis Deck Depth	$h_s := 0.05\cdot m$	
	Lewis Deck Weight	$w_s := 0.058\cdot kN\cdot m^{-2}$	

Elite Designers Ltd
12 Princeton Court
55 Felsham Road
London SW15 1AZ

020 8785 4499

Project: **5 Highfields Grove, London, SN6 6HN**
Section: **Structural Calculations**
Calc by: **MD** Date: **13/10/14**

Job Ref: **2014-207**
Sheet No./Rev. **0**
Check/App'd by: **JF** Date: **13/10/14**

Density of Concrete	$\rho_c := 24\cdot kN\cdot m^{-3}$	
Lewis Dead Load	$R_d := w_s + (h_s\cdot \rho_c)$	$R_d = 1.258\,m^{-2}\cdot kN$
10mm Screed	$s_{cr} := 0.01\cdot m\cdot \rho_c$	$s_{cr} = 0.24\,m^{-2}\cdot kN$
12.5mm Stone Finish	$s_t := 0.0125\cdot m\cdot \rho_c$	$s_t = 0.3\,m^{-2}\cdot kN$
Total Floor Load	$f_2 := f_1 + R_d + s_{cr} + s_t$	$f_2 = 2.268\,m^{-2}\cdot kN$

TR80+ Metal Decking - Ground, LGF & Basement Floors

Designed for Vehicle Load 2.5kN/m² to span < 3.75m

Density of Concrete	$\rho_c := 24\cdot kN\cdot m^{-3}$	
Holorib Deck Depth	$h_{D80} := 0.14\cdot m$	
Holorib Deck Weight	$w_{D80} := 0.123\cdot kN\cdot m^{-2}$	
Volume of Concrete	$V_{CD80} := 0.098\cdot m^3\cdot m^{-2}$	
Holorib Dead Load	$DL_{D80} := w_{D80} + (V_{CD80}\cdot \rho_c)$	$DL_{D80} = 2.475\,m^{-2}\cdot kN$
50mm Screed	$scr_{50} := 0.05\cdot m\cdot \rho_c$	$scr_{50} = 1.2\,m^{-2}\cdot kN$
Services	$s_v := 0.5\cdot kN\cdot m^{-2}$	
Total Floor Load	$f_3 := DL_{D80} + scr_{50} + s_v$	$f_3 = 4.175\,m^{-2}\cdot kN$

Imposed Loading

Floor Load Table 5 BS6399	$I_{fl} := 1.5\cdot 10^3\cdot newton\cdot m^{-2}$
Roof Load	$I_{rl} := 0.6\cdot 10^3\cdot newton\cdot m^{-2}$
Storage Load	$I_{sl} := 0.75\cdot 10^3\cdot newton\cdot m^{-2}$

Safety Factors

Live Load Safety Factor γfl	$\gamma fl := 1.6$
Dead Load Safety Factor γfd	$\gamma fd := 1.4$

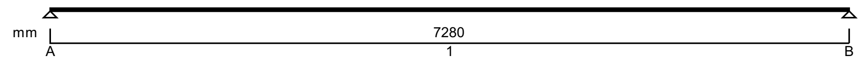
DESIGN DATA

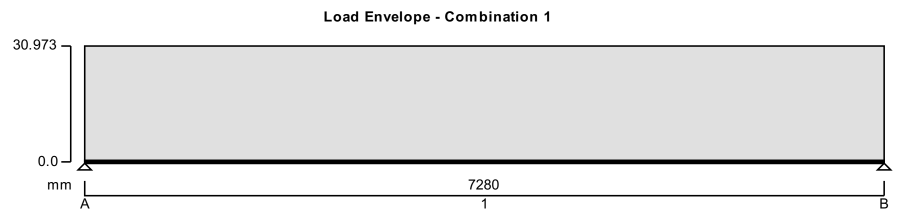
Heights	
Height: Basement to Ground Floor	$h_1 := 2.700m$
Height: Ground to First Floor	$h_2 := 3.240m$
Spans	
Max span for Ground Floor Steelwork (above basement)	$sp_1 := 3.900m$
Angles	
Angle of Main Roof	$\theta_1 := 45.00deg$
Angle of Rear Extension Skirt	$\theta_2 := 35.00deg$
Angle of Stairs	$\theta_3 := 40.00deg$

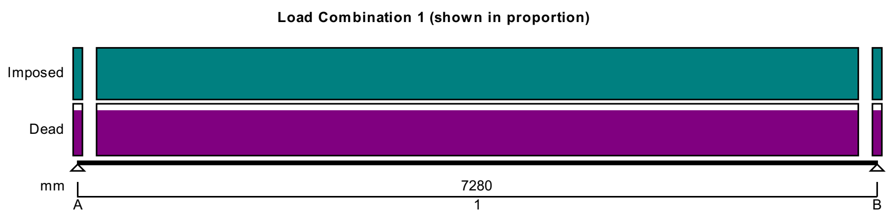
STEEL BEAM ANALYSIS & DESIGN (BS5950)

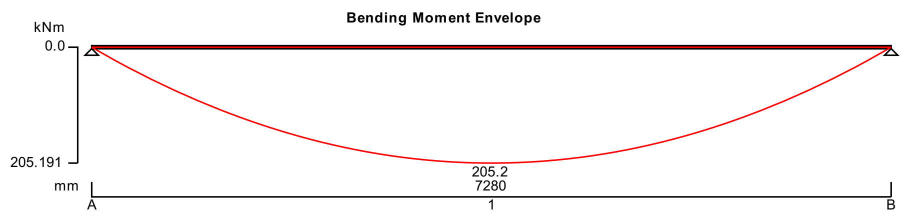
In accordance with BS5950-1:2000 incorporating Corrigendum No.1

TEDDS calculation version 3.0.04



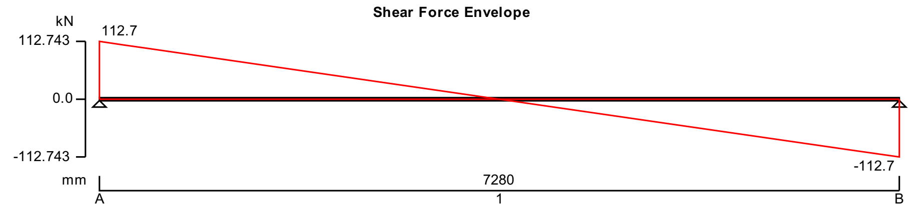






Tedds	Project	Job no.
	5 Highfields Grove, London SN6 6HN	2014-207
	Calcs for	Start page no./Revision
	Steel beam above new basement	2

Calcs by	Calcs date	Checked by	Checked date	Approved by	Approved date
MD	13/10/2014	JF			



Support conditions

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

Applied loading

Beam loads	Imposed full UDL 4.07 kN/m
	Dead full UDL 16.6 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

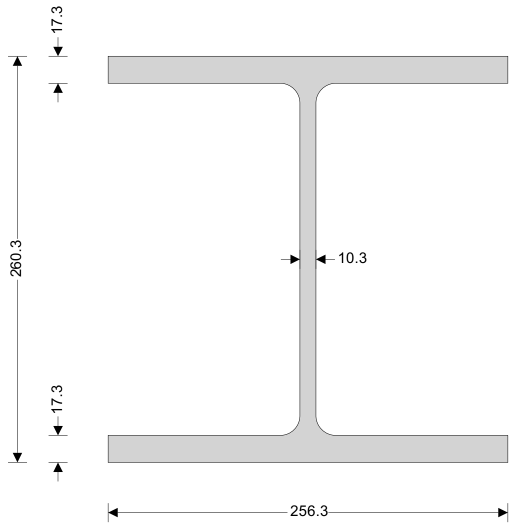
Maximum moment	M_{max} = **205.2** kNm	M_{min} = **0** kNm
Maximum shear	V_{max} = **112.7** kN	V_{min} = **-112.7** kN
Deflection	δ_{max} = **5.1** mm	δ_{min} = **0** mm
Maximum reaction at support A	R_{A_max} = **112.7** kN	R_{A_min} = **112.7** kN
Unfactored dead load reaction at support A	R_{A_Dead} = **63.6** kN	
Unfactored imposed load reaction at support A	$R_{A\_Imposed}$ = **14.8** kN	
Maximum reaction at support B	R_{B_max} = **112.7** kN	R_{B_min} = **112.7** kN
Unfactored dead load reaction at support B	R_{B_Dead} = **63.6** kN	
Unfactored imposed load reaction at support B	$R_{B\_Imposed}$ = **14.8** kN	

Section details

Section type	**UC 254x254x89 (BS4-1)**
Steel grade	**S275**

From table 9: Design strength p_y

Thickness of element	max(T, t) = **17.3** 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**

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 = **200.3** mm

$d / t = 19.1 \times \varepsilon <= 80 \times \varepsilon$ Class 1 plastic

Outstand flanges - Table 11

Width of section b = B / 2 = **128.2** mm

$b / T = 7.3 \times \varepsilon <= 9 \times \varepsilon$ Class 1 plastic

Section is class 1 plastic

Shear capacity - Section 4.2.3

Design shear force $F_v = \max(\text{abs}(V_{max}), \text{abs}(V_{min}))$ = **112.7** kN

$d / t < 70 \times \varepsilon$

Web does not need to be checked for shear buckling

Shear area $A_v = t \times D$ = **2681** mm^2

Design shear resistance $P_v = 0.6 \times p_y \times A_v$ = **426.3** kN

PASS - Design shear resistance exceeds design shear force

Moment capacity - Section 4.2.5

Design bending moment $M = \max(\text{abs}(M_{s1\_max}), \text{abs}(M_{s1\_min}))$ = **205.2** kNm

Moment capacity low shear - cl.4.2.5.2 $M_c = \min(p_y \times S_{xx}, 1.2 \times p_y \times Z_{xx})$ = **324.3** kNm

PASS - Moment capacity exceeds design bending moment

Check vertical deflection - Section 2.5.2

Consider deflection due to imposed loads

Limiting deflection $\delta_{lim} = L_{s1} / 360$ = **20.222** mm

Maximum deflection span 1 $\delta = \max(\text{abs}(\delta_{max}), \text{abs}(\delta_{min}))$ = **5.089** mm

PASS - Maximum deflection does not exceed deflection limit

RC SLAB DESIGN (BS8110:PART1:1997)

TEDDS calculation version 1.0.04

TWO WAY SPANNING SLAB DEFINITION – SIMPLY SUPPORTED

Overall depth of slab h = **400** mm

Outer sagging steel

Cover to outer tension reinforcement resisting sagging c_{sag} = **35** mm

Trial bar diameter D_{tryx} = **20** mm

Depth to outer tension steel (resisting sagging)

$d_x = h - c_{sag} - D_{tryx}/2$ = **355** mm

Inner sagging steel

Trial bar diameter D_{tryy} = **20** mm

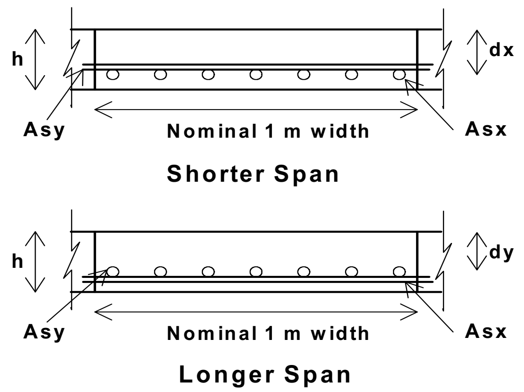
Depth to inner tension steel (resisting sagging)

$d_y = h - c_{sag} - D_{tryx} - D_{tryy}/2$ = **335** mm

Materials

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

Characteristic strength of concrete f_{cu} = **35** N/mm^2



Two-way spanning slab
(simple)

MAXIMUM DESIGN MOMENTS

Length of shorter side of slab l_x = **6.000** m

Length of longer side of slab l_y = **7.070** m

Design ultimate load per unit area n_s = **3.5** kN/m^2

Moment coefficients

$\alpha_{sx} = (l_y / l_x)^4 / (8 \times (1+(l_y / l_x)^4))$ = **0.082**

$\alpha_{sy} = (l_y / l_x)^2 / (8 \times (1+(l_y / l_x)^4))$ = **0.059**

Maximum moments per unit width - simply supported slabs

$m_{sx} = \alpha_{sx} \times n_s \times l_x^2$ = **10.4** kNm/m

$m_{sy} = \alpha_{sy} \times n_s \times l_x^2$ = **7.5** kNm/m

CONCRETE SLAB DESIGN – SAGGING – OUTER LAYER OF STEEL (CL 3.5.4)

Design sagging moment (per m width of slab) m_{sx} = **10.4** kNm/m

Moment Redistribution Factor β_{bx} = **1.0**

Area of reinforcement required

$K_x = abs(m_{sx}) / (d_x^2 \times f_{cu})$ = **0.002**

$K'_x = \min (0.156 , (0.402 \times (\beta_{bx} - 0.4)) - (0.18 \times (\beta_{bx} - 0.4)^2))$ = **0.156**

Outer compression steel not required to resist sagging

Slab requiring outer tension steel only - bars (sagging)

$z_x = \min ((0.95 \times d_x),(d_x \times (0.5+\sqrt{(0.25-K_x/0.9)})))$ = **337** mm

Neutral axis depth $x_x = (d_x - z_x) / 0.45$ = **39** mm

Area of tension steel required

$A_{sx\_req} = abs(m_{sx}) / (1/\gamma_{ms} \times f_y \times z_x)$ = **71** mm^2/m

Tension steel

Provide 10 dia bars @ 100 centres outer tension steel resisting sagging

$A_{sx\_prov} = A_{sx}$ = **785** mm^2/m

Area of outer tension steel provided sufficient 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} = **7.5** kNm/m

Moment Redistribution Factor β_{by} = **1.0**

Area of reinforcement required

$K_y = abs(m_{sy}) / (d_y^2 \times f_{cu})$ = **0.002**

$K'_y = \min (0.156 , (0.402 \times (\beta_{by} - 0.4)) - (0.18 \times (\beta_{by} - 0.4)^2))$ = **0.156**

Inner compression steel not required to resist sagging

Slab requiring inner tension steel only - bars (sagging)

$z_y = \min ((0.95 \times d_y),(d_y \times (0.5+\sqrt{(0.25-K_y/0.9)})))$ = **318** mm

Neutral axis depth $x_y = (d_y - z_y) / 0.45$ = **37** mm

Area of tension steel required

$A_{sy\_req} = abs(m_{sy}) / (1/\gamma_{ms} \times f_y \times z_y)$ = **54** mm^2/m

Tension steel

Provide 10 dia bars @ 100 centres inner tension steel resisting sagging

A_{sy_prov} = A_{sy} = **785** mm^2/m

Area of inner tension steel provided sufficient to resist sagging

Check min and max areas of steel resisting sagging

Total area of concrete A_c = h = **400000** mm^2/m

Minimum % reinforcement k = **0.13** %

A_{st_min} = k × A_c = **520** mm^2/m

A_{st_max} = 4 % × A_c = **16000** mm^2/m

Steel defined:

Outer steel resisting sagging A_{sx_prov} = **785** mm^2/m

Area of outer steel provided (sagging) OK

Inner steel resisting sagging A_{sy_prov} = **785** mm^2/m

Area of inner steel provided (sagging) OK

CONCRETE SLAB DEFLECTION CHECK (CL 3.5.7)

Slab span length l_x = **6.000** m

Design ultimate moment in shorter span per m width m_{sx} = **10** kNm/m

Depth to outer tension steel d_x = **355** mm

Tension steel

Area of outer tension reinforcement provided A_{sx_prov} = **785** mm^2/m

Area of tension reinforcement required A_{sx_req} = **71** 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.

$f_s = 2 \times f_y \times A_{sx\_req} / (3 \times A_{sx\_prov} \times \beta_{bx}) = \mathbf{30.0}$ N/mm^2

$factor_{tens} = \min(2, 0.55 + (477\ N/mm^2 - f_s) / (120 \times (0.9\ N/mm^2 + m_{sx} / d_x^2))) = \mathbf{2.000}$

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 $l_{max} = ratio_{span\_depth} \times factor_{tens} \times d_x = \mathbf{14.20}$ m

Check the actual beam span

Actual span/depth ratio l_x / d_x = **16.90**

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 = **400** mm

Effective depth to bottom outer tension reinforcement d_x = **355.0** mm

Diameter of tension reinforcement D_x = **10** mm

Diameter of links L_{diax} = **0** mm

Cover to outer tension reinforcement

$c_{tenx} = h - d_x - D_x / 2 = \mathbf{40.0}$ mm

Nominal cover to links steel

$c_{nomx} = c_{tenx} - L_{diax} = \mathbf{40.0}$ mm

Permissable minimum nominal cover to all reinforcement (Table 3.4)

c_{min} = **35** 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 = **400** mm

Outer sagging steel

Cover to outer tension reinforcement resisting sagging c_{sag} = **35** mm

Trial bar diameter D_{tryx} = **20** mm

Depth to outer tension steel (resisting sagging)

$d_x = h - c_{sag} - D_{tryx}/2 = \mathbf{355}$ mm

Inner sagging steel

Trial bar diameter D_{tryy} = **20** mm

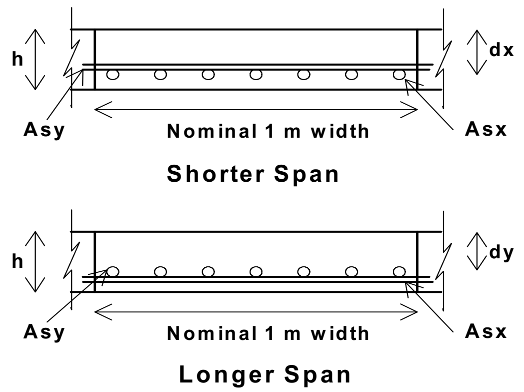
Depth to inner tension steel (resisting sagging)

$d_y = h - c_{sag} - D_{tryx} - D_{tryy}/2 = \mathbf{335}$ mm

Materials

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

Characteristic strength of concrete f_{cu} = **35** N/mm^2



Two-way spanning slab
(simple)

MAXIMUM DESIGN MOMENTS

Length of shorter side of slab l_x = **2.200** m

Length of longer side of slab l_y = **9.490** m

Design ultimate load per unit area n_s = **3.5** kN/m^2

Moment coefficients

$\alpha_{sx} = (l_y / l_x)^4 / (8 \times (1+(l_y / l_x)^4)) = \mathbf{0.125}$

$\alpha_{sy} = (l_y / l_x)^2 / (8 \times (1+(l_y / l_x)^4)) = \mathbf{0.007}$

Maximum moments per unit width - simply supported slabs

$m_{sx} = \alpha_{sx} \times n_s \times l_x^2 = \mathbf{2.1}$ kNm/m

$m_{sy} = \alpha_{sy} \times n_s \times l_x^2 = \mathbf{0.1}$ kNm/m

CONCRETE SLAB DESIGN – SAGGING – OUTER LAYER OF STEEL (CL 3.5.4)

Design sagging moment (per m width of slab) m_{sx} = **2.1** kNm/m

Moment Redistribution Factor β_{bx} = **1.0**

Area of reinforcement required

$K_x = abs(m_{sx}) / (d_x^2 \times f_{cu}) = \mathbf{0.000}$

$K'_x = min (0.156 , (0.402 \times (\beta_{bx} - 0.4)) - (0.18 \times (\beta_{bx} - 0.4)^2)) = \mathbf{0.156}$

Outer compression steel not required to resist sagging

Slab requiring outer tension steel only - bars (sagging)

$z_x = min ((0.95 \times d_x),(d_x \times (0.5+\sqrt{(0.25-K_x/0.9)}))) = \mathbf{337}$ mm

Neutral axis depth $x_x = (d_x - z_x) / 0.45 = \mathbf{39}$ mm

Area of tension steel required

$A_{sx\_req} = abs(m_{sx}) / (1/\gamma_{ms} \times f_y \times z_x) = \mathbf{14}$ mm^2/m

Tension steel

Provide 10 dia bars @ 100 centres outer tension steel resisting sagging

$A_{sx\_prov} = A_{sx} = \mathbf{785}$ mm^2/m

Area of outer tension steel provided sufficient 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} = **0.1** kNm/m

Moment Redistribution Factor β_{by} = **1.0**

Area of reinforcement required

$K_y = abs(m_{sy}) / (d_y^2 \times f_{cu}) = \mathbf{0.000}$

$K'_y = min (0.156 , (0.402 \times (\beta_{by} - 0.4)) - (0.18 \times (\beta_{by} - 0.4)^2)) = \mathbf{0.156}$

Inner compression steel not required to resist sagging

Slab requiring inner tension steel only - bars (sagging)

$z_y = min ((0.95 \times d_y),(d_y \times (0.5+\sqrt{(0.25-K_y/0.9)}))) = \mathbf{318}$ mm

Neutral axis depth $x_y = (d_y - z_y) / 0.45 = \mathbf{37}$ mm

Area of tension steel required

$A_{sy\_req} = abs(m_{sy}) / (1/\gamma_{ms} \times f_y \times z_y) = \mathbf{1}$ mm^2/m

Tension steel

Provide 10 dia bars @ 100 centres inner tension steel resisting sagging

$A_{sy\_prov} = A_{sy} = \mathbf{785}$ mm²/m

Area of inner tension steel provided sufficient to resist sagging

Check min and max areas of steel resisting sagging

Total area of concrete $A_c = h = \mathbf{400000}$ mm²/m

Minimum % reinforcement $k = \mathbf{0.13}$ %

$A_{st\_min} = k \times A_c = \mathbf{520}$ mm²/m

$A_{st\_max} = 4\ \% \times A_c = \mathbf{16000}$ mm²/m

Steel defined:

Outer steel resisting sagging $A_{sx\_prov} = \mathbf{785}$ mm²/m

Area of outer steel provided (sagging) OK

Inner steel resisting sagging $A_{sy\_prov} = \mathbf{785}$ mm²/m

Area of inner steel provided (sagging) OK

CONCRETE SLAB DEFLECTION CHECK (CL 3.5.7)

Slab span length $l_x = \mathbf{2.200}$ m

Design ultimate moment in shorter span per m width $m_{sx} = \mathbf{2}$ kNm/m

Depth to outer tension steel $d_x = \mathbf{355}$ mm

Tension steel

Area of outer tension reinforcement provided $A_{sx\_prov} = \mathbf{785}$ mm²/m

Area of tension reinforcement required $A_{sx\_req} = \mathbf{14}$ mm²/m

Moment Redistribution Factor $\beta_{bx} = \mathbf{1.00}$

Modification Factors

Basic span / effective depth ratio (Table 3.9) $ratio_{span\_depth} = \mathbf{20}$

The modification factor for spans in excess of 10m (ref. cl 3.4.6.4) has not been included.

$f_s = 2 \times f_y \times A_{sx\_req} / (3 \times A_{sx\_prov} \times \beta_{bx}) = \mathbf{6.1}$ N/mm²

$factor_{tens} = \min(2, 0.55 + (477\ N/mm^2 - f_s) / (120 \times (0.9\ N/mm^2 + m_{sx} / d_x^2))) = \mathbf{2.000}$

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 $l_{max} = ratio_{span\_depth} \times factor_{tens} \times d_x = \mathbf{14.20}$ m

Check the actual beam span

Actual span/depth ratio $l_x / d_x = \mathbf{6.20}$

Span depth limit $ratio_{span\_depth} \times factor_{tens} = \mathbf{40.00}$

Span/Depth ratio check satisfied

CHECK OF NOMINAL COVER (SAGGING) – (BS8110:PT 1, TABLE 3.4)

Slab thickness $h = \mathbf{400}$ mm

Effective depth to bottom outer tension reinforcement $d_x = \mathbf{355.0}$ mm

Diameter of tension reinforcement $D_x = \mathbf{10}$ mm

Diameter of links $L_{diax} = \mathbf{0}$ mm

Cover to outer tension reinforcement

$c_{tenx} = h - d_x - D_x / 2 = \mathbf{40.0}$ mm

Nominal cover to links steel

$c_{nomx} = c_{tenx} - L_{diax} = \mathbf{40.0}$ mm

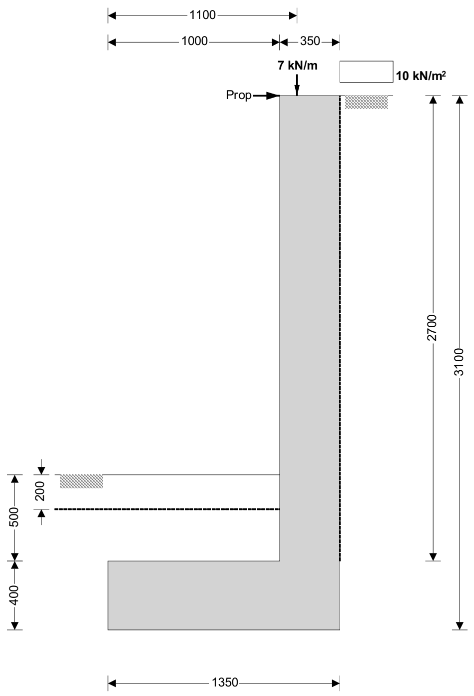
Permissable minimum nominal cover to all reinforcement (Table 3.4)

$c_{min} = \mathbf{35}$ mm

Cover over steel resisting sagging OK

RETAINING WALL ANALYSIS (BS 8002:1994)

TEDDS calculation version 1.2.01.06



Wall details

Retaining wall type	**Cantilever propped at top**
Height of retaining wall stem	h_{stem} = **2700** mm
Thickness of wall stem	t_{wall} = **350** mm
Length of toe	l_{toe} = **1000** mm
Length of heel	l_{heel} = **0** mm
Overall length of base	$l_{base} = l_{toe} + l_{heel} + t_{wall}$ = **1350** mm
Thickness of base	t_{base} = **400** mm
Depth of downstand	d_{ds} = **0** mm
Position of downstand	l_{ds} = **950** mm
Thickness of downstand	t_{ds} = **400** mm
Height of retaining wall	$h_{wall} = h_{stem} + t_{base} + d_{ds}$ = **3100** mm
Depth of cover in front of wall	d_{cover} = **500** mm
Depth of unplanned excavation	d_{exc} = **200** mm
Height of ground water behind wall	h_{water} = **0** mm
Height of saturated fill above base	$h_{sat} = \max(h_{water} - t_{base} - d_{ds}, 0 \text{ mm})$ = **0** 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	$h_{eff} = h_{wall} + l_{heel} \times \tan(\beta)$ = **3100** mm

Retained material details

Mobilisation factor	M = **1.5**
Moist density of retained material	γ_m = **21.0** kN/m^3

Saturated density of retained material	γ_s = **23.0** kN/m^3
Design shear strength	ϕ' = **22.6** deg
Angle of wall friction	δ = **17.3** deg

Base material details

Peat (very variable)

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

$$K_a = \sin(\alpha + \phi')^2 / (\sin(\alpha)^2 \times \sin(\alpha - \delta) \times [1 + \sqrt{(\sin(\phi' + \delta) \times \sin(\phi' - \beta) / (\sin(\alpha - \delta) \times \sin(\alpha + \beta)))}]^2) = \mathbf{0.392}$$

Passive pressure coefficient for base material

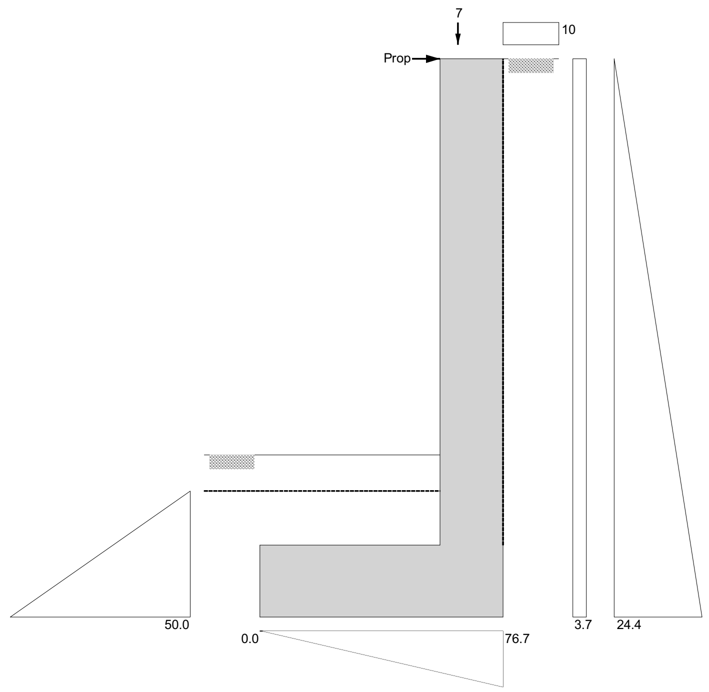
$$K_p = \sin(90 - \phi'_b)^2 / (\sin(90 - \delta_b) \times [1 - \sqrt{(\sin(\phi'_b + \delta_b) \times \sin(\phi'_b) / (\sin(90 + \delta_b)))}]^2) = \mathbf{4.187}$$

At-rest pressure

At-rest pressure for retained material	$K_0 = 1 - \sin(\phi')$ = **0.616**

Loading details

Surcharge load on plan	Surcharge = **10.0** kN/m^2
Applied vertical dead load on wall	W_{dead} = **6.0** kN/m
Applied vertical live load on wall	W_{live} = **1.3** kN/m
Position of applied vertical load on wall	l_{load} = **1100** 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

Tedds	Project: 5 Highfields Grove, London SN6 6HN			Job no.: 2014-207	
	Calcs for: Retaining Wall Type 1 propped PLUS BEAM			Start page no./Revision: 3	
	Calcs by: MD	Calcs date: 13/10/2014	Checked by: JF	Checked date	Approved by	Approved date

Vertical forces on wall

Wall stem $w_{wall} = h_{stem} \times t_{wall} \times \gamma_{wall}$ = **22.3** kN/m

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

Soil in front of wall $w_p = l_{toe} \times d_{cover} \times \gamma_{mb}$ = **9** kN/m

Applied vertical load $W_v = W_{dead} + W_{live}$ = **7.3** kN/m

Total vertical load $W_{total} = w_{wall} + w_{base} + w_p + W_v$ = **51.3** kN/m

Horizontal forces on wall

Surcharge $F_{sur} = K_a \times \cos(90 - \alpha + \delta) \times \text{Surcharge} \times h_{eff}$ = **11.6** kN/m

Moist backfill above water table $F_{m\_a} = 0.5 \times K_a \times \cos(90 - \alpha + \delta) \times \gamma_m \times (h_{eff} - h_{water})^2$ = **37.8** kN/m

Total horizontal load $F_{total} = F_{sur} + F_{m\_a}$ = **49.4** kN/m

Calculate propping force

Passive resistance of soil in front of wall $F_p = 0.5 \times K_p \times \cos(\delta_b) \times (d_{cover} + t_{base} + d_{ds} - d_{exc})^2 \times \gamma_{mb}$ = **17.5** kN/m

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

F_{prop} = **18.1** kN/m

Overturning moments

Surcharge $M_{sur} = F_{sur} \times (h_{eff} - 2 \times d_{ds}) / 2$ = **18** kNm/m

Moist backfill above water table $M_{m\_a} = F_{m\_a} \times (h_{eff} + 2 \times h_{water} - 3 \times d_{ds}) / 3$ = **39.1** kNm/m

Total overturning moment $M_{ot} = M_{sur} + M_{m\_a}$ = **57.1** kNm/m

Restoring moments

Wall stem $M_{wall} = w_{wall} \times (l_{toe} + t_{wall} / 2)$ = **26.2** kNm/m

Wall base $M_{base} = w_{base} \times l_{base} / 2$ = **8.6** kNm/m

Design vertical dead load $M_{dead} = W_{dead} \times l_{load}$ = **6.6** kNm/m

Total restoring moment $M_{rest} = M_{wall} + M_{base} + M_{dead}$ = **41.4** kNm/m

Check bearing pressure

Propping force $M_{prop} = F_{prop} \times (h_{wall} - d_{ds})$ = **56.2** kNm/m

Soil in front of wall $M_{p\_r} = w_p \times l_{toe} / 2$ = **4.5** kNm/m

Design vertical live load $M_{live} = W_{live} \times l_{load}$ = **1.4** kNm/m

Total moment for bearing $M_{total} = M_{rest} - M_{ot} + M_{prop} + M_{p\_r} + M_{live}$ = **46.4** kNm/m

Total vertical reaction $R = W_{total}$ = **51.3** kN/m

Distance to reaction $x_{bar} = M_{total} / R$ = **904** mm

Eccentricity of reaction $e = \text{abs}((l_{base} / 2) - x_{bar})$ = **229** mm

Reaction acts outside middle third of base

Bearing pressure at toe $p_{toe} = 0 \text{ kN/m}^2$ = **0** kN/m²

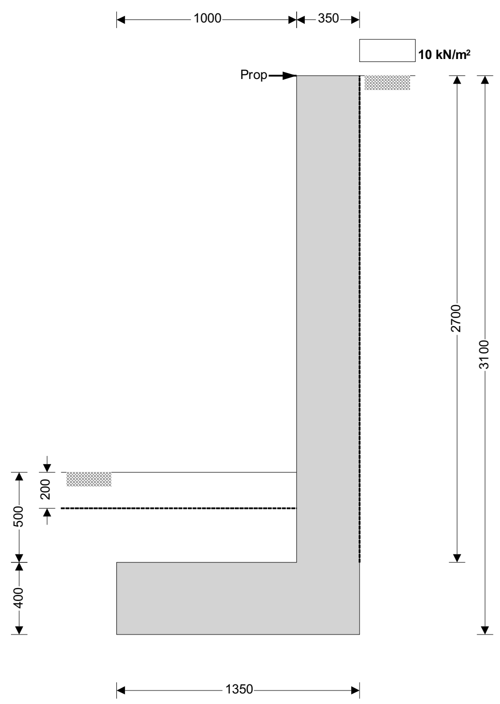
Bearing pressure at heel $p_{heel} = R / (1.5 \times (l_{base} - x_{bar}))$ = **76.7** kN/m²

PASS - Maximum bearing pressure is less than allowable bearing pressure

Tedds	Project: 5 Highfields Grove, London SN6 6HN			Job no.: 2014-207	
	Calcs for: Retaining Wall Type 1 propped			Start page no./Revision: 1	
	Calcs by: MD	Calcs date: 13/10/2014	Checked by: JF	Checked date	Approved by	Approved date

RETAINING WALL ANALYSIS (BS 8002:1994)

TEDDS calculation version 1.2.01.06



Wall details

Retaining wall type **Cantilever propped at top**

Height of retaining wall stem h_{stem} = **2700** mm

Thickness of wall stem t_{wall} = **350** mm

Length of toe l_{toe} = **1000** mm

Length of heel l_{heel} = **0** mm

Overall length of base $l_{base} = l_{toe} + l_{heel} + t_{wall}$ = **1350** mm

Thickness of base t_{base} = **400** mm

Depth of downstand d_{ds} = **0** mm

Position of downstand l_{ds} = **950** mm

Thickness of downstand t_{ds} = **400** mm

Height of retaining wall $h_{wall} = h_{stem} + t_{base} + d_{ds}$ = **3100** mm

Depth of cover in front of wall d_{cover} = **500** mm

Depth of unplanned excavation d_{exc} = **200** mm

Height of ground water behind wall h_{water} = **0** mm

Height of saturated fill above base $h_{sat} = \max(h_{water} - t_{base} - d_{ds}, 0 \text{ mm})$ = **0** mm

Density of wall construction γ_{wall} = **23.6** kN/m³

Density of base construction γ_{base} = **23.6** kN/m³

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 $h_{eff} = h_{wall} + l_{heel} \times \tan(\beta)$ = **3100** mm

Retained material details

Mobilisation factor M = **1.5**

Tedds	Project 5 Highfields Grove, London SN6 6HN			Job no. 2014-207	
	Calcs for Retaining Wall Type 1 propped			Start page no./Revision 2	
	Calcs by MD	Calcs date 13/10/2014	Checked by JF	Checked date	Approved by	Approved date

Moist density of retained material γ_m = **21.0** kN/m^3

Saturated density of retained material γ_s = **22.5** kN/m^3

Design shear strength ϕ' = **22.6** deg

Angle of wall friction δ = **17.3** 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 $P_{bearing}$ = **100** kN/m^2

Using Coulomb theory

Active pressure coefficient for retained material

$K_a = \sin(\alpha + \phi')^2 / (\sin(\alpha)^2 \times \sin(\alpha - \delta) \times [1 + \sqrt{(\sin(\phi' + \delta) \times \sin(\phi' - \beta) / (\sin(\alpha - \delta) \times \sin(\alpha + \beta)))}]^2)$ = **0.392**

Passive pressure coefficient for base material

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

At-rest pressure

At-rest pressure for retained material $K_0 = 1 - \sin(\phi')$ = **0.616**

Loading details

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

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

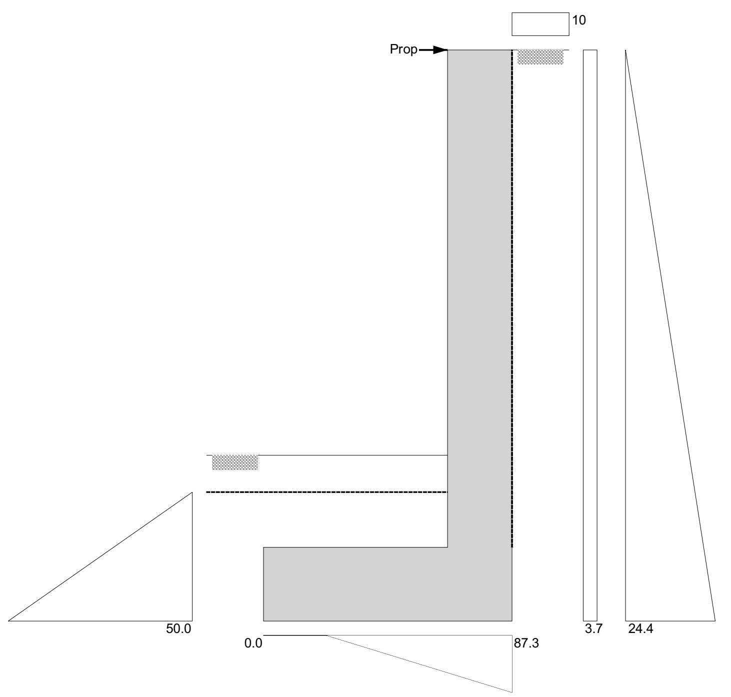
Applied vertical live load on wall W_{live} = **0.0** kN/m

Position of applied vertical load on wall l_{load} = **0** 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

Tedds	Project 5 Highfields Grove, London SN6 6HN			Job no. 2014-207	
	Calcs for Retaining Wall Type 1 propped			Start page no./Revision 3	
	Calcs by MD	Calcs date 13/10/2014	Checked by JF	Checked date	Approved by	Approved date

Vertical forces on wall

Wall stem $w_{wall} = h_{stem} \times t_{wall} \times \gamma_{wall}$ = **22.3** kN/m

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

Soil in front of wall $w_p = l_{toe} \times d_{cover} \times \gamma_{mb}$ = **9** kN/m

Total vertical load $W_{total} = w_{wall} + w_{base} + w_p$ = **44** kN/m

Horizontal forces on wall

Surcharge $F_{sur} = K_a \times \cos(90 - \alpha + \delta) \times Surcharge \times h_{eff}$ = **11.6** kN/m

Moist backfill above water table $F_{m\_a} = 0.5 \times K_a \times \cos(90 - \alpha + \delta) \times \gamma_m \times (h_{eff} - h_{water})^2$ = **37.8** kN/m

Total horizontal load $F_{total} = F_{sur} + F_{m\_a}$ = **49.4** kN/m

Calculate propping force

Passive resistance of soil in front of wall $F_p = 0.5 \times K_p \times \cos(\delta_b) \times (d_{cover} + t_{base} + d_{ds} - d_{exc})^2 \times \gamma_{mb}$ = **17.5** kN/m

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

F_{prop} = **20.1** kN/m

Overturning moments

Surcharge $M_{sur} = F_{sur} \times (h_{eff} - 2 \times d_{ds}) / 2$ = **18** kNm/m

Moist backfill above water table $M_{m\_a} = F_{m\_a} \times (h_{eff} + 2 \times h_{water} - 3 \times d_{ds}) / 3$ = **39.1** kNm/m

Total overturning moment $M_{ot} = M_{sur} + M_{m\_a}$ = **57.1** kNm/m

Restoring moments

Wall stem $M_{wall} = w_{wall} \times (l_{toe} + t_{wall} / 2)$ = **26.2** kNm/m

Wall base $M_{base} = w_{base} \times l_{base} / 2$ = **8.6** kNm/m

Total restoring moment $M_{rest} = M_{wall} + M_{base}$ = **34.8** kNm/m

Check bearing pressure

Propping force $M_{prop} = F_{prop} \times (h_{wall} - d_{ds})$ = **62.4** kNm/m

Soil in front of wall $M_{p\_r} = w_p \times l_{toe} / 2$ = **4.5** kNm/m

Total moment for bearing $M_{total} = M_{rest} - M_{ot} + M_{prop} + M_{p\_r}$ = **44.7** kNm/m

Total vertical reaction $R = W_{total}$ = **44.0** kN/m

Distance to reaction $x_{bar} = M_{total} / R$ = **1014** mm

Eccentricity of reaction $e = abs((l_{base} / 2) - x_{bar})$ = **339** mm

Reaction acts outside middle third of base

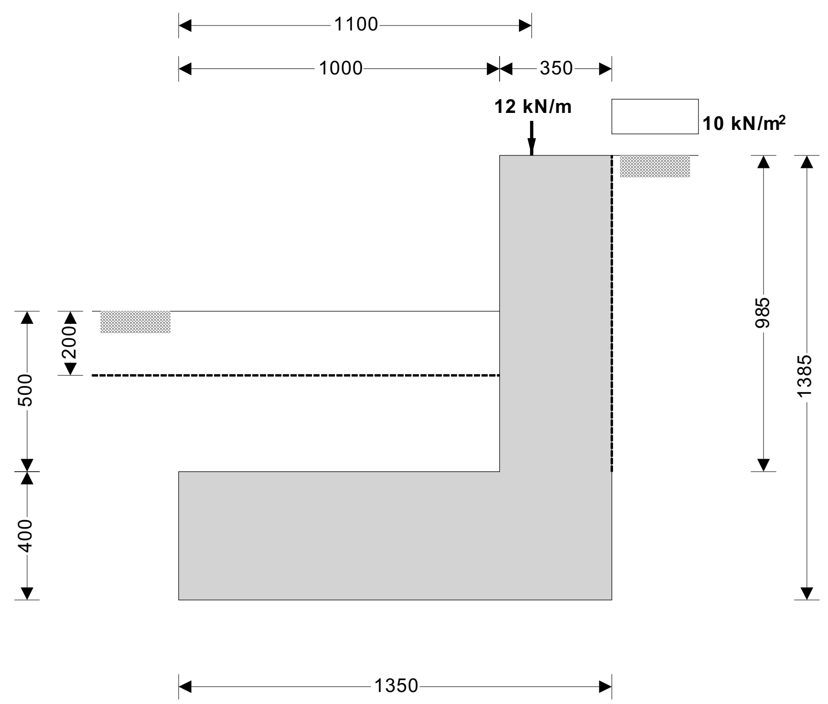
Bearing pressure at toe $p_{toe} = 0\ kN/m^2$ = **0** kN/m^2

Bearing pressure at heel $p_{heel} = R / (1.5 \times (l_{base} - x_{bar}))$ = **87.3** kN/m^2

PASS - Maximum bearing pressure is less than allowable bearing pressure

RETAINING WALL ANALYSIS (BS 8002:1994)

TEDDS calculation version 1.2.01.06



Wall details

Retaining wall type	**Unpropped cantilever**
Height of retaining wall stem	h_{stem} = **985** mm
Thickness of wall stem	t_{wall} = **350** mm
Length of toe	l_{toe} = **1000** mm
Length of heel	l_{heel} = **0** mm
Overall length of base	$l_{base} = l_{toe} + l_{heel} + t_{wall}$ = **1350** mm
Thickness of base	t_{base} = **400** mm
Depth of downstand	d_{ds} = **0** mm
Position of downstand	l_{ds} = **950** mm
Thickness of downstand	t_{ds} = **400** mm
Height of retaining wall	$h_{wall} = h_{stem} + t_{base} + d_{ds}$ = **1385** mm
Depth of cover in front of wall	d_{cover} = **500** mm
Depth of unplanned excavation	d_{exc} = **200** mm
Height of ground water behind wall	h_{water} = **0** mm
Height of saturated fill above base	$h_{sat} = \max(h_{water} - t_{base} - d_{ds}, 0 \text{ mm})$ = **0** 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	$h_{eff} = h_{wall} + l_{heel} \times \tan(\beta)$ = **1385** mm

Retained material details

Mobilisation factor	M = **1.5**

Moist density of retained material	γ_m = **21.0** kN/m^3
Saturated density of retained material	γ_s = **23.0** kN/m^3
Design shear strength	ϕ' = **22.6** deg
Angle of wall friction	δ = **17.3** 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	$P_{bearing}$ = **100** kN/m^2

Using Coulomb theory

Active pressure coefficient for retained material

$$K_a = \sin(\alpha + \phi')^2 / (\sin(\alpha)^2 \times \sin(\alpha - \delta) \times [1 + \sqrt{(\sin(\phi' + \delta) \times \sin(\phi' - \beta) / (\sin(\alpha - \delta) \times \sin(\alpha + \beta)))}]^2) = \mathbf{0.392}$$

Passive pressure coefficient for base material

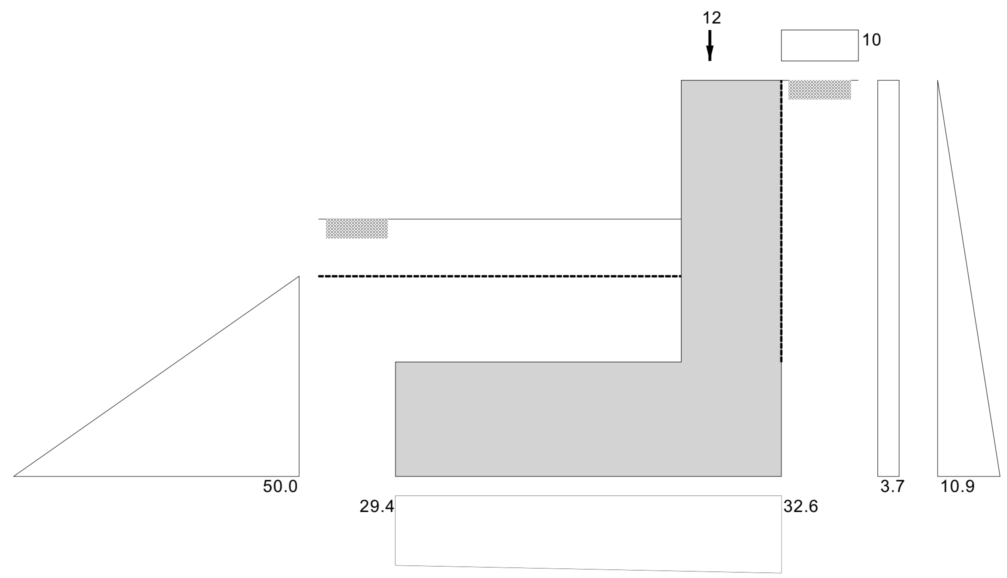
$$K_p = \sin(90 - \phi'_b)^2 / (\sin(90 - \delta_b) \times [1 - \sqrt{(\sin(\phi'_b + \delta_b) \times \sin(\phi'_b) / (\sin(90 + \delta_b)))}]^2) = \mathbf{4.187}$$

At-rest pressure

At-rest pressure for retained material	$K_0 = 1 - \sin(\phi')$ = **0.616**

Loading details

Surcharge load on plan	Surcharge = **10.0** kN/m^2
Applied vertical dead load on wall	W_{dead} = **9.3** kN/m
Applied vertical live load on wall	W_{live} = **2.6** kN/m
Position of applied vertical load on wall	l_{load} = **1100** 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	$w_{wall} = h_{stem} \times t_{wall} \times \gamma_{wall}$ = **8.1** kN/m
Wall base	$w_{base} = l_{base} \times t_{base} \times \gamma_{base}$ = **12.7** kN/m

Soil in front of wall	$w_p = l_{toe} \times d_{cover} \times \gamma_{mb}$ = **9** kN/m
Applied vertical load	$W_v = W_{dead} + W_{live}$ = **11.9** kN/m
Total vertical load	$W_{total} = w_{wall} + w_{base} + w_p + W_v$ = **41.8** kN/m

Horizontal forces on wall

Surcharge	$F_{sur} = K_a \times \cos(90 - \alpha + \delta) \times Surcharge \times h_{eff}$ = **5.2** kN/m
Moist backfill above water table	$F_{m\_a} = 0.5 \times K_a \times \cos(90 - \alpha + \delta) \times \gamma_m \times (h_{eff} - h_{water})^2$ = **7.5** kN/m
Total horizontal load	$F_{total} = F_{sur} + F_{m\_a}$ = **12.7** kN/m

Calculate stability against sliding

Passive resistance of soil in front of wall	$F_p = 0.5 \times K_p \times \cos(\delta_b) \times (d_{cover} + t_{base} + d_{ds} - d_{exc})^2 \times \gamma_{mb}$ = **17.5** kN/m
Resistance to sliding	$F_{res} = F_p + (W_{total} - w_p - W_{live}) \times \tan(\delta_b)$ = **27.7** kN/m

PASS - Resistance force is greater than sliding force

Overturning moments

Surcharge	$M_{sur} = F_{sur} \times (h_{eff} - 2 \times d_{ds}) / 2$ = **3.6** kNm/m
Moist backfill above water table	$M_{m\_a} = F_{m\_a} \times (h_{eff} + 2 \times h_{water} - 3 \times d_{ds}) / 3$ = **3.5** kNm/m
Total overturning moment	$M_{ot} = M_{sur} + M_{m\_a}$ = **7.1** kNm/m

Restoring moments

Wall stem	$M_{wall} = w_{wall} \times (l_{toe} + t_{wall} / 2)$ = **9.6** kNm/m
Wall base	$M_{base} = w_{base} \times l_{base} / 2$ = **8.6** kNm/m
Design vertical dead load	$M_{dead} = W_{dead} \times l_{load}$ = **10.2** kNm/m
Total restoring moment	$M_{rest} = M_{wall} + M_{base} + M_{dead}$ = **28.4** kNm/m

Check stability against overturning

Total overturning moment	M_{ot} = **7.1** kNm/m
Total restoring moment	M_{rest} = **28.4** kNm/m

PASS - Restoring moment is greater than overturning moment

Check bearing pressure

Soil in front of wall	$M_{p\_r} = w_p \times l_{toe} / 2$ = **4.5** kNm/m
Design vertical live load	$M_{live} = W_{live} \times l_{load}$ = **2.9** kNm/m
Total moment for bearing	$M_{total} = M_{rest} - M_{ot} + M_{p\_r} + M_{live}$ = **28.7** kNm/m
Total vertical reaction	$R = W_{total}$ = **41.8** kN/m
Distance to reaction	$x_{bar} = M_{total} / R$ = **687** mm
Eccentricity of reaction	$e = abs((l_{base} / 2) - x_{bar})$ = **12** mm

Reaction acts within middle third of base

Bearing pressure at toe	$p_{toe} = (R / l_{base}) - (6 \times R \times e / l_{base}^2)$ = **29.4** kN/m^2
Bearing pressure at heel	$p_{heel} = (R / l_{base}) + (6 \times R \times e / l_{base}^2)$ = **32.6** kN/m^2

PASS - Maximum bearing pressure is less than allowable bearing pressure

RETAINING WALL ANALYSIS

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

Tedds calculation version 2.2.01

Retaining wall details

Stem type	Propped cantilever
Stem height	h_{stem} = **2900** mm
Prop height	h_{prop} = **2900** mm
Stem thickness	t_{stem} = **350** mm
Angle to rear face of stem	α = **90** deg
Stem density	γ_{stem} = **25** kN/m^3
Toe length	l_{toe} = **1000** mm
Base thickness	t_{base} = **350** mm
Base density	γ_{base} = **25** kN/m^3
Height of retained soil	h_{ret} = **2900** mm
Angle of soil surface	β = **0** deg
Depth of cover	d_{cover} = **0** mm
Depth of excavation	d_{exc} = **200** mm

Retained soil properties

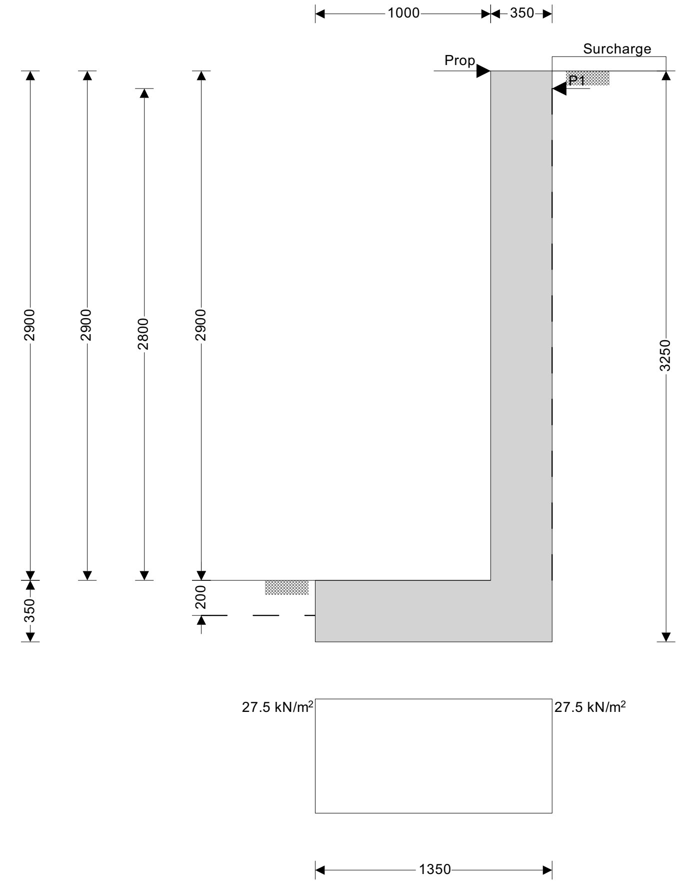
Soil type	Medium dense well graded sand and gravel
Moist density	γ_{mr} = **20** kN/m^3
Saturated density	γ_{sr} = **22.3** kN/m^3
Characteristic effective shear resistance angle	$\phi'_{r.k}$ = **30** deg
Characteristic wall friction angle	$\delta_{r.k}$ = **15** deg

Base soil properties

Soil type	Organic clay
Moist density	γ_{mb} = **15** kN/m^3
Characteristic cohesion	$c'_{b.k}$ = **0** kN/m^2
Characteristic effective shear resistance angle	$\phi'_{b.k}$ = **18** deg
Characteristic wall friction angle	$\delta_{b.k}$ = **9** deg
Characteristic base friction angle	$\delta_{bb.k}$ = **12** deg

Loading details

Variable surcharge load	$Surcharge_Q$ = **10** kN/m^2
Horizontal line load at 2800 mm	P_{G1} = **20** kN/m



Calculate retaining wall geometry

Base length $l_{base} = l_{toe} + t_{stem} = \mathbf{1350}$ mm

Moist soil height $h_{moist} = h_{soil} = \mathbf{2900}$ mm

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

- Distance to vertical component $x_{sur\_v} = l_{base} - l_{heel} / 2 = \mathbf{1350}$ mm

Effective height of wall $h_{eff} = h_{base} + d_{cover} + h_{ret} = \mathbf{3250}$ mm

- Distance to horizontal component $x_{sur\_h} = h_{eff} / 2 = \mathbf{1625}$ mm

Area of wall stem $A_{stem} = h_{stem} \times t_{stem} = \mathbf{1.015}$ m^2

- Distance to vertical component $x_{stem} = l_{toe} + t_{stem} / 2 = \mathbf{1175}$ mm

Area of wall base $A_{base} = l_{base} \times t_{base} = \mathbf{0.473}$ m^2

- Distance to vertical component $x_{base} = l_{base} / 2 = \mathbf{675}$ mm

Partial factors on actions - Table A.3 - Combination 1

Permanent unfavourable action $\gamma_G = \mathbf{1.35}$

Permanent favourable action $\gamma_{Gf} = \mathbf{1.00}$

Variable unfavourable action $\gamma_Q = \mathbf{1.50}$

Variable favourable action $\gamma_{Qf} = \mathbf{0.00}$

Partial factors for soil parameters – Table A.4 - Combination 1

Angle of shearing resistance $\gamma_{\phi'} = \mathbf{1.00}$

Effective cohesion $\gamma_{c'} = \mathbf{1.00}$

Weight density $\gamma_{\gamma} = \mathbf{1.00}$

Retained soil properties

Design effective shear resistance angle $\phi'_{r.d} = \text{atan}(\tan(\phi'_{r.k}) / \gamma_{\phi'}) = \mathbf{30}$ deg

Design wall friction angle $\delta_{r.d} = \text{atan}(\tan(\delta_{r.k}) / \gamma_{\phi'}) = \mathbf{15}$ deg

Base soil properties

Design effective shear resistance angle $\phi'_{b.d} = \text{atan}(\tan(\phi'_{b.k}) / \gamma_{\phi'}) = \mathbf{18}$ deg

Design wall friction angle $\delta_{b.d} = \text{atan}(\tan(\delta_{b.k}) / \gamma_{\phi'}) = \mathbf{9}$ deg

Design base friction angle $\delta_{bb.d} = \text{atan}(\tan(\delta_{bb.k}) / \gamma_{\phi'}) = \mathbf{12}$ deg

Design effective cohesion $c'_{b.d} = c'_{b.k} / \gamma_{c'} = \mathbf{0}$ kN/m^2

Using Coulomb theory

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

Passive pressure coefficient $K_P = \sin(90 - \phi'_{b.d})^2 / (\sin(90 + \delta_{b.d}) \times [1 - \sqrt{[}\sin(\phi'_{b.d} + \delta_{b.d}) \times \sin(\phi'_{b.d}) / (\sin(90 + \delta_{b.d}))]]^2) = \mathbf{2.359}$

Bearing pressure check

Vertical forces on wall

Wall stem $F_{stem} = \gamma_G \times A_{stem} \times \gamma_{stem} = \mathbf{34.3}$ kN/m

Wall base $F_{base} = \gamma_G \times A_{base} \times \gamma_{base} = \mathbf{15.9}$ kN/m

Total $F_{total\_v} = F_{stem} + F_{base} = \mathbf{50.2}$ kN/m

Horizontal forces on wall

Surcharge load $F_{sur\_h} = K_A \times \cos(\delta_{r.d}) \times \gamma_Q \times \text{Surcharge}_Q \times h_{eff} = \mathbf{14.2}$ kN/m

Line loads $F_{P\_h} = \gamma_G \times P_{G1} = \mathbf{27}$ kN/m

Moist retained soil $F_{moist\_h} = \gamma_G \times K_A \times \cos(\delta_{r.d}) \times \gamma_{mr} \times h_{eff}^2 / 2 = \mathbf{41.5}$ kN/m

Total $F_{total\_h} = F_{moist\_h} + F_{sur\_h} + F_{P\_h} = \mathbf{82.7}$ kN/m

Moments on wall

Wall stem $M_{stem} = F_{stem} \times x_{stem} = \mathbf{40.3}$ kNm/m

Wall base $M_{base} = F_{base} \times x_{base} = \mathbf{10.8}$ kNm/m

Surcharge load $M_{sur} = -F_{sur\_h} \times x_{sur\_h} = \mathbf{-23.1}$ kNm/m

Line loads $M_P = -\gamma_G \times P_{G1} \times (p_1 + t_{base}) = \mathbf{-85.1}$ kNm/m

Moist retained soil $M_{moist} = -F_{moist\_h} \times x_{moist\_h} = \mathbf{-45}$ kNm/m

Total $M_{total} = M_{stem} + M_{base} + M_{moist} + M_{pass} + M_{sur} + M_P = \mathbf{-102.1}$ kNm/m

Check bearing pressure

Maximum friction force $F_{friction\_max} = F_{total\_v} \times \tan(\delta_{bb.d}) = \mathbf{10.7}$ kN/m

Maximum base soil resistance $F_{pass\_h\_max} = \gamma_{Gf} \times K_P \times \cos(\delta_{b.d}) \times \gamma_{mb} \times (d_{cover} + h_{base})^2 / 2 = \mathbf{2.1}$ kN/m

Base soil resistance $F_{pass\_h} = \min(\max((M_{total} + F_{total\_h} \times (h_{prop} + t_{base}) + F_{friction\_max} \times (h_{prop} + t_{base}) - F_{total\_v} \times l_{base} / 2) / (x_{pass\_h} - h_{prop} - t_{base}), 0\text{ kN/m}), F_{pass\_h\_max}) = \mathbf{0}$ kN/m

Propping force $F_{prop\_stem} = \min((F_{total\_v} \times l_{base} / 2 - M_{total}) / (h_{prop} + t_{base}), F_{total\_h}) = \mathbf{41.8}$ kN/m

Friction force $F_{friction} = F_{total\_h} - F_{pass\_h} - F_{prop\_stem} = \mathbf{40.9}$ kN/m

Moment from propping force $M_{prop} = F_{prop\_stem} \times (h_{prop} + t_{base}) = \mathbf{136}$ kNm/m

Distance to reaction $\bar{x} = (M_{total} + M_{prop}) / F_{total\_v} = \mathbf{675}$ mm

Eccentricity of reaction $e = \bar{x} - l_{base} / 2 = \mathbf{0}$ mm

Loaded length of base $l_{load} = l_{base} = \mathbf{1350}$ mm

Bearing pressure at toe $q_{toe} = F_{total\_v} / l_{base} = \mathbf{37.2}$ kN/m^2

Bearing pressure at heel	$q_{heel} = F_{total\_v} / l_{base} = \mathbf{37.2}$ kN/m^2
Effective overburden pressure	$q = (t_{base} + d_{cover}) \times \gamma_{mb} = \mathbf{5.3}$ kN/m^2
Design effective overburden pressure	$q' = q / \gamma_{\gamma} = \mathbf{5.3}$ kN/m^2
Bearing resistance factors	$N_q = Exp(\pi \times \tan(\phi'_{b.d})) \times (\tan(45 \text{ deg} + \phi'_{b.d} / 2))^2 = \mathbf{5.258}$
	$N_c = (N_q - 1) \times \cot(\phi'_{b.d}) = \mathbf{13.104}$
	$N_{\gamma} = 2 \times (N_q - 1) \times \tan(\phi'_{b.d}) = \mathbf{2.767}$
Foundation shape factors	$s_q = 1$
	$s_{\gamma} = 1$
	$s_c = 1$
Load inclination factors	$H = F_{total\_h} - F_{prop\_stem} - F_{friction} = \mathbf{0}$ kN/m
	$V = F_{total\_v} = \mathbf{50.2}$ kN/m
	$m = 2$
	$i_q = [1 - H / (V + l_{load} \times c'_{b.d} \times \cot(\phi'_{b.d}))]^m = \mathbf{1}$
	$i_{\gamma} = [1 - H / (V + l_{load} \times c'_{b.d} \times \cot(\phi'_{b.d}))]^{(m + 1)} = \mathbf{1}$
	$i_c = i_q - (1 - i_q) / (N_c \times \tan(\phi'_{b.d})) = \mathbf{1}$
Net ultimate bearing capacity	$n_f = c'_{b.d} \times N_c \times s_c \times i_c + q' \times N_q \times s_q \times i_q + 0.5 \times \gamma_{mb} \times l_{load} \times N_{\gamma} \times s_{\gamma} \times i_{\gamma} = \mathbf{55.6}$ kN/m^2
Factor of safety	$FoS_{bp} = n_f / \max(q_{toe}, q_{heel}) = \mathbf{1.496}$

PASS - Allowable bearing pressure exceeds maximum applied bearing pressure

Partial factors on actions - Table A.3 - Combination 2

Permanent unfavourable action	$\gamma_G = \mathbf{1.00}$
Permanent favourable action	$\gamma_{Gf} = \mathbf{1.00}$
Variable unfavourable action	$\gamma_Q = \mathbf{1.30}$
Variable favourable action	$\gamma_{Qf} = \mathbf{0.00}$

Partial factors for soil parameters – Table A.4 - Combination 2

Angle of shearing resistance	$\gamma_{\phi'} = \mathbf{1.25}$
Effective cohesion	$\gamma_{c'} = \mathbf{1.25}$
Weight density	$\gamma_{\gamma} = \mathbf{1.00}$

Retained soil properties

Design effective shear resistance angle	$\phi'_{r.d} = \text{atan}(\tan(\phi'_{r.k}) / \gamma_{\phi'}) = \mathbf{24.8}$ deg
Design wall friction angle	$\delta_{r.d} = \text{atan}(\tan(\delta_{r.k}) / \gamma_{\phi'}) = \mathbf{12.1}$ deg

Base soil properties

Design effective shear resistance angle	$\phi'_{b.d} = \text{atan}(\tan(\phi'_{b.k}) / \gamma_{\phi'}) = \mathbf{14.6}$ deg
Design wall friction angle	$\delta_{b.d} = \text{atan}(\tan(\delta_{b.k}) / \gamma_{\phi'}) = \mathbf{7.2}$ deg
Design base friction angle	$\delta_{bb.d} = \text{atan}(\tan(\delta_{bb.k}) / \gamma_{\phi'}) = \mathbf{9.7}$ deg
Design effective cohesion	$c'_{b.d} = c'_{b.k} / \gamma_{c'} = \mathbf{0}$ kN/m^2

Using Coulomb theory

Active pressure coefficient	$K_A = \sin(\alpha + \phi'_{r.d})^2 / (\sin(\alpha)^2 \times \sin(\alpha - \delta_{r.d}) \times [1 + \sqrt{[\sin(\phi'_{r.d} + \delta_{r.d}) \times \sin(\phi'_{r.d} - \beta) / (\sin(\alpha - \delta_{r.d}) \times \sin(\alpha + \beta))]}]^2) = \mathbf{0.371}$
Passive pressure coefficient	$K_P = \sin(90 - \phi'_{b.d})^2 / (\sin(90 + \delta_{b.d}) \times [1 - \sqrt{[\sin(\phi'_{b.d} + \delta_{b.d}) \times \sin(\phi'_{b.d}) / (\sin(90 + \delta_{b.d}))]}]^2) = \mathbf{1.965}$

Bearing pressure check

Vertical forces on wall

Wall stem	$F_{stem} = \gamma_G \times A_{stem} \times \gamma_{stem} = \mathbf{25.4}$ kN/m

Wall base	$F_{base} = \gamma_G \times A_{base} \times \gamma_{base} = \mathbf{11.8}$ kN/m
Total	$F_{total\_v} = F_{stem} + F_{base} = \mathbf{37.2}$ kN/m

Horizontal forces on wall

Surcharge load	$F_{sur\_h} = K_A \times \cos(\delta_{r.d}) \times \gamma_Q \times \text{Surcharge}_Q \times h_{eff} = \mathbf{15.3}$ kN/m
Line loads	$F_{P\_h} = \gamma_G \times P_{G1} = \mathbf{20}$ kN/m
Moist retained soil	$F_{moist\_h} = \gamma_G \times K_A \times \cos(\delta_{r.d}) \times \gamma_{mr} \times h_{eff}^2 / 2 = \mathbf{38.3}$ kN/m
Total	$F_{total\_h} = F_{moist\_h} + F_{sur\_h} + F_{P\_h} = \mathbf{73.6}$ kN/m

Moments on wall

Wall stem	$M_{stem} = F_{stem} \times x_{stem} = \mathbf{29.8}$ kNm/m
Wall base	$M_{base} = F_{base} \times x_{base} = \mathbf{8}$ kNm/m
Surcharge load	$M_{sur} = -F_{sur\_h} \times x_{sur\_h} = \mathbf{-24.9}$ kNm/m
Line loads	$M_P = -\gamma_G \times P_{G1} \times (p_1 + t_{base}) = \mathbf{-63}$ kNm/m
Moist retained soil	$M_{moist} = -F_{moist\_h} \times x_{moist\_h} = \mathbf{-41.5}$ kNm/m
Total	$M_{total} = M_{stem} + M_{base} + M_{moist} + M_{pass} + M_{sur} + M_P = \mathbf{-91.6}$ kNm/m

Check bearing pressure

Maximum friction force	$F_{friction\_max} = F_{total\_v} \times \tan(\delta_{bb.d}) = \mathbf{6.3}$ kN/m
Maximum base soil resistance	$F_{pass\_h\_max} = \gamma_{Gf} \times K_P \times \cos(\delta_{b.d}) \times \gamma_{mb} \times (d_{cover} + h_{base})^2 / 2 = \mathbf{1.8}$ kN/m
Base soil resistance	$F_{pass\_h} = \min(\max((M_{total} + F_{total\_h} \times (h_{prop} + t_{base}) + F_{friction\_max} \times (h_{prop} + t_{base}) - F_{total\_v} \times l_{base} / 2) / (x_{pass\_h} - h_{prop} - t_{base}), 0 \text{ kN/m}), F_{pass\_h\_max}) = \mathbf{0}$ kN/m
Propping force	$F_{prop\_stem} = \min((F_{total\_v} \times l_{base} / 2 - M_{total}) / (h_{prop} + t_{base}), F_{total\_h}) = \mathbf{35.9}$ kN/m
Friction force	$F_{friction} = F_{total\_h} - F_{pass\_h} - F_{prop\_stem} = \mathbf{37.7}$ kN/m
Moment from propping force	$M_{prop} = F_{prop\_stem} \times (h_{prop} + t_{base}) = \mathbf{116.7}$ kNm/m
Distance to reaction	$\bar{x} = (M_{total} + M_{prop}) / F_{total\_v} = \mathbf{675}$ mm
Eccentricity of reaction	$e = \bar{x} - l_{base} / 2 = \mathbf{0}$ mm
Loaded length of base	$l_{load} = l_{base} = \mathbf{1350}$ mm
Bearing pressure at toe	$q_{toe} = F_{total\_v} / l_{base} = \mathbf{27.5}$ kN/m^2
Bearing pressure at heel	$q_{heel} = F_{total\_v} / l_{base} = \mathbf{27.5}$ kN/m^2
Effective overburden pressure	$q = (t_{base} + d_{cover}) \times \gamma_{mb} = \mathbf{5.3}$ kN/m^2
Design effective overburden pressure	$q' = q / \gamma_{\gamma} = \mathbf{5.3}$ kN/m^2
Bearing resistance factors	$N_q = Exp(\pi \times \tan(\phi'_{b.d})) \times (\tan(45 \text{ deg} + \phi'_{b.d} / 2))^2 = \mathbf{3.784}$
	$N_c = (N_q - 1) \times \cot(\phi'_{b.d}) = \mathbf{10.711}$
	$N_{\gamma} = 2 \times (N_q - 1) \times \tan(\phi'_{b.d}) = \mathbf{1.447}$
Foundation shape factors	$s_q = 1$
	$s_{\gamma} = 1$
	$s_c = 1$
Load inclination factors	$H = F_{total\_h} - F_{prop\_stem} - F_{friction} = \mathbf{0}$ kN/m
	$V = F_{total\_v} = \mathbf{37.2}$ kN/m
	$m = 2$
	$i_q = [1 - H / (V + l_{load} \times c'_{b.d} \times \cot(\phi'_{b.d}))]^m = \mathbf{1}$
	$i_{\gamma} = [1 - H / (V + l_{load} \times c'_{b.d} \times \cot(\phi'_{b.d}))]^{(m + 1)} = \mathbf{1}$
	$i_c = i_q - (1 - i_q) / (N_c \times \tan(\phi'_{b.d})) = \mathbf{1}$
Net ultimate bearing capacity	$n_f = c'_{b.d} \times N_c \times s_c \times i_c + q' \times N_q \times s_q \times i_q + 0.5 \times \gamma_{mb} \times l_{load} \times N_{\gamma} \times s_{\gamma} \times i_{\gamma} = \mathbf{34.5}$ kN/m^2

Tedds	Project 5 Highfields Grove, London SN6 6HN					Job no. 2014-207	
	Calcs for Retaining Wall Type 3 by extg foundations					Start page no./Revision 6	
	Calcs by MD	Calcs date 13/10/2014	Checked by JF	Checked date		Approved by	Approved date

Factor of safety $FoS_{bp} = n_f / \max(q_{toe}, q_{heel})$ = **1.253**

PASS - Allowable bearing pressure exceeds maximum applied bearing pressure

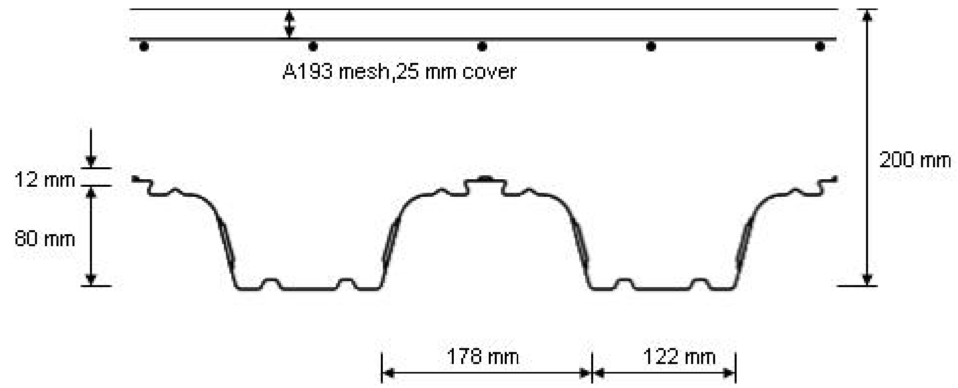
SMD structural metal decks ltd
Engineers in Steel Decking Systems

BS5950

Client		**Calculation By**	Marcin Dylowski
Project Name	5 Highfields Grove, London SN6	**Company Name**	Elite Designers Ltd
Project Ref.	2014-207	**Date**	13/10/2014
Slab Ref.	Basement slab	**Location**	
Comments			
Revision	00		

1 Overall Summary

Construction Stage	**PASS**	**Max. UF**	**0.89**
Composite Stage	**PASS**	**Max. UF**	**0.37**
Fire Stage	**PASS**	**Max. UF**	**0.00**



2 Input Parameters

2.1 Deck/Span Properties

Deck Type	TR80+, 1.2mm, S350	Span	3.900m
Span Type	Single	Support Width	100mm
Number of Props	N/A	Prop Width	N/A

2.2 Slab Properties

Slab Depth	200mm	Concrete Type	C30
Slab Type	Single	Wet/Dry Density	2400/2350 kg/m³
Concrete Volume	0.156m³/m²	Modular Ratio	12.62
Mesh Design Method	User Defined	Bar Design Method	User Defined
Mesh Yield Strength	500 N/mm²	Bar Yield Strength	500 N/mm²

2.3 Loadings

	SLS (kN/m²)	**ULS (kN/m²)**
Concrete Weight (wet)	3.67	5.14
Deck + Reinforcement	0.18	0.25
Additional slab due to ponding	0.40	0.55
Total Slab (Construction Stage)	4.25	5.94
Construction Load	1.50	2.40
Screed	0.98	1.37
Imposed Load	1.50	2.40
Ceilings + Services	0.50	0.70
Finishes	0.47	0.66
Partitions	1.00	1.60
Total Selfweight	4.16	5.83

2.4 Concentrated Loading

Name	**Type**	**Live** (kN/(m))	**Dead** (kN/(m))	**Finishes** (mm)	**Width** (mm)	**Location** (mm)	**Length** (mm)	**Start** (mm)	**Finish** (mm)

No concentrated loading

3 Design Criteria

Fire Period	0.5 hrs	Fire Analysis Method	Fire Engineering
Proportion of Live Load	0 %	Fire Load Factor	0.80
Live Load Factor	1.60	Dead Load Factor	1.40
Superimposed Load Factor	1.40		

4 Construction Stage

	Applied	Capacity/Limit	Unity Factor
Web Shear	16.19 kN/m	101.56 kN/m	0.16
Web Crushing	16.19 kN/m	34.43 kN/m	0.47
Bending (Sagging)	15.70 kNm/m	18.73 kNm/m	0.84
Deflection	26.4 mm	29.8 mm	0.89
	(Deflection limit is the lesser of Span/130 and 30mm)		

5 Composite Stage

Average Composite Inertia 36199599 mm^4

	Applied	Capacity/Limit	Unity Factor
Horizontal Shear	13.13 kN/m	51.69 kN/m	0.25
Vertical Shear	24.49 kN/m	67.04 kN/m	0.37
Bending Resistance	23.87 kNm/m	81.78 kNm/m	0.29
Imposed Load Deflection	1.0 mm	11.1 mm	0.09
	(Deflection limit is the lesser of Span/350 and 20 mm)		
Total Load Deflection	1.8 mm	15.6 mm	0.12
	(Deflection limit is the lesser of Span/250 and 20 mm)		

6 Fire Stage

	Applied	Capacity/Limit	Unity Factor
Moment Resistance	15.54 kNm/m	0.00 kNm/m	0.00

Appendix C: Geotechnical & Services

- **Basement Impact assessment**
- **Borehole log**

15 Highfields Grove, SN6 6HN

Basement Impact Assessment

Job No. 2012-207 October 2014, Rev 00.

PROJECT INFORMATION

Client: Safran Holdings Ltd

Site Address: 15 Highfields Grove, London SN6 6HN

Nature of Works: Basement Impact Assessment for the construction of a basement to the proposed property at 15 Highfields Grove.

Contents

Introduction: 2
Stage 1 - Screening: 3
Description of Site & Works: 3
Stage 2 - Scoping: 4
Subterranean Flow Screening issues: 4
Slope Stability Screening issues: 4
Surface Flow screening issues: 5
Conceptual ground model: 5
Stage 3 - Site Investigation & Desk Study: 6
Ground Conditions: 6
Monitoring: 7
Stage 4 - Impact Assessment: 7
General comments: 7
Soil Site Investigation Report 8

Introduction:

This report sets out the design philosophy for the proposed basement floor construction and should be read in conjunction with the Method Statement and the structural detail drawings attached in appendix A and calculations attached in appendix B which detail both the temporary and permanent design stages of the subterranean development. The aim of the Basement Impact Assessment is to ensure safe and proper construction of the proposed works and ensure no adverse affects to existing or neighbouring structures.

Stage 1 - Screening:

Preliminary assessment of the land stability suggested no potential issue but it was felt that further investigation was necessary. It was felt also the any potential effects on surface water flows required further investigation.

From the screening process the following items were identified as requiring further investigation:

1. The site proximity to local watercourses is unknown and needs to be investigated further,
2. The impact in the change of hard standing will needs to be investigated
3. The drainage of any additional hard standing to be discussed.
4. The level at which London clay is met is to be established.
5. Trees around the development will need to be assessed.
6. The site proximity to the highway will need to be assessed.
7. Any issues with differential depths of foundations will need to be assessed.
8. Will the basement influence the quality of surface water being received by neighbouring properties?

Description of Site & Works:

The site for the proposed property is situated on Highfields Grove towards the Southern end of Fitzroy Park leading to The Grove, just off B519 (Hampstead Lane) in the London Borough of Camden. Highfields Grove is a residential street consisting of a varied mix of residential houses. The development proposal is for the construction of a 2 basements at the front and rear of the property partly underneath existing footprint of the existing structure.

This Basement Impact Assessment should only be used as a guide. Responsibility for site safety and the implementation of applicable building practices and British Standards are the responsibility of the Main

Contractor. This BIA is not exhaustive and assumes the Main Contractor has the competence and relevant experience to undertake building works of this nature.

Stage 2 - Scoping:

Subterranean Flow Screening issues:

1. The site is situated above an aquifer however the site investigations carried out suggest the development will not impact on this aquifer. ***No potential impact from the development.***
2. Although initial investigations suggest the basement will not enter the water table, the potential impact of this is to make construction techniques difficult and the sequencing would need to be altered if this is the case. Local boreholes suggest water table in excess of 10m.
3. An initial site walk and desk stud showed no surrounding signs of watercourses. The basement is therefore a minimum of 100m from a watercourse. ***No potential impact from the development.***
4. The site is located outside the catchment area for the chain ponds on Hampstead Heath. ***No potential impact from the development.***
5. The basement will not increase the current area of hardstand on the site. The basement extends out underneath the existing areas of hard standing of the currently approved scheme. ***No potential impact from the development.***
6. The addition of the basement will not increase the runoff requirements of the currently approved scheme therefore there will not be more surface water site drainage demands than currently approved. ***No potential impact from the development.***
7. Further investigations are required to establish the relationship between lowest level of excavation and any surrounding ponds/springs. The potential impact if the basement is lower is that water may flow from these areas into the excavation during works. Further investigations are required.

Slope Stability Screening issues:

1. The site is on hill and this has been assessed and allowed for within design. ***No potential impact from the development.***
2. Any re-profiling of the landscape will not include slopes in excess of 7”. ***No potential impact from the development.***
3. The site is away from neighboring properties and they will not be affected by the proposed construction. ***No potential impact from the development.***
4. The site in on a wider hillside with slopes in excess of 7”. ***No potential impact from the development.***
5. On site the London clay is covered by made ground. The shallowest stratum is therefore this made ground. ***No potential impact from the development.***
6. No trees will be felled as part of the basement works. The site includes a garden so there will be some clearance of existing vegetation but this will not impact on ground moisture levels given the nature and type of vegetation. ***No potential impact from the development.***

7. While the underlying soil type is London clay there are no signs of damage on the site or to surrounding properties from historical seasonal shrink/swell subsidence. ***No potential impact from the development.***
8. The top layer of soil on the site is made ground which by its nature is disturbed. However the basement will sit into the London clay underneath and will be unaffected by the top layers of soil. An initial site walk and desk stud showed no surrounding signs of watercourses. The basement is therefore a minimum of 100m from a watercourse. ***No potential impact from the development.***
9. The basement will sit over a potential aquifer but will not be below the water table. A dewatering system is therefore not required. ***No potential impact from the development.***
10. The site is not within 50m of Hampstead Heath ponds. ***No potential impact from the development.***
11. The site is within 100m of a highway. There is no potential damage here to the road way and underlying services. Additional surcharge loading will not need to be accounted for in the design.
12. The exact levels of the foundations of the closest neighbouring properties are unknown. However existing foundation distance away will not be undermined by development.
13. The site is not over or within exclusion zones of any tunnels. ***No potential impact from the development.***

Surface Flow screening issues:

1. The site is not within the catchment area of the pond chains on Hampstead Heath. ***No potential impact from the development.***
2. The proposed works will not increase the surface water drainage requirement over and above the existing approved scheme. ***No potential impact from the development.***
3. The basement will not increase the current area of hardstanding on the site. The basement extends out underneath the existing areas of hard standing and rear and front garden of the currently approved scheme. The site is situated on a hill and this will not be an issue for surface flow. ***No potential impact from the development.***
4. The basement sits under a detached property and therefore the influence of it on the flows of surface water will be minimal and not impact on the profile of inflows to adjacent properties. ***No potential impact from the development.***
5. The basement should have no influence on the quality of surface water being received by adjacent properties or downstream watercourses. ***No potential impact from the development.***

Conceptual ground model:

The site is in London. The geology of the locality comprises made ground overlying London Clay. The latter is more than 70 metres thick and beneath it are the Lambeth Group, Thanet Sand and Chalk which together make up the Lower Aquifer. This information can be obtained from the 1:50,000 geological maps and the Geological Memoir for London. The London Clay is sufficiently thick that it isolates the strata of the Lower Aquifer from any shallow groundwater and surface water systems: the strata of relevance are the made ground and the surface of the London Clay.

The site is located on hill and there is no issue with run off. A proportion of the rainfall incident on this ground will run off, a proportion will evaporate, and a proportion will be retained in the soil and root layer near the surface, and some will percolate down and enter a shallow groundwater system. There are no perennial streams within several hundred metres of the property, and the ground is what a farmer or gardener would describe as well-drained. If there is a water table, it is likely to be 10 metres below ground surface. The slope of the land surface is quite flat and therefore groundwater flows are likely to be small and slow. The introduction of the basement is unlikely to influence the flows greatly.
The houses on either side of the proposed new basement development potentially have existing basements. There would however be sufficient space between all basements for groundwater to pass through the gap between the two houses. It is unlikely that any effect would extend further than a few metres beyond the house.

Stage 3 - Site Investigation & Desk Study:

Ground Conditions:

Local knowledge of the area backed up by the results of a site investigation (attached in appendix C) on the site suggest the underlying soil to be made ground (to 1m) over London clay (1m to 129m).The water table was not encountered in either borehole above 10m and therefore the lowest extent of the basement will be above the groundwater level. If measures to counteract any occasional uplift are required, this will require additional reinforcement of the ground slab and tying the slab into the retaining structure. Generally low surcharge loads can be counteracted by the self weight of the slab itself. As the water table is well below the construction zone, no measures need to be taken to drain the site during construction.

Given the depths at which the water table appears in excess of 10m and the proposed depth to which it is planned to excavate the sub levels, it is safe to conclude there will be no adverse affects by the development to the local hydrology of the area.

A site walk has established also that the levels of the lowest formation of the basement will be above any surrounding springs or watercourse which would have the potential to flow into the works during excavations. New basement formation level will match existing foundation levels.

Clays, in particular the London clays, are considered to stand up well for the proposed type of construction and can easily assume bearing pressures in excess of 150kn/m^2 which has been assumed in the design of both the temporary works and permanent retaining structures. We have constructed similar basements using the proposed typical basement retaining wall techniques.

A desk top investigation has been carried out in order to establish the positions of any underground utilities, main drainage or infrastructure to ensure no impact on these. Investigations suggest that none are present however; the contractor should carry out works under the assumption that there may be, taking all necessary precautions. It will be necessary to carry out some works to the drainage locally within the curtilage of the development to allow for the new requirements on both surface and foul water drainage of the new layouts but these will not impact in any way on the neighboring properties.

The desktop study also showed the proximity of the development to the highway on Hampstead Lane. As outside the zone of influence it is decided that that highways surcharge loading should not be taken into account in the design for the basement construction. The surcharge loadings may need to be increase on the side of the closest neighbouring property to ensure settlements are limited to ensure no damage is caused by the differential level of the foundations.

The depth of construction is approximately 3.5m below the existing surface level and if the basement is constructed as per the suggested method on drawings, then temporary works should not be required. The contractor is advised to have some sheeting available to deal with any unexpected pockets of poor ground.

The attached site soil investigation report would seem to agree with the above discussions with London clay encountered approximately 1m below garden ground level.

Monitoring:

While preliminary analysis (maximum category 1 damage is to be expected) was carried out on the potential impact on the surrounding properties, it is suggested that a monitoring system be put in place prior to the works to ensure that any potential issue are discovered as soon as possible. The contractor will need to provide a detail statement of how this is to be achieved along with a triggering system in line with BRE and CIRIA guidelines.

Stage 4 - Impact Assessment:

General Comments:

We do not anticipate any significant damage (max category 1 in line with CIRIA C580) to adjoining structures as a consequence of these works if carried out in the approved manner as described above by competent contractors. There should not be any impact on the integrity of the neighbouring structures. Due to the soil conditions determined from detailed site investigations, stiff clay gives a safe bearing pressure in excess of 150kN/m^2; we do not anticipate any significant settlement following the excavation. There will be no slope stability issues as a result of the development. The proposed structure is a reinforced concrete retaining wall with reinforced concrete slab supported by the piles, this form of construction will provide adequate support to the adjoining gardens and structures and we anticipate no adverse effects on the surrounding properties.

In addition, detailed investigation of the local watercourse, spring and ponds suggest that these all lie well outside the zone of influence of the proposed development and will therefore not be affect by the works as currently proposed. Within the site boundary, investigations show the water table level to be below the formation level of the works but this is to be continually monitored to ensure adjustment in seasonality don't change this fact.

The appendices of this report show the results of the site investigation and assessment of the potential impact of the works on surrounding buildings and the local watercourses. The additional reports back up the decisions and discussion above.

The new excavation is remote from the drainage of the adjacent structures and will have no impact on them.

There are a number of small trees surrounding the development but consideration of the protection of the root zone has been undertaken and we consider that all these trees will remain unaffected by the works.

Waterproofing, insulation and fit out will follow completion of the reinforced concrete structure.

Minimal temporary works are necessary for the proposed basement excavation as the structure has been developed to allow for all loading which may occur during both the construction phases and the permanent load cases.

In summary all potential impacts have been assessed in accordance with the screening and scoping flowcharts and were necessary the design has been adjusted to mitigate or allow for the reduction of any potential negative impacts.

Site Soil Investigation Report:

Site Investigation Report

Client:	Safran Holding Ltd	**Scale:**	N.T.S	**Sheet No:**	1 of 1	**Weather:**	Overcast	**Date:**	04.09.14
Site:	5 Highfields Grove, London, SN6	**Job No:**	4954	**Borehole No:**	1	**Boring method:**	GEO 205 CFA		

Depth Mtrs.	Description of strata	Thick -ness	Legend	Sample	Test Type	Test Result	Root Information	Depth to water	Depth Mtrs.
0.00	MADE UP GROUND: medium compact, dark brown, gravelly sandy silt with numerous concrete and brick fragments	1.0		D	CPT N =21		Roots of live appearance to 1mmØ to 2.0m		0.5
1.00				D					1.0
				D					1.5
				D			No roots observed below 2.0m	10.0	2.0
				D					2.5
				D					4.0
	CLAY: stiff, mid brown, sandy, with occasional gavel	128			CPT N =20				
				D					5.5
129.0	THANET SANDS: medium dense, mid brown, gravely coarse			D					8.0
		18		D	CPT N =18				8.5
147.0	CHALK WITH FLINTS	57			CPT N =17				
				D					9.5
204.0	Borehole ends at 204m								

Drawn by: MM **Approved by:** ME

Remarks: Groundwater strike at 10.0m. Groundwater standing at 11.0 at completion. Borehole open on completion. Standpipe installed at 5.4m. Unable to extract U100 samples due to ground conditions

Key T.D./T.D. Too Dense to [illegible]
D Small Disturbed Sample
B Bulk Disturbed Sample
U Undisturbed Sample (U100)
W Water Sample
J Jar Sample
V Pilcon Vane (kPa)
M Mackintosh Probe
N Standard Penetration Test Blow Count

Appendix D: Damage category classification from CIRIA C580

Category of damage		Description of typical damage (ease of repair is underlined)	Approximate crack width (mm)	Limiting tensile strain %
0	Negligible	Hairline cracks of less than about 0.1mm are classes as negligible.	<0.1	0.0-0.05
1	Very Slight	Fine cracks that can easily be treated during normal decoration. Perhaps isolated slight fracture in building. Cracks in external brickwork visible on inspection.	<1	0.05-0.075
2	Slight	Cracks easily filled. Redecoration probably required. Several slight fractures showing inside of building. Cracks are visible externally and some repointing may be required externally to ensure weathertightness. Doors and windows may stick slightly.	<5	0.075-0.15
3	Moderate	The cracks require some opening up and can be patched by a mason. Recurrent cracks can be masked by suitable linings. Repointing of external brickwork and possibly a small amount of brickwork to be replaced. Doors and windows sticking. Service pipes may fracture. Weathertightness often impaired.	5-15 or a number of cracks >3	0.15-0.3
4	Severe	Extensive repair work involving breaking-out and replacing sections of walls, especially over doors and windows. Windows and frames distorted, floors sloping noticeably. Walls leaning or bulging noticeably, some loss of bearing in beams. Service pipes disrupted.	15-25 but also depends on number of cracks	>0.3
5	Very Severe	This requires a major repair involving partial or complete rebuilding. Beams lose bearings, walls lean badly and require shoring. Windows broken with distortion. Danger of instability.	Usually >25 but depends on number of cracks	